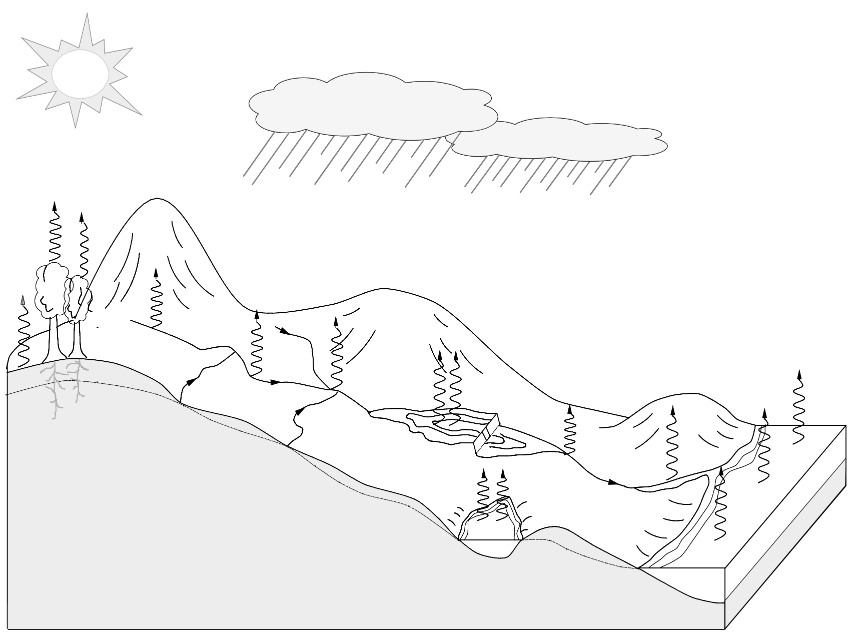
US Army Corps
of Engineers
Hydrologic Engineering Center

Hydrologic Modeling System HEC-HMS



Technical Reference Manual

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13. ABSTRACT *(Maximum 200 words)*
The Hydrologic Modeling System (HEC-HMS) is designed to simulate the precipitation-runoff processes of dendritic watershed systems. It supercedes HEC-1 and provides a similar variety of options but represents a significant advancement in terms of both computer science and hydrologic engineering. The program includes a variety of mathematical models for simulating precipitation, evapotranspiration, infiltration, excess precipitation transformation, baseflow, and open channel routing. A versatile parameter estimation option is also included to assist in calibrating the various models.

This technical reference manual describes the mathematical models that are included as part of the computer program, including equation derivations and solution algorithms where appropriate. In addition, the manual provides information and guidance regarding how and when to use the various models. Guidance for estimating a model's parameters with measured field data, empirical relationships, and the parameter estimation option is included.

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Hydrologic Modeling System HEC-HMS, User's Manual

The above information and the full License Agreement are presented for acceptance when the software is first used. That information is also provided in Appendix F of this manual for reference.

PREFACE

The HEC-HMS program was developed at the Hydrologic Engineering Center (HEC) of the US Army Corps of Engineers. HEC-HMS is a component of the HEC *Next Generation Software Development Project*. This project is under the guidance of Darryl Davis, Director, HEC. Arlen Feldman manages the HEC-HMS project.

The program was developed by a team of HEC staff and consultants. Elisabeth Pray, HEC, developed the majority of the graphical user interface and integrated the various components to produce the finished program. Paul Ely, contractor, developed the computation engine and hydrologic algorithm library. William Scharffenberg, HEC, contributed to graphical user interface design, managed testing, and wrote the program user’s manual. Thomas Evans, HEC, developed the algorithms for storing gridded data. Richard Raichle, contractor, developed the soil moisture accounting graphical user interface. Shannon Newbold, contractor, developed the meteorological model graphical user interface. Todd Bennett, HEC, provided technical evaluations that led to the design of the soil moisture accounting loss method. Jessica Thomas, HEC, conducted testing and prepared validation documents.

The program continues to benefit from many individuals who contributed to previous versions. John Peters, HEC, managed the development team until his retirement in 1998. During that time he designed the graphical user interface, managed development, and wrote the first version of the user's manual. Arthur Pabst, HEC, and Tony Slocum, consultant, provided essential input to the object-oriented design of the program. Slocum also wrote the code for the schematic representation of the basin model. William Charley, HEC, developed the design for the computation engine. David Ford Consulting Engineers provided recommendations for the scope and content of the optimization manager. Troy Nicolini, HEC, led the Version 1.0 beta testing team and managed the maiden release. Several students from the University of California, Davis, working as temporary employees at HEC, provided excellent assistance to the software development, testing, and documentation: Ken Sheppard, Jake Gusman, and Dan Easton.

David Ford Consulting Engineers wrote original drafts of this manual, supplementing material provided by HEC with new, original text and figures. HEC staff reviewed and modified the drafts to produce the final manual.

CHAPTER 1

Introduction

What's in this Manual?

This document is the technical reference manual for the Hydrologic Modeling System (HEC-HMS). The program is a product of the US Army Corps of Engineers' research and development program, and is produced by the Hydrologic Engineering Center (HEC). The program simulates precipitation-runoff and routing processes, both natural and controlled. The program is the successor to and replacement for the Flood Hydrograph Package HEC-1 (USACE, 1998) and for various specialized versions of HEC-1. The program improves upon the capabilities of HEC-1 and provides additional capabilities for distributed modeling and continuous simulation.

This technical reference manual describes the mathematical models that are included as part of the program. In addition, the manual provides information and guidance regarding how and when to use the models and how to estimate a model's parameters.

The presentation of the models is aimed at an engineer or scientist who has studied hydrology in a university-level course. Thus, examples of common models are not provided; such information may be found by consulting available texts and journals. On the other hand, examples of the computations for the new or uncommon models within the program are included.

Program Overview

For precipitation-runoff-routing simulation, the program provides the following components:

- Precipitation-specification options which can describe an observed (historical) precipitation event, a frequency-based hypothetical precipitation event, or a event that represents the upper limit of precipitation possible at a given location.
- Loss models which can estimate the volume of runoff, given the precipitation and properties of the watershed.
- Direct runoff models that can account for overland flow, storage and energy losses as water runs off a watershed and into the stream channels.
- Hydrologic routing models that account for storage and energy flux as water moves through stream channels.
- Models of naturally occurring confluences and bifurcations.
- Models of water-control measures, including diversions and storage facilities.

These models are similar to those included in HEC-1. In addition to these, the program includes:

- A distributed runoff model for use with distributed precipitation data, such as the data available from weather radar.
- A continuous soil-moisture-accounting model used to simulate the long-term response of a watershed to wetting and drying.

The program also includes:

- An automatic calibration package that can estimate certain model parameters and initial conditions, given observations of hydrometeorological conditions.

Links to a database management system that permits data storage, retrieval and connectivity with other analysis tools available from HEC and other sources.

Other Program References

Two references are available in addition to this manual:

- The *Hydrologic Modeling System HEC-HMS User's Manual* (USACE, 1998b) describes how to use the computer program. While the user's manual identifies the models that are included in the program, its focus is the program's user interface. Thus, the user's manual provides a description of how to use the interface to provide data, to specify model parameters, to execute the program, and to review the results. It provides examples of all of these tasks.
- The *HEC-HMS on-line help system* is a component of the computer program. It is essentially an electronic version of the user's manual, but it also includes some material from this reference manual. Because it is in electronic form, text searches for keywords, and jumping from topic to topic using hyperlinks are possible.

The user's manual and the HEC-HMS program are available on the Hydrologic Engineering Center's web site. The address is *www.hec.usace.army.mil*.

Organization of this Manual

Table 1 shows how this manual is organized. Chapters 4-8 and 10 present the equations of the models, define the terms of the equations, and explain the solution algorithms used in the program. In addition, parameters of the models and methods for estimating the parameter are described.

Because of the importance of model calibration, Chapter 9 describes the automated calibration feature of the program in detail. This can be used to estimate model parameters with measured precipitation and streamflow.

Table 1. Summary of contents of HEC-HMS Technical Reference Manual.

Chapter	Topic	Description of Contents
1	Introduction	Provides an overview of HEC-HMS and the technical reference manual
2	Primer on precipitation-runoff-routing simulation models	Defines terms used throughout the manual and describes basic concepts and components of models
3	HEC-HMS components	Describes how HEC-HMS represents the runoff process and identifies the models that are included in the program
4	Describing precipitation for modeling with HEC-HMS	Identifies each type of precipitation event that may be analyzed with HEC-HMS, describes the format of the data for each, and presents the precipitation processing algorithms
5	Computing runoff volumes with HEC-HMS	Summarizes the models that are included for estimating runoff volume, given precipitation
6	Modeling direct runoff with HEC-HMS	Summarizes the models available in HEC-HMS for computing runoff hydrographs, given runoff volume
7	Modeling baseflow with HEC-HMS	Describes the HEC-HMS model of sub-surface flow
8	Modeling channel flow with HEC-HMS	Describes the alternative models of open channel flow that are available and provides guidance for usage
9	Calibrating the HEC-HMS models	Describes how HEC-HMS may be calibrated with historical precipitation and runoff data
10	Modeling water-control measures	Describes the HEC-HMS models of diversion and detention
Appx A	CN tables	Tables of parameters for SCS loss model
Appx B	SMA model details	More information about the HEC-HMS soil-moisture accounting model
Appx C	Glossary	Briefly defines important terms

References

US Army Corps of Engineers, USACE (1998). *HEC-1 flood hydrograph package user's manual.* Hydrologic Engineering Center, Davis, CA.

USACE (2000). *HEC-HMS hydrologic modeling system user's manual*. Hydrologic Engineering Center, Davis, CA.

CHAPTER 2

Primer on Models

This chapter explains basic concepts of modeling and the most important properties of models. It also defines essential terms used throughout this technical reference manual.

What is a Model?

Hydrologic engineers are called upon to provide information for activities for a variety of water resource studies:

- Planning and designing new hydraulic-conveyance and water-control facilities.
- Operating and/or evaluating existing hydraulic-conveyance and water-control facilities.
- Preparing for and responding to floods.
- Regulating floodplain activities.

In rare cases, the record of historical flow, stage or precipitation satisfies the information need. More commonly, watershed runoff must be predicted to provide the information. For example, a flood-damage reduction study may require an estimate of the increased volume of runoff for proposed changes to land use in a watershed. However, no record will be available to provide this information because the change has not yet taken place. Similarly, a forecast of reservoir inflow may be needed to determine releases if a tropical storm alters its course and moves over a watershed. Waiting to observe the flow is not acceptable. The alternative is to use a *model* to provide the information.

A model relates something unknown (the output) to something known (the input). In the case of the models that are included in the program, the known input is precipitation and the unknown output is runoff, or the known input is upstream flow and the unknown output is downstream flow.

Model Classification

Models take a variety of forms. *Physical models* are reduced-dimension representations of real world systems. A physical model of a watershed, such as the model constructed in the lab at Colorado State University, is a large surface with overhead sprinkling devices that simulate the precipitation input. The surface can be altered to simulate various land uses, soil types, surface slopes, and so on; and the rainfall rate can be controlled. The runoff can be measured, as the system is closed. A more common application of a physical model is simulation of open channel flow. The Corps of Engineers Waterways Experiment Station has constructed many such models and used these to provide information for answering questions about flow in complex hydraulic systems.

Researchers also have developed *analog models* that represent the flow of water with the flow of electricity in a circuit. With those models, the input is controlled by adjusting the amperage, and the output is measured with a voltmeter. Historically, analog models have been used to calculate subsurface flow.

The program includes models in a third category—*mathematical models*. In this manual, that term defines an equation or a set of equations that represents the response of a hydrologic system component to a change in hydrometeorological conditions. Table 2 shows some other definitions of mathematical models; each of these applies to the models included in the program.

Table 2. What is a mathematical model?

…a quantitative expression of a process or phenomenon one is observing, analyzing, or predicting (Overton and Meadows, 1976)
…simplified systems that are used to represent real-life systems and may be substitutes of the real systems for certain purposes. The models express formalized concepts of the real systems (Diskin, 1970)
…a symbolic, usually mathematical representation of an idealized situation that has the important structural properties of the real system. A theoretical model includes a set of general laws or theoretical principles and a set of statements of empirical circumstances. An empirical model omits the general laws and is in reality a representation of the data (Woolhiser and Brakensiek, 1982)
…idealized representations…They consist of mathematical relationships that state a theory or hypothesis (Meta Systems, 1971)

Mathematical models, including those that are included in the program, can be classified using the criteria shown in Table 3. These focus on the mechanics of the model: how it deals with time, how it addresses randomness, and so on. While knowledge of this classification is not necessary to use the program, it is helpful in deciding *which* of the models to use for various applications. For example, if the goal is to create a model for predicting runoff from an ungaged watershed, the fitted-parameter models included in the program that require unavailable data are a poor choice. For long-term runoff forecasting, use a continuous model, rather than a single-event model; the former will account for system changes between rainfall events, while the latter will not.

Table 3. Categorization of mathematical models (from Ford and Hamilton, 1996)

Category	Description
Event or continuous	This distinction applies primarily to models of watershed-runoff processes. An event model simulates a single storm. The duration of the storm may range from a few hours to a few days. A continuous model simulates a longer period, predicting watershed response both during and between precipitation events. Most of the models included in HEC-HMS are event models.
Lumped or distributed	A *distributed model* is one in which the spatial (geographic) variations of characteristics and processes are considered explicitly, while in a *lumped model*, these spatial variations are averaged or ignored. HEC-HMS includes primarily lumped models. The ModClark model is an exception.
Empirical (system theoretic) or conceptual	This distinction focuses on the knowledge base upon which the mathematical models are built. A *conceptual model* is built upon a base of knowledge of the pertinent physical, chemical, and biological processes that act on the input to produce the output. An *empirical model*, on the other hand, is built upon observation of input and output, without seeking to represent explicitly the process of conversion. HEC-HMS includes both empirical and conceptual models. For example, Snyder's unit hydrograph (UH) model is empirical: the model is fitted with observed precipitation and runoff. The kinematic-wave runoff model is conceptual: it is based upon fundamental principles of shallow free-surface flow.
Deterministic or stochastic	If all input, parameters, and processes in a model are considered free of random variation and known with certainty, then the model is a *deterministic model*. If instead the model describes the random variation and incorporates the description in the predictions of output, the model is a *stochastic model*. All models included in HEC-HMS are deterministic.
Measured-parameter or fitted-parameter	This distinction is critical in selecting models for application when observations of input and output are unavailable. A *measured-parameter model* is one in which model parameters can be determined from system properties, either by direct measurement or by indirect methods that are based upon the measurements. A *fitted-parameter model*, on the other hand, includes parameters that cannot be measured. Instead, the parameters must be found by fitting the model with observed values of the input and the output. HEC-HMS includes both measured-parameter models and fitted-parameter models. For example, the baseflow model of Chapter 7 is empirical, so its parameters cannot be measured. Instead, for a selected watershed, the baseflow-model parameters are found by calibration, as described in Chapter 9. On the other hand, the Green and Ampt loss model of Chapter 5 has parameters that are based upon soil characteristics that can be sampled.

Constituents of a Model

The mathematical models that are included in the program describe how a watershed responds to precipitation falling on it or to upstream water flowing into it. While the equations and the solution procedures vary, all the models have the various components in common.

State Variables

These terms in the model's equations represent the state of the hydrologic system at a particular time and location. For example, the deficit and constant-rate loss model that is described in Chapter 5 tracks the mean volume of water in natural storage in the watershed. This volume is represented by a state variable in the deficit and constant-rate loss model's equations. Likewise, in the detention model of Chapter 10, the pond storage at any time is a state variable; the variable describes the state of the engineered storage system.

Parameters

These are numerical measures of the properties of the real-world system. They control the relationship of the system input to system output. An example of this is the constant rate that is a constituent of the runoff-volume-accounting model described in Chapter 5. This rate, a single number specified when using the model, represents complex properties of the real-world soil system. If the number increases, the computed runoff volume will decrease. If the number decreases, the runoff volume will increase.

Parameters can be considered the "tuning knobs" of a model. The parameter values are adjusted so that the model accurately predicts the physical system response. For example, the Snyder unit hydrograph model has two parameters, the basin lag, *tp*, and peaking coefficient, *Cp*. The values of these parameters can be adjusted to "fit" the model to a particular physical system. Adjusting the values is referred to as calibration. Calibration is discussed in Chapter 9.

Parameters may have obvious physical significance, or they may be purely empirical. For example, the Muskingum-Cunge channel model includes the channel slope, a physically significant, measurable parameter. On the other hand, the Snyder unit hydrograph model has a peaking coefficient, *Cp*. This parameter has no direct relationship to any physical property; it can only be estimated by calibration.

Boundary Conditions

These are the values of the system input—the forces that act on the hydrologic system and cause it to change. The most common boundary condition in the program is precipitation; applying this boundary condition causes runoff from a watershed. Another example is the upstream (inflow) flow hydrograph to a channel reach; this is the boundary condition for a routing model.

Initial Conditions

All models included in the program are unsteady-flow models; that is, they describe changes in flow over time. They do so by solving, in some form, differential equations that describe a component of the hydrologic system. For example, the routing models that are described in Chapter 8 solve the differential equations that describe, in one dimension, the flow of water in an open channel.

Solving the Constituents

The solution of any differential equation is a report of how much the output changes with respect to changes in the input, the parameters, and other critical variables in the modeled process. For example, the solution of the routing equations will tell us the value of $\Delta Q/\Delta t$, the rate of change of flow with respect to time. But in using the models for planning, designing, operating, responding, or regulating, the flow values at various times are needed, not just the rate of change. Given an initial value of flow, Q at some time t, in addition to the rate of change, then the required values are computed using the following equation in a recursive fashion:

$$Q_t = Q_{t-\Delta t} + (\Delta Q / \Delta t) \tag{1}$$

In this equation, $Q_{t\text{ -}\Delta t}$ is the *initial condition*; the known value with which the computations start.

The initial conditions must be specified to use any of the models included in the program. With the volume-computation models, the initial conditions represent the initial state of soil moisture in the watershed. With the runoff models, the initial conditions represent the runoff at the start of the storm being analyzed. With the routing models, initial conditions represent the flows in the channel at the start of the storm. Moreover, with the models of detention storage, the initial condition is the state of storage at the beginning of the runoff event.

Models and Computer Programs

For clarity, this manual makes a distinction between a *mathematical model*, a *computer program* and the *input* to a computer program. These terms are used as follows:

Model

As noted above, the term model means the equations that represent the behavior of hydrologic system components. For example, the combination of the continuity and momentum equations together form a model of open-channel flow for routing.

Program

If the equations of a mathematical model are too numerous or too complex to solve with pencil, paper, and calculator, they are translated into computer code and an appropriate equation solver (an algorithm) is used. The result is a computer program. Thus, HEC-HMS is a computer program that includes a variety of models.

Programs may be classified broadly as those developed for a specific set of parameters, boundary conditions or initial conditions, and those that are data-driven. Programs in the first category are "hard wired" to represent the system of interest. To change parameters, boundary conditions or initial conditions, the program code must be changed and recompiled. HEC-HMS is in the second category of programs—those that require no such changes. Instead, these program are tailored to the system of interest through changes to data in a database or changes to parameters, boundary conditions, or initial conditions in the input.

Input

When the equations of a mathematical model are solved with site-specific conditions and parameters, the model simulates the processes and predicts what will happen within a particular watershed or hydrologic system. In this manual, this is referred to as an application of the model. In using a program to solve the equations of the model, input to the program is necessary. With HEC-1, the predecessor to HEC-HMS, the input is an ASCII text file. This text file includes codes that specify which models and parameters, initial conditions, and boundary conditions to use. With HEC-HMS, the same or similar information is supplied by completing forms in the graphical user interface. The input may also include data from an HEC-DSS database (USACE, 1995).

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CHAPTER 3

Program Components

This chapter describes how the models included in the program conceptually represent watershed behavior. It also identifies and categorizes these models.

Runoff Processes

Figure 1 is a systems diagram of the watershed runoff process, at a scale that is consistent with the scale modeled well with the program. The processes illustrated begin with precipitation. (Currently precipitation is limited to analysis of runoff from rainfall. Subsequent versions will provide capability to analyze snowmelt also.) In the simple conceptualization shown, the precipitation can fall on the watershed's vegetation, land surface, and water bodies (streams and lakes).

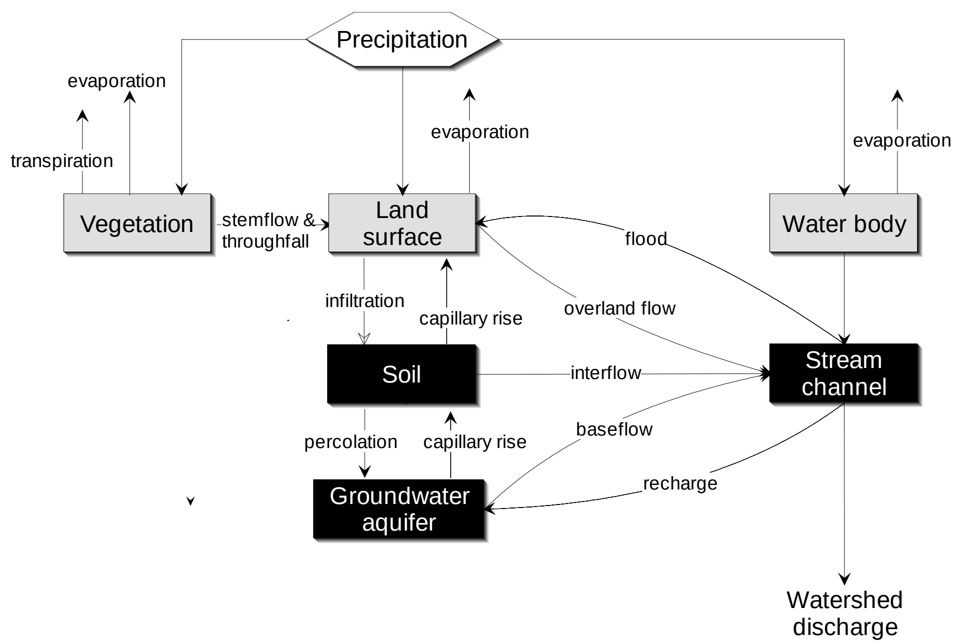

Figure 1. Systems diagram of the runoff process at local scale (after Ward, 1975).

In the natural hydrologic system, much of the water that falls as precipitation returns to the atmosphere through evaporation from vegetation, land surfaces, and water bodies and through transpiration from vegetation. During a storm event, this evaporation and transpiration is limited.

Some precipitation on vegetation falls through the leaves or runs down stems, branches and trunks to the land surface, where it joins the precipitation that fell directly onto the surface. There, the water may pond, and depending upon the soil type, ground cover, antecedent moisture and other watershed properties, a portion may infiltrate. This infiltrated water is stored temporarily in the upper, partially saturated layers of soil. From there, it rises to the surface again by capillary action, moves horizontally as interflow just beneath the surface, or it percolates vertically to the groundwater aquifer beneath the watershed. The interflow eventually moves into the stream channel. Water in the aquifer moves slowly, but eventually, some returns to the channels as baseflow.

Water that does not pond or infiltrate moves by overland flow to a stream channel. The stream channel is the combination point for the overland flow, the precipitation that falls directly on water bodies in the watershed, and the interflow and baseflow. Thus, resultant streamflow is the total watershed outflow.

Representation of the Runoff Process

The appropriate representation of the system shown in Figure 1 depends upon the information needs of a hydrologic-engineering study. For some analyses, a detailed accounting of the movement and storage of water through all components of the system is required. For example, to estimate changes due to watershed land use changes, it may be appropriated to use a long record of precipitation to construct a corresponding long record of runoff, which can be statistically analyzed. In that case, evapotranspiration, infiltration, percolation, and other movement and storage should be tracked over a long period. To do so, a detailed accounting model is required. The program includes such a model.

On the other hand, such a detailed accounting is not necessary for many of the reasons for conducting a water resources study. For example, if the goal of a study is to determine the area inundated by a storm of selected risk, a detailed accounting and reporting of the amount of water stored in the upper soil layers is not needed. Instead, the model need only compute and report the peak, or the volume, or the hydrograph of watershed runoff. In this and similar cases, the "view" of the hydrologic process can be somewhat simpler. Then, as illustrated in Figure 2, only those components necessary to predict runoff are represented in detail, and the other components are omitted or lumped. For example, in a common application, detailed accounting of movement of water within the soil can be omitted. In this "reductionist" mode, the program is configured to include models of infiltration from the land surface, but it does not model storage and movement of water vertically within the soil layer. It implicitly combines the near surface flow and overland flow and models this as direct runoff. It does not include a detailed model of interflow or flow in the groundwater aquifer, instead representing only the combined outflow as baseflow.

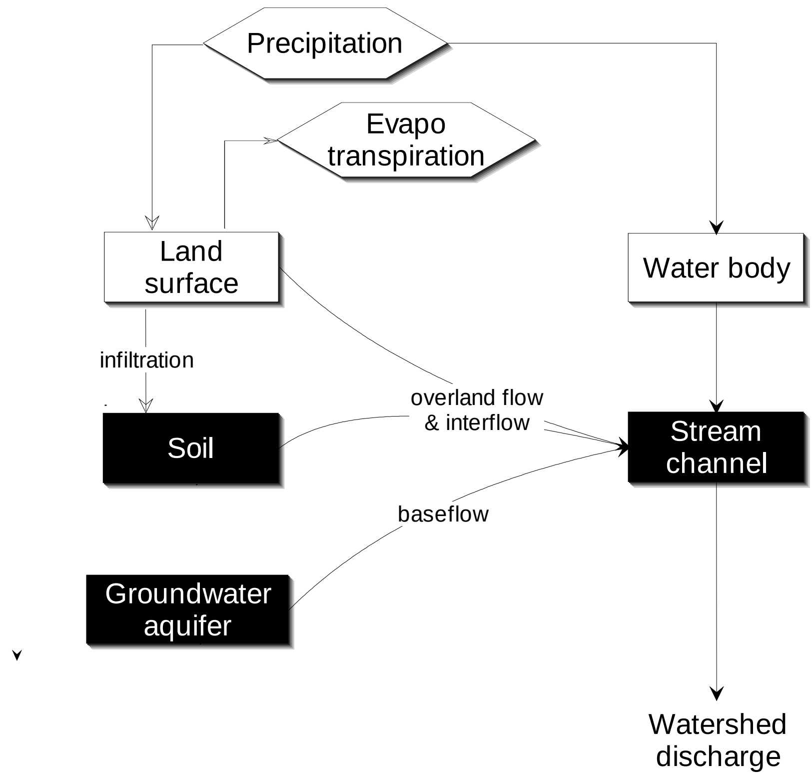

Figure 2. Typical representation of watershed runoff.

Synopsis of Included Models

The program uses a separate model to represent each component of the runoff process that is illustrated in Figure 2, including:

- Models that compute runoff volume.
- Models of direct runoff, including overland flow and interflow.
- Models of baseflow.
- Models of channel flow.

The models that compute runoff volume are listed in Table 4. Refer to Table 3 for definitions of the categorizations. These models address questions about the volume of precipitation that falls on the watershed: How much infiltrates on pervious surfaces? How much runs off of the impervious surfaces? When does it run off?

The models of direct runoff are listed in Table 5. These models describe what happens as water that has not infiltrated or been stored on the watershed moves over or just beneath the watershed surface. Table 6 lists the models of baseflow. These simulate the slow subsurface drainage of water from the system into the channels.

Table 4. Runoff-volume models.

Model	Categorization
Initial and constant-rate	event, lumped, empirical, fitted parameter
SCS curve number (CN)	event, lumped, empirical, fitted parameter
Gridded SCS CN	event, distributed, empirical, fitted parameter
Green and Ampt	event, distributed, empirical, fitted parameter
Deficit and constant rate	continuous, lumped, empirical, fitted parameter
Soil moisture accounting (SMA)	continuous, lumped, empirical, fitted parameter
Gridded SMA	continuous, distributed, empirical, fitted parameter

Table 5. Direct-runoff models.

Model	Categorization
User-specified unit hydrograph (UH)	event, lumped, empirical, fitted parameter
Clark's UH	event, lumped, empirical, fitted parameter
Snyder's UH	event, lumped, empirical, fitted parameter
SCS UH	event, lumped, empirical, fitted parameter
ModClark	event, distributed, empirical, fitted parameter
Kinematic wave	event, lumped, conceptual, measured parameter
User-specified unit hydrograph (UH)	event, lumped, empirical, fitted parameter

Table 6. Baseflow models.

Model	Categorization
Constant monthly	event, lumped, empirical, fitted parameter
Exponential recession	event, lumped, empirical, fitted parameter
Linear reservoir	event, lumped, empirical, fitted parameter

The choices for modeling channel flow with HEC-HMS are listed in Table 7. These so-called routing models simulate one-dimensional open channel flow.

Table 7. Routing models.

Model	Categorization
Kinematic wave	event, lumped, conceptual, measured parameter
Lag	event, lumped, empirical, fitted parameter
Modified Puls	event, lumped, empirical, fitted parameter
Muskingum	event, lumped, empirical, fitted parameter
Muskingum-Cunge Standard Section	event, lumped, quasi-conceptual, measured parameter
Muskingum-Cunge 8-point Section	event, lumped, quasi-conceptual, measured parameter
Confluence	continuous, conceptual, measured parameter
Bifurcation	continuous, conceptual, measured

parameter

In addition to the models of runoff and channel processes, models are included for simulating a water control structure such as a diversion or a reservoir/detention pond. Those models are described in Chapter 10.

Program Setup and Application

To analyze a hydrologic system, the program user must complete the following steps:

1. Start a new project.
2. Create gage data.
3. Enter basin model data.
4. Enter precipitation model data.
5. Enter control specifications.
6. Create and execute a "run" (an application of the program.)
7. View the results.
8. Exit the program.

To complete step 3, the user must select the models that will be used for the analysis. This requires a volume model from Table 4, a direct-runoff model from Table 5, and a baseflow model from Table 6. For routing computations, a routing model from Table 7 must be selected. For each model, the user must specify the initial conditions and the model parameters.

For step 4, the user must select the appropriate form of precipitation, the boundary condition for a rainfall-runoff model. To make this selection properly, the user must answer the question: Does historical observed rainfall provide the necessary information, or is an event with specified frequency needed? These alternatives are described in more detail in Chapter 4 of this document.

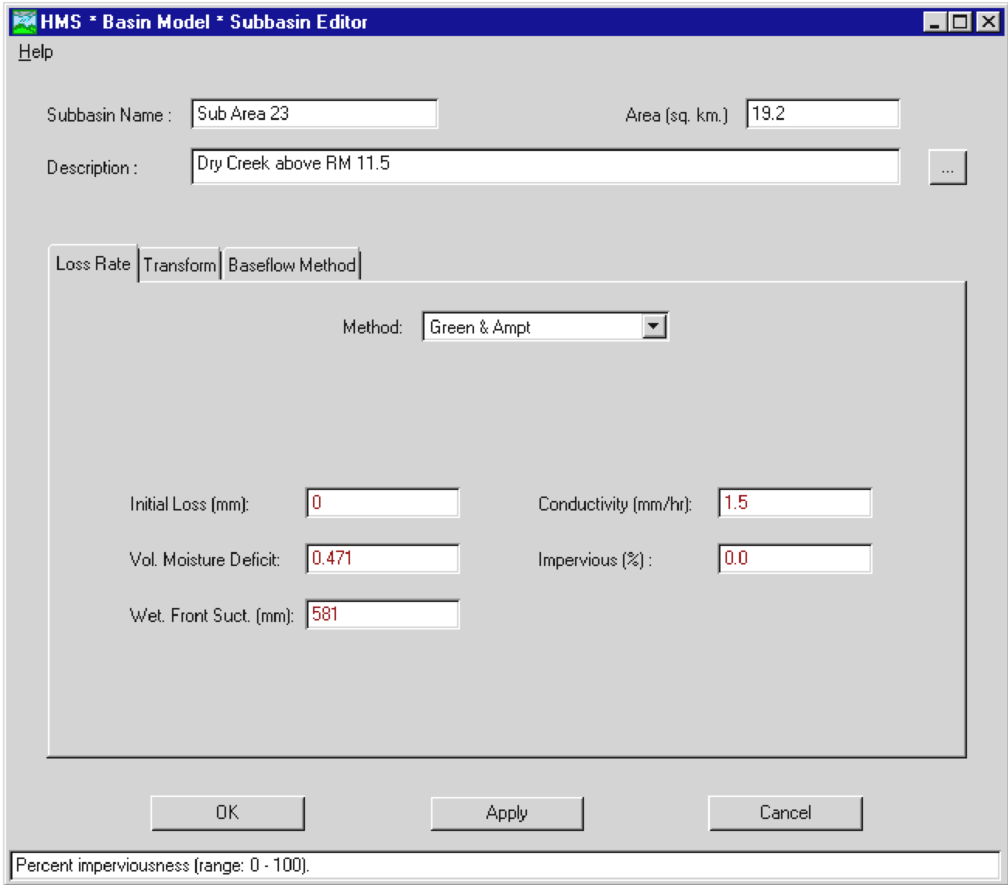

Figure 3. Example input screen for runoff-model parameters.

The data-entry steps, program execution, and result visualization are easy. The user indicates model choices and specifies initial conditions and parameters using a graphical user interface (GUI). With this GUI, a user can start a project; draw on the screen a schematic of the watershed; fill in forms to specify basin-model information, precipitation-model information, and control specifications (as illustrated in Figure 3); run the models; and view the results. The HEC-HMS user's manual (USACE, 2000) and the on-line help system provide additional details about this.

References

USACE (1998). *HEC-HMS user's manual*. Hydrologic Engineering Center, Davis, CA.

Ward R.C. (1975). *Principles of hydrology*. McGraw-Hill Book Company (UK) Limited, London.

CHAPTER 4

Describing Precipitation

In the view of watershed hydrology, as illustrated by Figure 2, the response of a watershed is driven by precipitation that falls on the watershed and evapotranspiration from the watershed. The precipitation may be observed rainfall from a historical event, it may be a frequency-based hypothetical rainfall event, or it may be an event that represents the upper limit of precipitation possible at a given location. (In future versions of the program, the precipitation may also be snowmelt.) Historical precipitation data are useful for calibration and verification of model parameters, for real-time forecasting, and for evaluating the performance of proposed designs or regulations. Data from the second and third categories—commonly referred to as *hypothetical* or *design storms*—are useful if performance must be tested with events that are outside the range of observations or if the risk of flooding must be described. Similarly, the evapotranspiration data used may be observed values from a historical record, or they may be hypothetical values. This chapter describes methods of specifying and analyzing historical or hypothetical-storm precipitation and evapotranspiration.

Field-Monitored Historical Precipitation

Precipitation Measurement

Each of the precipitation measuring devices described in Table 8 captures rainfall or snowfall in a storage container that is open to the atmosphere. The depth of the collected water is then observed, manually or automatically, and from those observations, the depth of precipitation at the location of the gage is obtained.

Table 8. Precipitation field monitoring options (WMO, 1994)

Option	Categorization
Manual (also referred to as *non-recording*, *totalizer,* or *accumulator* gage)	This gage is read by a human observer. An example is shown in Figure 4. Often such gages are read daily, so detailed information about the short-term temporal distribution of the rainfall is not available.
Automatic hydrometeorological observation station	This type of gage observes and records precipitation automatically. An example is a weighing gage with a strip-chart data logger. With this gage, the temporal distribution is known, as a continuous time record is available. In the *HEC-HMS User's Manual*, a gage at which the temporal distribution is known is referred to as a *recording* gage.
Telemetering hydrometeorological observation station	This type of gage observes and transmits precipitation depth automatically, but does not store it locally. An example is an ALERT system tipping bucket raingage with UHF radio transmitter. Telemetering gages are typically recording gages. Figure 5 is an example of such a gage.
Telemetering automatic hydrometeorological observation station	This type of gage observes, records, and transmits automatically. It is a recording gage.

Runoff-Computation Requirements

Chapter 6 provides details of the models for computing direct runoff from precipitation: the alternatives are various forms of the unit-hydrograph model and the kinematic-wave model. Inherent in models of both types is an assumption that the precipitation is distributed uniformly over the watershed for a given time duration. This, in turn, requires specifying the properties of this uniform rainfall. In the program, these properties include (1) the total depth of the watershed precipitation, and (2) the temporal distribution of that precipitation.

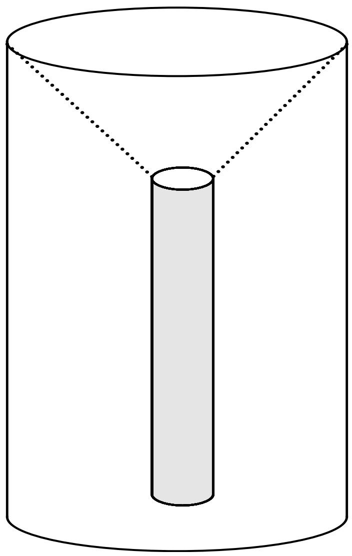

Figure 4.   Manual precipitation gage.

Figure 5.   Telemetering precipitation observation gage.

Mean-Areal Precipitation Depth Computation

The required watershed precipitation depth can be inferred from the depths at gages using an averaging scheme. Thus:

$$P_{MAP} = \frac{\sum_i \left(w_i \sum_t p_i(t) \right)}{\sum_i w_i} \qquad (2)$$

where P_{MAP} = total storm mean areal precipitation (MAP) depth over the watershed; $p_i(t)$ = precipitation depth measured at time t at gage i; and w_i = weighting factor assigned to gage/observation i. If gage i is not a recording device, only the quantity $\Sigma p_i(t)$, the total storm precipitation at gage i, will be available and used in the computation.

Common methods for determining the gage weighting factors for MAP depth computation include:

- **Arithmetic mean**. This method assigns a weight to each gage equal to the reciprocal of the total number of gages used for the MAP computation. Gages in or adjacent to the watershed can be selected.

- **Thiessen polygon**. This is an area-based weighting scheme, based upon an assumption that the precipitation depth at any point within a watershed is the same as the precipitation depth at the nearest gage in or near the watershed. Thus, it assigns a weight to each gage in proportion to the area of the watershed that is closest to that gage.

  As illustrated in Figure 6(a), the gage nearest each point in the watershed may be found graphically by connecting the gages, and constructing perpendicular bisecting lines; these form the boundaries of polygons surrounding each gage. The area within each polygon is nearest the enclosed gage, so the weight assigned to the gage is the fraction of the total area that the polygon represents.

  Details and examples of the procedure are presented in Chow, Maidment, and Mays (1988), Linsley, Koehler, and Paulus (1982), and most hydrology texts.

- **Isohyetal**. This too is an area-based weighting scheme. Contour lines of equal precipitation are estimated from the point measurements, as illustrated by Figure 6(b). This allows a user to exercise judgment and knowledge of a basin while constructing the contour map. MAP is estimated by finding the average precipitation depth between each pair of contours (rather than precipitation at individual gages), and weighting these depths by the fraction of total area enclosed by the pair of contours.

  Again, details and examples of the procedure are presented in most hydrology texts.

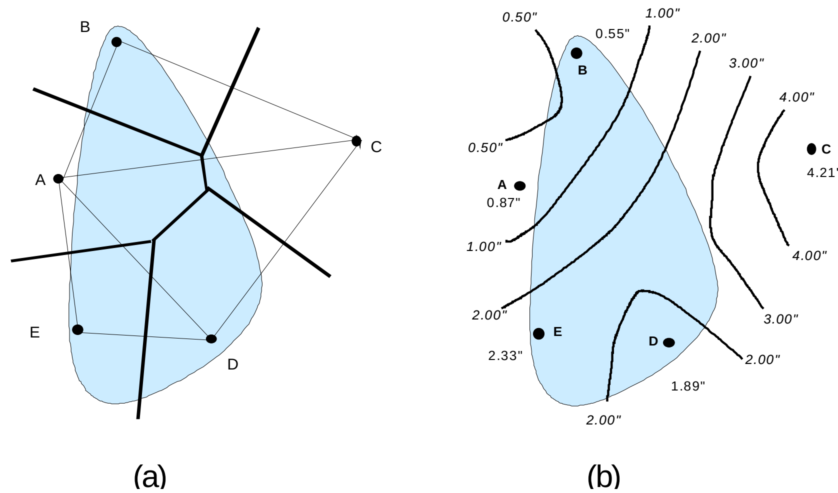

Figure 6. Illustration of MAP depth computation schemes.

Temporal Distribution of Precipitation

To compute a hydrograph, which represents flow variations with time, information about the MAP variations with time must be provided. To do so, a precipitation pattern with ordinates, $p_{pattern}(t)$ is defined and the temporal distribution of the MAP depth is computed as (from Equation 2):

$$p_{MAP}(t) = \left[\frac{p_{pattern}(t)}{\sum_t p_{pattern}(t)} \right] P_{MAP} \qquad (3)$$

in which $p_{MAP}(t)$ = the watershed MAP at time t. As with total storm depth, the pattern can be inferred from gage observations with a weighting scheme:

$$p_{pattern}(t) = \frac{\sum w_i(t) p_i(t)}{\sum w_i(t)} \qquad (4)$$

in which $p_i(t)$ = precipitation measured at gage i at time t, and $w_i(t)$ = weighting factor assigned to gage i at time t. In this computation, only recording gages are used.

If a single recording gage is used in Equation 3, the resulting MAP hyetograph will have the same relative distribution as the observed hyetograph. For example, if the gage recorded 10% of the total precipitation in 30 minutes, the MAP hyetograph will have 10% of the MAP in the same 30-minute period.

On the other hand, if two or more gages are used, the pattern will be an average of that observed at those gages. Consequently, if the temporal distribution at those gages is significantly different, as it might be with a moving storm, the average pattern may obscure information about the precipitation on the watershed. This is illustrated by the temporal distributions shown in Figure 7. Here, hyetographs of rainfall at two gages are shown. At gage A, rain fell at a uniform rate of 10 mm/hr from 0000 hours until 0200 hours. No rain was measured at gage A after 0200. At gage B, no rain was observed until 0200, and then rainfall at a uniform rate of 10 mm/hr was observed until 0400. The likely pattern is that the storm moved across the watershed from gage A to gage B. If these gage data are used with Equations 4-2 and 4-3 to compute an average pattern, weighting each gage equally, the result is a uniform rate of 5 mm/hr from 0000 until 0400. This may fail to represent well the average temporal pattern. A better scheme might be to use one of the gages as a pattern for the watershed average.

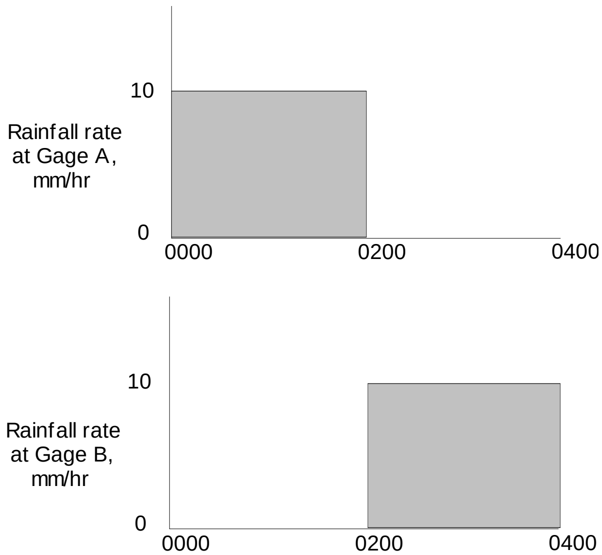

Figure 7. Illustration of the hazard of averaging rainfall temporal distributions.

Inverse-Distance-Squared Method

As an alternative to separately defining the total MAP depth and combining this with a pattern temporal distribution to derive the MAP hyetograph, one can select a scheme that computes the MAP hyetograph directly. This so-called inverse-distance-squared weighting method computes $P(t)$, the watershed precipitation at time t, by dynamically applying a weighting scheme to precipitation measured at watershed precipitation gages at time t.

The scheme relies on the notion of *nodes* that are positioned within a watershed such that they provide adequate spatial resolution of precipitation in the watershed. The program computes the precipitation hyetograph for each node using gages near that node. To select these gages, hypothetical north-south and east-west axes are constructed through each node and the nearest gage is found in each quadrant defined by the axes. This is illustrated in Figure 8. Weights are computed and assigned to the gages in inverse proportion to the square of the distance from the node to the gage. For example, in Figure 8, the weight for the gage C in the northeastern quadrant of the grid is computed as:

$$w_C = \frac{\frac{1}{d_C^2}}{\frac{1}{d_C^2} + \frac{1}{d_D^2} + \frac{1}{d_E^2} + \frac{1}{d_A^2}} \qquad (5)$$

in which w_C = weight assigned to gage C; d_C = distance from node to gage C; d_D = distance from node to gage D in southeastern quadrant; d_E = distance from node to gage E in southwestern quadrant; and d_F = distance from node to gage F in northwestern quadrant of grid. Weights for gages D, E and A are computed similarly.

With the weights thus computed, the node hyetograph ordinate at time t is computed as:

$$p_{node}(t) = w_A\, p_A(t) + w_C\, p_C(t) + w_D\, p_D(t) + w_E\, p_E(t) \qquad (6)$$

This computation is repeated for all times *t*.

Note that gage *B* in Figure 8 is not used in this example, as it is not nearest to the node in the northwestern quadrant. However, for any time that the precipitation ordinate is missing for gage *A*, the data from gage *B* will be used. In general terms, the nearest gage in the quadrant with data (including a zero value) will be used to compute the MAP.

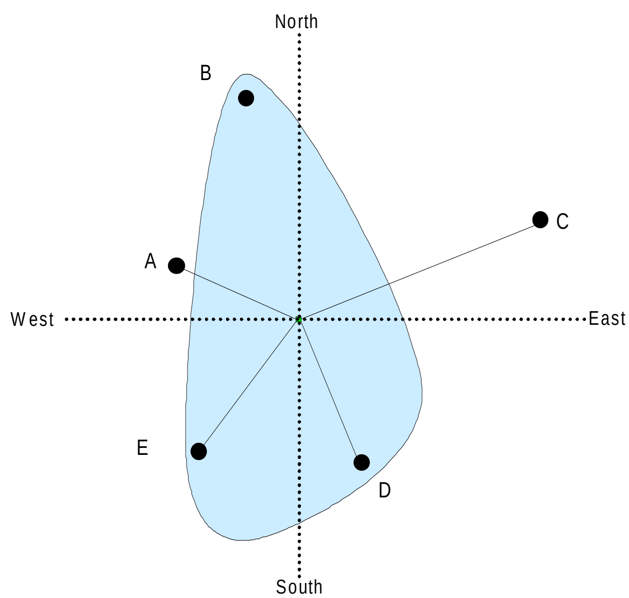

Figure 8. Illustration of inverse-distance-squared scheme.

Radar "Observations" of Historical Precipitation

Figure 9 shows a typical (but very simple) situation. Runoff is to be predicted for the watershed shown. Rainfall depths are measured at reporting gages *A* and *B* near the watershed.

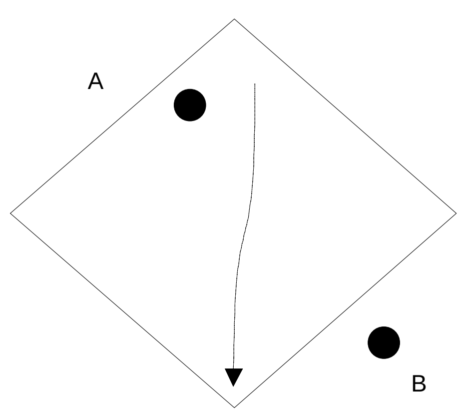

Figure 9. MAP can be computed as a weighted-average of depths at gages A and B.

From the gaged data, one might estimate MAP as a weighted average of the depths observed. The weights assigned might depend, for example, on how far the gage is from one or more user-specified index points in the watershed. In this example, if an index point at the centroid of the watershed is selected, then the weights will be approximately equal, so the MAP will equal the arithmetic average of the depths observed at gages *A* and *B*.

The MAP estimated from the gage network in this manner is a good representation of rainfall on a subwatershed if the raingage network is adequately dense in the vicinity of the storm. The gages near the storm must also be in operation, and must not be subject to inadvertent inconsistencies (Curtis and Burnash, 1996).

The National Weather Service provides guidelines on the density of a raingage network. These suggest that the minimum number of raingages, *N*, for a local flood warning network is:

$$N = A^{0.33} \qquad (7)$$

in which *A* = area in square miles. However, even with this network of more than the minimum number of gages, not all storms may be adequately gaged. Precipitation gages such as those illustrated in Figure 4 and Figure 1 are typically 8-12 in (20-30 cm) in diameter. Thus, in a one sq-mi (2.6 km^2) watershed, the catch surface of the gage thus represents a sample of precipitation on approximately 1/100,000,000th of the total watershed area. With this small sample size, isolated storms may not be measured well if the storm cells are located over areas in which "holes" exist in the gage network or if the precipitation is not truly uniform over the watershed.

The impact of these "holes" is illustrated by Figure 10. Figure 10(a) shows the watershed from Figure 9, but with a storm superimposed. In this case, observations at gages A and B would not represent well the rainfall because of the areal distribution of the rainfall field. The "true" MAP likely would exceed the MAP computed as an average of the observations. In that case, the runoff would be under-predicted. Similarly, the gage observations do not represent well the true rainfall in the case shown in Figure 10(b). There, the storm cell is over gage A, but because of the location of the gage, it is not a good sampler of rainfall for this watershed. Thus, in the second case the runoff might be over-predicted.

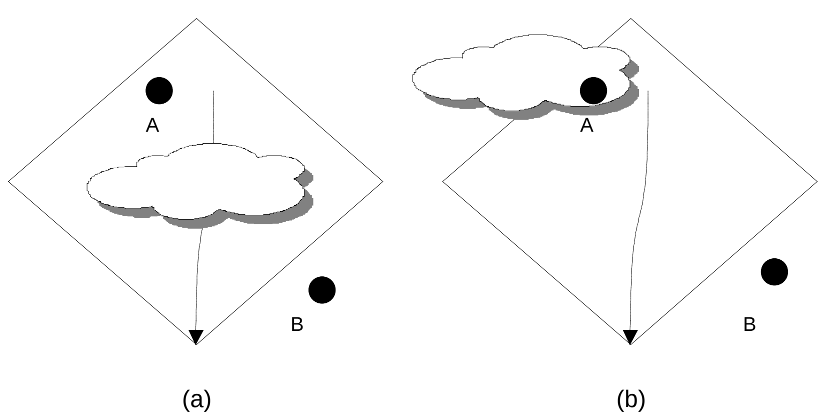

Figure 10. Lack of coverage can complicate MAP estimation.

One potential solution to the problem of holes in the rainfall observations is to increase the number of gages in the network. But even as the number of gages is increased, one cannot be assured of measuring adequately the rainfall for all storm events. Unless the distance between gages is less than the principal dimension of a typical storm cell, the rainfall on a watershed may well be misestimated.

A second solution is use of rainfall depth estimates from weather radar.

Radar Data

The WMO *Guide to hydrological practices* (1994) explains that

> *Radar permits the observation of the location and movement of areas of precipitation, and certain types of radar equipment can yield estimates of rainfall rates over areas within range of the radar.*

Weather radar data are available from National Weather Service (NWS) Weather Surveillance Radar Doppler units (WSR-88D) throughout the US. Each of these units provides coverage of a 230-km-radius circular area. The WSR-88D radar transmits an S-band signal that is reflected when it encounters a raindrop or another obstacle in the atmosphere. The power of the reflected signal, which is commonly expressed in terms of reflectivity, is measured at the transmitter during 360° azimuthal scans, centered at the radar unit. Over a 5- to 10-minute period, successive scans are made with 0.5° increments in elevation. The reflectivity observations from these scans are integrated over time and space to yield estimates of particle size and density in an atmospheric column over a particular location. To simplify data management, display and analysis, the NWS digitizes and reports reflectivity for cells in a Hydrologic Rainfall Analysis Project (HRAP) grid. Cells of the grid are approximately 4 km by 4 km.

Given the reflectivity, the rainfall rate for each of the HRAP cells can be inferred because the power of the reflected signal is related to the size of and density of the reflecting obstacles. The simplest model to estimate rainfall from reflectivity is a Z-R relationship, and the most commonly-used of these is:

$$Z = a\,R^{b} \qquad (8)$$

in which Z = reflectivity factor; R = the rainfall intensity; and a and b = empirical coefficients. Thus, as a product of the weather radar, rainfall for cells of a grid that is centered about a radar unit can be estimated. This estimate is the MAP for that cell and does not necessarily suggest the rain depth at any particular point in the cell.

The NWS, Department of Defense, and Department of Transportation (Federal Aviation Administration) cooperatively operate the WSR-88D network. They collect and disseminate the weather radar data to federal government users. The NEXRAD Information Dissemination Service (NIDS) was established to provide access to the weather radar data for users outside of the federal government. Each WSR-88D unit that is designated to support the NIDS program has four ports to which selected vendors may connect. The NIDS vendors, in turn, disseminate the data to their clients using their own facilities, charging the clients for the products provided and for any value added. For example, one NIDS vendor in 1998 was distributing a 1-km x 1-km mosaic of data. This mosaic is a combined image of reflectivity data from several radar units with overlapping or contiguous scans. Combining images in this manner increases the chance of identifying and eliminating anomalies. It also provides a better view of storms over large basins.

Figure 11 illustrates the advantages of acquiring weather radar data. Figure 11(a) shows the watershed from Figure 10, but with an HRAP-like grid system superimposed. Data from a radar unit will provide an estimate of rainfall in each cell of the grid. Commonly these radar-rainfall estimates are presented in graphical format, as illustrated in Figure 11(b), with color codes for various intensity ranges. (This is similar to the images seen on television weather reports.)

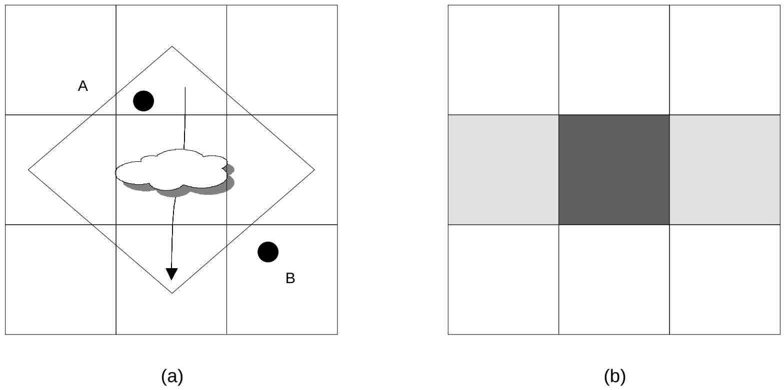

Figure 11. Weather radar provides rainfall "observations" on a grid.

With estimates of rainfall in grid cells, a "big picture" of the rainfall field over a watershed is presented. With this, better estimates of the MAP at any time are possible due to knowledge of the extent of the storm cells, the areas of more intense rainfall, and the areas of no rainfall. By using successive sweeps of the radar, a time series of average rainfall depths for cells that represent each watershed can be developed.

Computations with Radar-Measured Precipitation

From the time-series of average rainfall depths, the required MAP series can be computed, now accounting explicitly for the spatial variability of rainfall. The MAP computations are relatively simple: MAP for each time step is the average of the rainfall in the set of cells that represents the watershed.

The program includes algorithms for MAP computation from radar data that are stored in either HRAP format or in HEC's standard hydrologic grid (SHG). (The latter is described in the March 1996 issue of HEC's *Advances in Hydrologic Engineering*.) Software for reformatting radar data provided by the NWS into the format required by the program is available from HEC.

The radar-estimated precipitation should be compared or corrected to correlate with field observations. Radar measures only the movement of water in the atmosphere, not the volume of water falling on the watershed. Only options shown in Table 8 can measure this. Ideally, the average rainfall would combine radar and raingage networks; in the US, the NWS Stage 3 reports do so.

Hypothetical Storms

Standards-Based Design Concepts

Standards-based criteria are commonly used for planning and designing new water-control facilities, preparing for and responding to floods, and regulating floodplain activities (WEF/ASCE, 1992). With the standards-based criteria, a threshold or standard is set for an acceptable level of risk to the public, and actions are taken to satisfy this standard. For example, levees in parts of the western US have been designed to provide protection from flooding should a selected large historical event re-occur.

Standards-based criteria commonly limit risk by constraining the long-term average time between exceedances of the capacity of drainage facilities. For example, the criteria might limit development in a floodplain so that the annual probability is no more than 0.01 that water rises above the first floor of structures. This limit is known as the annual exceedance probability (AEP). To meet the standard, the specified AEP discharge and stage must be estimated. In many cases, additional information about the volume and time of runoff may be required. For example, runoff volume must be estimated to provide information for sizing a detention pond for flood protection.

When sufficient streamflow data are available for the stream of interest, design discharges for specified AEP can be estimated using statistical-analysis methods. In the US, guidelines for conducting such statistical analyses were proposed by the Interagency Advisory Committee on Water Data and published in *Bulletin 17B* (1982). The Bulletin 17B procedure uses recorded annual maximum discharge to calibrate a log-Pearson type III statistical model, and uses this calibrated statistical model to predict the flows with selected AEP. Designs based upon non-exceedance of this flow will meet the standards.

The statistical-analysis procedure of Bulletin 17B is of limited use for estimating discharge in many cases, because:

- Few streams are gaged, and those that are, usually do not have a record long enough for the statistical model to be fitted accurately.
- Land-use changes alter the response of a watershed to rainfall, so hypothetical-flood discharges determined with data for undeveloped or natural conditions do not reflect discharges expected with developed conditions.
- The statistical-analysis procedure does not provide information about runoff volume and timing.

Consequently, in many cases an alternative analysis procedure is required. A common alternative analysis procedure relies upon use of rainfall of specified AEP (also known as a design or hypothetical storm), coupled with a mathematical model of the processes by which rainfall is transformed to runoff. The notion is that if median or average values of all model parameters are used, the AEP of the discharge computed from the hypothetical storm should equal the AEP of the precipitation (Pilgrim and Cordery, 1975).

Three alternative standards-based storms are included:

1. A balanced frequency-based storm.
2. The standard project storm (SPS).
3. A user-defined storm depth and temporal distribution.

Frequency-Based Hypothetical Storm

The objective of the frequency-based hypothetical storm that is included in the program is to define an event for which the precipitation depths for various durations within the storm have a consistent exceedance probability. Use the following steps to develop the storm:

1. Specify the total point-precipitation depths for the selected exceedance probability for durations from 5 minutes through the desired total duration of the hypothetical storm (but no longer than 10 days). Depths for durations less than the time interval selected for runoff modeling are not necessary. For example, if the analysis requires a 24-hour storm, and the runoff from a 0.01-AEP event is sought, the user must specify the 0.01-AEP depths for durations from 5 minutes to 24 hours.

   In the US, depths for various durations for a specified exceedance probability may be obtained by consulting locally-developed depth-duration-frequency functions, NOAA Atlas 2 for the western US (Miller, et al., 1973) or NWS TP-40 (Herschfield, 1961) and TP-49 (Miller, 1964) for the eastern US. If the depths are found from isopluvial maps in one of these sources, the values should be plotted and smoothed by prior to input to ensure that the storm hyetograph is reasonably shaped.

2. The program applies an area correction factor to the specified depths. Precipitation estimates from depth-duration-frequency studies, such as those presented in NOAA Atlas 2 or TP 40, commonly are point estimates. However, intense rainfall is unlikely to be distributed uniformly over a large watershed. For a specified frequency and duration, the average rainfall depth over an area is less than the depth at a point. To account for this, the U.S. Weather Bureau (1958) derived, from averages of annual series of point and areal values for several dense, recording-raingage networks, factors by which point depths are to be reduced to yield areal-average depths. The factors, expressed as a percentage of point depth, are a function of area and duration, as shown in Figure 12.

   In accordance with the recommendation of the World Meteorological Organization (1994), point values should be used without reduction for areas less than 9.6 sq. mi. Furthermore, in accordance with the recommendation of HEC (USACE, 1982), no adjustment should be made for durations less than 30 minutes. A short duration is appropriate for a watershed with a short time of concentration. A short time of concentration, in turn, is indicative of a relatively small watershed, which, in turn, requires no adjustment.

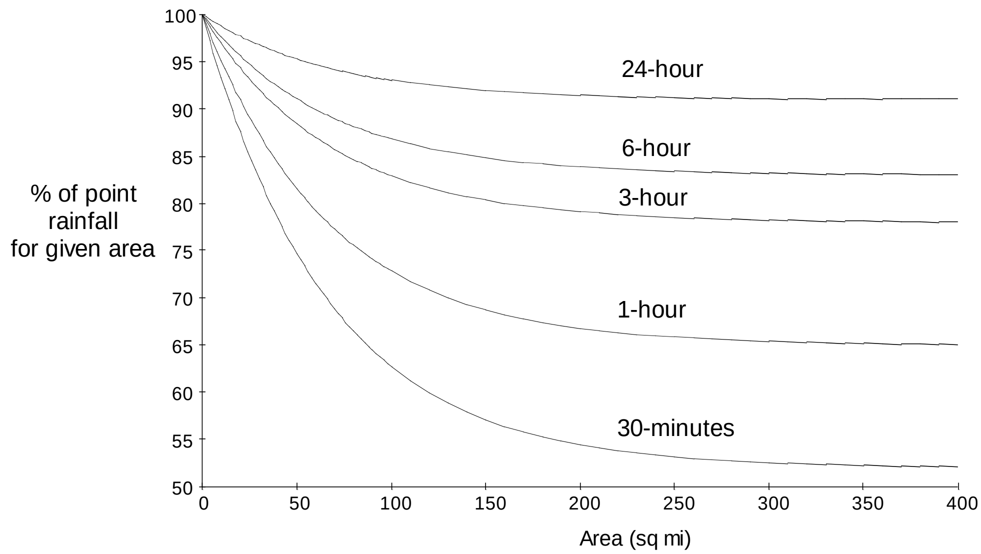

Figure 12. Point depth reduction factors.

3. The program interpolates to find depths for durations that are integer multiples of the time interval selected for runoff modeling. Linear interpolation is used, with logarithmically transformed values of depth and duration specified in Step 1.

4. Find successive differences in the cumulative depths from Step 3, thus computing a set of incremental precipitation depths, each of duration equal to the selected computation interval.

5. Use the alternating block method (Chow, Maidment, Mays, 1988) to develop a hyetograph from the incremental precipitation values (blocks). This method positions the block of maximum incremental depth at the middle of the required duration. The remaining blocks are arranged then in descending order, alternately before and after the central block. ZZ is an example of this temporal distribution; this shows the rainfall depths for a 24-hour hypothetical storm, with a 1-hour computation interval.

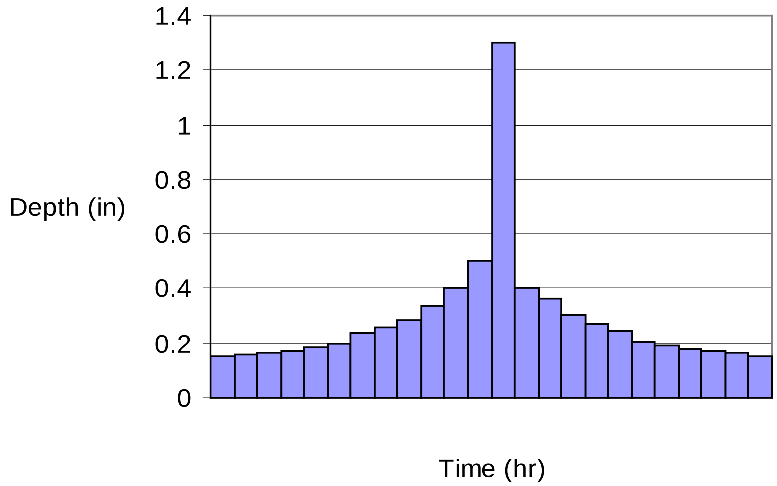

Figure 13. Example of distribution of frequency-based hypothetical storm.

The Standard Project Storm

The standard project storm (SPS) is

> *…a relationship of precipitation versus time that is intended to be reasonably characteristic of large storms that have or could occur in the locality of concern. It is developed by studying the major storm events in the region, excluding the most extreme. For areas east of 105 longitude the results of SPS studies are published in EM 1110-2-1411 as generalized regional relationships for depth, duration and area of precipitation. For areas west of 105 longitude, special studies are made to develop the appropriate SPS estimates. The standard project flood (SPF) [runoff from the SPS] is used as one convenient way to compare levels of protection between projects, calibrate watershed models, and provide a deterministic check of statistical flood frequency estimates.* (USACE, 1989)

The SPS model included in the program is the SPS applicable to basins east of 105° longitude (east of the Rocky Mountains); it is limited to areas 10 to 1,000 square miles. The SPS is rarely used now because of the emergence of risk-based design techniques, the inconsistency of the method between different geographic regions, the lack of a standard SPS west of 105° longitude, and no attached probability of occurrence. The 0.002 annual exceedance probability event has all but replaced the SPS for design and description purposes. However, to use the SPS model, an index precipitation, the area over which the storm occurs, and a temporal distribution are required. The index precipitation for an area can be estimated using the map in EM 1110-2-1411. EM 1110-2-1411 proposes using a shape factor (transposition coefficient) to adapt the ideal SPS if the watershed is not of "ideal" shape, if the storm is not centered over the watershed, or if the storm area is larger than the watershed area. The shape factor can be determined using procedures specified in EM 1110-2-1411. The temporal distribution can be the standard EM 1110-2-1411 distribution (USACE, 1952) or the Southwestern Division PMP distribution. The latter is the distribution of 100-yr precipitation at St. Louis, MO, as proposed by the NWS (Fredrick, et al., 1977). A more detailed description of the SPS can be found in the HEC *Training Document No. 15* (USACE, 1982).

Once the SPS precipitation depth is specified, the program calculates a total storm depth distributed over a 96-hour duration using:

$$Total\ depth = \sum_{i=1}^{4}(R_{24HR}(i) \cdot SPFE) \qquad (9)$$

where *SPFE* = standard-project-flood index-precipitation depth in inches; and $R_{24HR}(i)$ = percent of the index precipitation occurring during the i^{th} 24-hour period. $R_{24HR}(i)$ is given by:

$$R_{24HR(i)} = \begin{Bmatrix} 3.5 & if\ \ i=1 \\ 15.5 & if\ \ i=2 \\ 182.15 - 14.3537 * LOG_e(TRSDA + 80) & if\ \ i=3 \\ 6.0 & if\ \ i=4 \end{Bmatrix} \qquad (10)$$

where *TRSDA* = storm area, in square miles.

Each 24-hour period is divided into four 6-hour periods. The ratio of the 24-hour precipitation occurring during each 6-hour period is calculated as:

$$R_{6HR(i)} = \begin{Bmatrix} R_{6HR}(4) - 0.033 & if\ \ i=1 \\ 0.055 * (SPFE - 6.0)^{0.51} & if\ \ i=2 \\ \dfrac{13.42}{(SPFE + 11.0)^{0.93}} & if\ \ i=3 \\ 0.5 * (1.0 - R_{6HR}(3) - R_{6HR}(2)) + 0.0165 & if\ \ i=4 \end{Bmatrix} \qquad (11)$$

where $R_{6HR}(i)$ = ratio of 24-hour precipitation occurring during the i^{th} 6-hour period.

The program computes the precipitation for each time interval in the j^{th} 6-hour interval of the i^{th} 24-hour period (except the peak 6-hour period) with:

$$PRCP = 0.01 * R_{24HR}(i) * R_{6HR}(j) * SPFE * \frac{\Delta t}{6} \qquad (12)$$

where Δt = computation time interval, in hours.

The peak 6-hour precipitation of each day is distributed according to the percentages in Table 9. When using a computation time interval less than one hour, the peak 1-hour precipitation is distributed according to the percentages in Table 10. (The selected time interval must divide evenly into one hour.) When the time interval is larger than shown in Table 9 or Table 10, the percentage for the peak time interval is the sum of the highest percentages. For example, for a 2-hour time interval, the values are (14 + 12)%, (38 + 15)%, and (11 + 10)%. The interval with the largest percentage is preceded by the second largest and followed by the third largest. The second largest percentage is preceded by the fourth largest, the third largest percentage is followed by the fifth largest, and so on.

Table 9. Distribution of maximum 6-hour SPS in percent of 6-hour amount.

Duration (hr)	EM 1110-2-1411 Criteria (Standard)	Southwestern Division Criteria (SWD)
1	10	4
2	12	8
3	15	19
4	38	50
5	14	11
6	11	8

Table 10. Distribution of maximum 1-hour precipitation in the SPS.

Time (min)	Percent of Maximum 1-hr Precipitation in Each Time Interval	Accumulated Precent of Precipitation
5	3	3
10	4	7
15	5	12
20	6	18
25	9	27
30	17	44
35	25	69
40	11	80
45	8	88
50	5	93
55	4	97
60	3	100

User-Defined Hypothetical-Storm Distribution

The *User-Specified Hyetograph* option allows the user to define the depth and temporal distribution of a hypothetical storm. The hypothetical rainfall values entered are interpreted as if it were rainfall at a gage.

For example, for drainage planning in the US, Soil Conservation Service (SCS), now known as the Natural Resources Conservation Service (NRCS), hypothetical storms are commonly used. These storms were developed by the SCS as averages of rainfall patterns; they are represented in a dimensionless form in TR-55 (USDA, 1986). The choice of one of the storm types shown depends upon the location of the watershed. For example, near Davis, CA, the appropriate storm is an SCS Type I storm.

Storm Selection

The following important questions will help guide the selection of a proper hypothetical storm:

- What AEP event should be used when planning to use a risk-based event? If the goal is to define a regulatory floodplain, such as the so-called 100-yr floodplain, select a single hypothetical storm with the specified AEP, compute the runoff from that storm, and assign to the flow, volume, or stage the same AEP as that assigned to the storm.

  On the other hand, if the goal is to define a discharge-frequency function, the solution is to define hypothetical storms with AEP ranging from small, frequent events (say 0.50 AEP) to large, infrequent events (such as the 0.002-AEP event.) With these, compute the runoff and assign to the runoff peaks, volumes or states the same AEP as the hypothetical storm. Chapter 3 of EM 1110-2-1415 (USACE, 1993) and Chapter 17 of EM 1110-2-1417 (USACE, 1994) provide more information about this procedure.

- What duration should the event be? The included hypothetical storm options permit defining events that last from a few minutes to several days. The selected storm must be sufficiently long so that the entire watershed is contributing to runoff at the concentration point. Thus, the duration must exceed the time of concentration of the watershed; some argue that it should be 3 or 4 times the time of concentration (Placer County, 1990).

  The National Weather Service (Fredrick et al., 1977) reports that

  > *…in the contiguous US, the most frequent duration of runoff-producing rainfall is about 12 hr…at the end of any 6-hr period within a storm, the probability of occurrence of additional runoff-producing rain is slightly greater than 0.5…at the end of the first 6 hr, the probability that the storm is not over is approximately 0.75. It does not drop below 0.5 until the duration has exceeded 24 hr.*

  Using observed data, Levy and McCuen (1999) showed that 24 hr is a good hypothetical-storm length for watersheds in Maryland from 2 to 50 square miles. This leads to the conclusion that a 24-hr hypothetical storm is a reasonable choice if the storm duration exceeds the time of concentration of the watershed. Indeed, much drainage system planning in the US relies on use of a 24-hr event, and the SCS events are limited to storms of 24-hr duration. However, considering the likelihood of longer or shorter storms, this length should be used with care.

- Should a frequency-based hypothetical storm temporal distribution, the SPS distribution, or another distribution be used? The answer to this depends upon the information needs of the study. The SPS may be chosen to provide hydrological estimates for design of a major flood-control structure. On the other hand, a different distribution, such as the triangular temporal distribution, may be selected if flows for establishing frequency functions for determining optimal detention storage are necessary.

Risk-Based Design Concepts

The program includes features for specifying and computing runoff from a variety of standards-based storms, including frequency-based hypothetical storms. However, this does not form the basis for the Corps' flood-damage reduction projects. Instead, as outlined in EM 1110-2-1419 and EM 1110-2-1619, these projects are designed to provide protection from a range of events, with project features selected to maximize contribution to national economic development (NED), consistent with environmental and policy constraints. In this context, the frequency-based hypothetical storm capability may be used to estimate without-project and with-project flow or stage frequency functions, with which expected annual damage reduction may be computed.

Evaporation and Transpiration

Chapter 3 describes how, in common application, detailed accounting of evaporation and transpiration are omitted, as these are insignificant during a flood. In the case of shorter storms, such as the SPS, it may be appropriate to omit this accounting. However, with the soil-moisture accounting (SMA) model, which is described in detail in Chapter 5, it is possible to analyze watershed response to longer precipitation records—records that include both periods of rainfall and periods without rainfall. During periods without rainfall, the watershed moisture state continues to change, as water moves and is stored. Evaporation and transpiration are critical components of this movement.

Evaporation, as modeled in the program, includes vaporization of water directly from the soil and vegetative surface, and transpiration through plant leaves. This volume of evaporation and transpiration combined is estimated as an average volume. The evaporation and transpiration are combined and collectively referred to as evapotranspiration (ET) in the SMA model and in the meteorological input to the program. In this input, monthly-varying ET values are specified, along with an ET coefficient. The potential ET rate for all time periods within the month is computed as the product of the monthly value and the coefficient.

Chapter 5 describes in detail how specified ET rates are used in the soil-moisture accounting model.

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CHAPTER 5

Computing Runoff Volumes

As illustrated by Figure 2, HEC-HMS computes runoff volume by computing the volume of water that is intercepted, infiltrated, stored, evaporated, or transpired and subtracting it from the precipitation. Interception and surface storage are intended to represent the surface storage of water by trees or grass, local depressions in the ground surface, cracks and crevices in parking lots or roofs, or a surface area where water is not free to move as overland flow. Infiltration represents the movement of water to areas beneath the land surface. Interception, infiltration, storage, evaporation, and transpiration collectively are referred to in the program and documentation as *losses*. This chapter describes the loss models and how to use them to compute runoff volumes.

Basic Concepts

The program considers that all land and water in a watershed can be categorized as either:

- Directly-connected impervious surface.
- Pervious surface.

Directly-connected impervious surface in a watershed is that portion of the watershed for which all contributing precipitation runs off, with no infiltration, evaporation, or other volume losses. Precipitation on the pervious surfaces is subject to losses. The following alternative models are included to account for the cumulative losses:

- The initial and constant-rate loss model.
- The deficit and constant-rate model.
- The SCS curve number (CN) loss model.
- The Green and Ampt loss model.

With each model, precipitation loss is found for each computation time interval, and is subtracted from the MAP depth for that interval. The remaining depth is referred to as precipitation excess. This depth is considered uniformly distributed over a watershed area, so it represents a volume of runoff.

Chapter 6 describes the two options for direct runoff hydrograph computations: the unit hydrograph (UH) model and the kinematic-wave model. With a UH model, the excess on pervious portions of the watershed is added to the precipitation on directly-connected impervious area, and the sum is used in runoff computations. With the kinematic-wave model, directly connected impervious areas may be modeled separately from pervious areas if two overland flow planes are defined.

Initial and Constant Loss Model

Basic Concepts and Equations

The underlying concept of the initial and constant-rate loss model is that the maximum potential rate of precipitation loss, f_c, is constant throughout an event. Thus, if p_t is the MAP depth during a time interval t to $t+\Delta t$, the excess, pe_t, during the interval is given by:

$$pe_t = \begin{Bmatrix} p_t - f_c & if \ \ p_t > f_c \\ 0 & otherwise \end{Bmatrix} \tag{13}$$

An initial loss, I_a, is added to the model to represent interception and depression storage. Interception storage is a consequence of absorption of precipitation by surface cover, including plants in the watershed. Depression storage is a consequence of depressions in the watershed topography; water is stored in these and eventually infiltrates or evaporates. This loss occurs prior to the onset of runoff.

Until the accumulated precipitation on the pervious area exceeds the initial loss volume, no runoff occurs. Thus, the excess is given by:

$$pe_t = \begin{Bmatrix} 0 & if \sum p_i < I_a \\ p_t - f_c & if \sum p_i > I_a \ and \ p_t > f_c \\ 0 & if \sum p_i > I_a \ and \ p_t < f_c \end{Bmatrix} \tag{14}$$

Estimating Initial Loss and Constant Rate

The initial and constant-rate model, in fact, includes one parameter (the constant rate) and one initial condition (the initial loss). Respectively, these represent physical properties of the watershed soils and land use and the antecedent condition.

If the watershed is in a saturated condition, I_a will approach zero. If the watershed is dry, then I_a will increase to represent the maximum precipitation depth that can fall on the watershed with no runoff; this will depend on the watershed terrain, land use, soil types, and soil treatment. Table 6-1 of EM 1110-2-1417 suggests that this ranges from 10-20% of the total rainfall for forested areas to 0.1-0.2 inches for urban areas.

The constant loss rate can be viewed as the ultimate infiltration capacity of the soils. The SCS (1986) classified soils on the basis of this infiltration capacity, and Skaggs and Khaleel (1982) have published estimates of infiltration rates for those soils, as shown in Table 11. These may be used in the absence of better information.

Because the model parameter is not a measured parameter, it and the initial condition are best determined by calibration. Chapter 9 of this manual describes the program's calibration capability.

Table 11. SCS soil groups and infiltration (loss) rates (SCS, 1986; Skaggs and Khaleel, 1982)

Soil Group	Description	Range of Loss Rates (in/hr)
A	Deep sand, deep loess, aggregated silts	0.30-0.45
B	Shallow loess, sandy loam	0.15-0.30
C	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay	0.05-0.15
D	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils	0.00-0.05

Deficit and Constant Loss Model

The program also includes a quasi-continuous variation on the initial and constant model of precipitation losses; this is known as the deficit and constant loss model. This model is different from the initial and constant loss model in that the initial loss can "recover" after a prolonged period of no rainfall. [This model is similar to the loss model included in computer program HEC-IFH (HEC, 1992).]

To use this model, the initial loss and constant rate plus the recovery rate must be specified. The moisture deficit is tracked continuously, computed as the initial abstraction volume less precipitation volume plus recovery volume during precipitation-free periods. The recovery rate could be estimated as the sum of the evaporation rate and percolation rate, or some fraction thereof.

SCS Curve Number Loss Model

Basic Concepts and Equations

The Soil Conservation Service (SCS) Curve Number (CN) model estimates precipitation excess as a function of cumulative precipitation, soil cover, land use, and antecedent moisture, using the following equation:

$$P_e = \frac{(P - I_a)^2}{P - I_a + S} \qquad (15)$$

where P_e = accumulated precipitation excess at time t; P = accumulated rainfall depth at time t; I_a = the initial abstraction (initial loss); and S = potential maximum retention, a measure of the ability of a watershed to abstract and retain storm precipitation. Until the accumulated rainfall exceeds the initial abstraction, the precipitation excess, and hence the runoff, will be zero.

From analysis of results from many small experimental watersheds, the SCS developed an empirical relationship of I_a and S:

$$I_a = 0.2\,S \qquad (16)$$

Therefore, the cumulative excess at time t is:

$$P_e = \frac{(P - 0.2\,S)^2}{P + 0.8\,S} \tag{17}$$

Incremental excess for a time interval is computed as the difference between the accumulated excess at the end of and beginning of the period.

The maximum retention, S, and watershed characteristics are related through an intermediate parameter, the curve number (commonly abbreviated *CN*) as:

$$S = \left\{ \begin{array}{lc} \dfrac{1000 - 10\,CN}{CN} & \text{(foot} - \text{pound system)} \\ \dfrac{25400 - 254\,CN}{CN} & \text{(SI)} \end{array} \right\} \tag{18}$$

CN values range from 100 (for water bodies) to approximately 30 for permeable soils with high infiltration rates.

Publications from the Soil Conservation Service (1971, 1986) provide further background and details on use of the CN model.

Estimating CN

The CN for a watershed can be estimated as a function of land use, soil type, and antecedent watershed moisture, using tables published by the SCS. For convenience, Appendix A of this document includes CN tables developed by the SCS and published in Technical Report 55 (commonly referred to as TR-55). With these tables and knowledge of the soil type and land use, the single-valued CN can be found. For example, for a watershed that consists of a tomato field on sandy loam near Davis, CA, the CN shown in Table 2-2b of the TR-55 tables is 78. (This is the entry for straight row crop, good hydrologic condition, B hydrologic soil group.) This CN is entered directly in the appropriate input form.

For a watershed that consists of several soil types and land uses, a composite CN is calculated as:

$$CN_{composite} = \frac{\sum A_i CN_i}{\sum A_i} \tag{19}$$

in which $CN_{composite}$ = the composite CN used for runoff volume computations; *i* = an index of watersheds subdivisions of uniform land use and soil type; CN_i = the CN for subdivision *i*; and A_i = the drainage area of subdivision *i.*

Users of the SCS model as implemented in the program should note that the tables in Appendix A include composite CN for urban districts, residential districts, and newly graded areas. That is, the CN shown are composite values for directly-connected impervious area and open space. If CN for these land uses are selected, no further accounting of directly-connected impervious area is required.

Gridded SCS Curve Number Loss Model

Alternatively, the grid-based CN modeling option can be used. With this option, the subdivisions in Equation 19 are grid cells. The description of each cell in the database includes: the location of the cell, the travel distance from the watershed outlet, the cell size, and the cell CN. The program computes precipitation excess for each cell independently, using Equation 17, and routes the excess to the watershed outlet, using the ModClark method.

Green and Ampt Loss Model

Basic Concepts and Equations

The Green and Ampt infiltration model included in the program is a conceptual model of infiltration of precipitation in a watershed. According to EM 1110-2-1417

> *…the transport of infiltrated rainfall through the soil profile and the infiltration capacity of the soil is governed by Richards' equation…[which is] derived by combining an unsaturated flow form of Darcy's law with the requirements of mass conservation.*

EM 1110-2-1417 describes in detail how the Green and Ampt model combines and solves these equations. In summary, the model computes the precipitation loss on the pervious area in a time interval as:

$$f_t = K\left[\frac{1+(\phi-\theta_i)S_f}{F_t}\right] \qquad (20)$$

in which f_t = loss during period t; K = saturated hydraulic conductivity; $(\phi - \theta_i)$ = volume moisture deficit; S_f = wetting front suction; and F_t = cumulative loss at time t. The precipitation excess on the pervious area is the difference in the MAP during the period and the loss computed with Equation 20.

As implemented, the Green and Ampt model also includes an initial abstraction. This initial condition represents surface ponding not otherwise included in the model.

Estimating Model Parameters

The Green and Ampt model in HEC-HMS requires specification of the parameters:

- **Initial loss**. This is a function of the watershed moisture at the beginning of the precipitation. It may be estimated in the same manner as the initial abstraction for other loss models.

- **Hydraulic conductivity**. Table 12 (which is derived from Table 6-2 of EM 1110-2-1417) provides estimates of this parameter as a function of texture class, which may be found from a soil survey. For additional details regarding the derivation of information in this table, see Rawls, et al. (1982).

- **Wetting front suction**. This can be estimated as a function of pore size distribution, which can, in turn, be correlated with texture class. Table 12 provides estimates of this.

- **Volume moisture deficit**. This is $(\phi - \theta_i)$ in Equation 20, the soil porosity less the initial water content. Rawls and Brakensiek (1982) and Rawls, et al. (1982) have correlated the porosity with soil texture class; Table 12 shows this relationship. The initial water content must be between zero and ϕ. For example, if the soil is saturated, $\theta_i = \phi$; for a completely dry soil, $\theta_i = 0$. EM 1110-2-1417 suggests that the initial water content may be related to an antecedent precipitation index.

Table 12. Texture class estimates (Rawls, et al., 1982)

Texture Class	Porosity, ϕ (cm^3/cm)	Hydraulic conductivity, θ, saturated (cm/hr)	Wetting front suction (cm)
Sand	0.437	21.00	10.6
Loamy sand	0.437	6.11	14.2
Sandy loam	0.453	2.59	22.2
Loam	0.463	1.32	31.5
Silt loam	0.501	0.68	40.4
Sandy clay loam	0.398	0.43	44.9
Clay loam	0.464	0.23	44.6
Silty clay loam	0.471	0.15	58.1
Sandy clay	0.430	0.12	63.6
Silty clay	0.479	0.09	64.7
Clay	0.475	0.06	71.4

Soil Moisture Accounting Loss Model

Models described thus far in this chapter are *event* models. They simulate behavior of a hydrologic system during a precipitation event, and to do so, they require specification of all conditions at the start of the event. The alternative is a *continuous* model—a model that simulates both wet and dry weather behavior. The soil moisture accounting model (SMA) does this.

Basic Concepts and Equations

The SMA model is patterned after Leavesley's Precipitation-Runoff Modeling System (1983) and is described in detail in Bennett (1998). The model simulates the movement of water through and storage of water on vegetation, on the soil surface, in the soil profile, and in groundwater layers. Given precipitation and potential evapotranspiration (ET), the model computes basin surface runoff, groundwater flow, losses due to ET, and deep percolation over the entire basin.

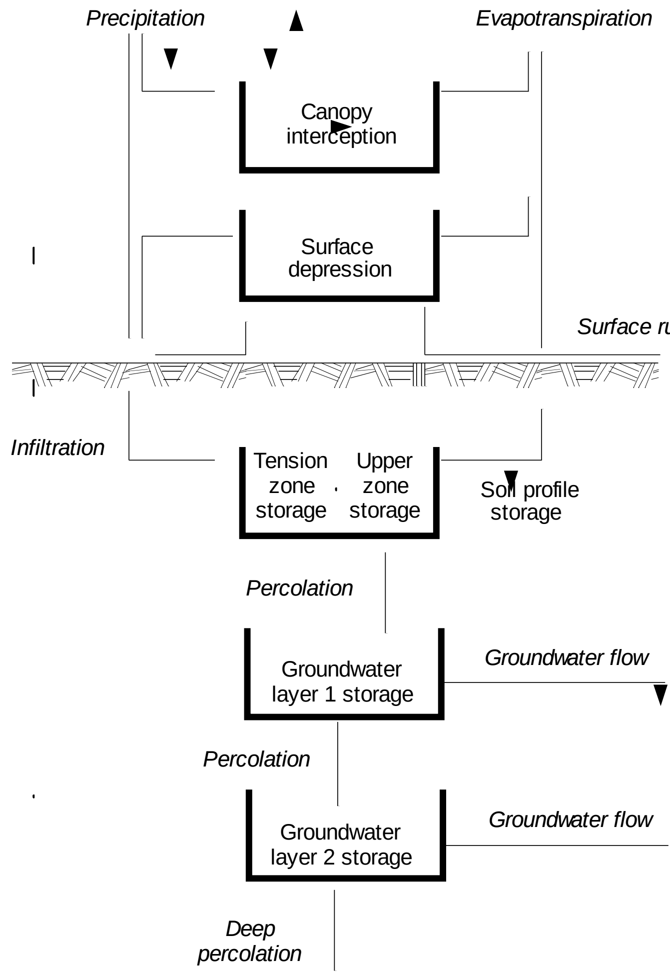

Figure 14. Conceptual schematic of the continuous soil moisture accounting algorithm (Bennett, 1998)

Storage Component

The SMA model represents the watershed with a series of storage layers, as illustrated by Figure 14. Rates of inflow to, outflow from, and capacities of the layers control the volume of water lost or added to each of these storage components. Current storage contents are calculated during the simulation and vary continuously both during and between storms. The different storage layers in the SMA model are:

- **Canopy-interception storage.** Canopy interception represents precipitation that is captured on trees, shrubs, and grasses, and does not reach the soil surface. Precipitation is the only inflow into this layer. When precipitation occurs, it first fills canopy storage. Only after this storage is filled does precipitation become available for filling other storage volumes. Water in canopy interception storage is held until it is removed by evaporation.

- **Surface-interception storage.** Surface depression storage is the volume of water held in shallow surface depressions. Inflows to this storage come from precipitation not captured by canopy interception and in excess of the infiltration rate. Outflows from this storage can be due to infiltration and to ET. Any contents in surface depression storage at the beginning of the time step are available for infiltration. If the water available for infiltration exceeds the infiltration rate, surface interception storage is filled. Once the volume of surface interception is exceeded, this excess water contributes to surface runoff.

- **Soil-profile storage.** The soil profile storage represents water stored in the top layer of the soil. Inflow is infiltration from the surface. Outflows include percolation to a groundwater layer and ET. The soil profile zone is divided into two regions, the *upper zone* and the *tension zone*. The upper zone is defined as the portion of the soil profile that will lose water to ET and/or percolation. The tension zone is defined as the area that will lose water to ET only. The upper zone represents water held in the pores of the soil. The tension zone represents water attached to soil particles. ET occurs from the upper zone first and tension zone last. Furthermore, ET is reduced below the potential rate occurring from the tension zone, as shown in Figure 15. This represents the natural increasing resistance in removing water attached to soil particles. ET can also be limited to the volume available in the upper zone during specified winter months, depicting the end of transpiration by annual plants.

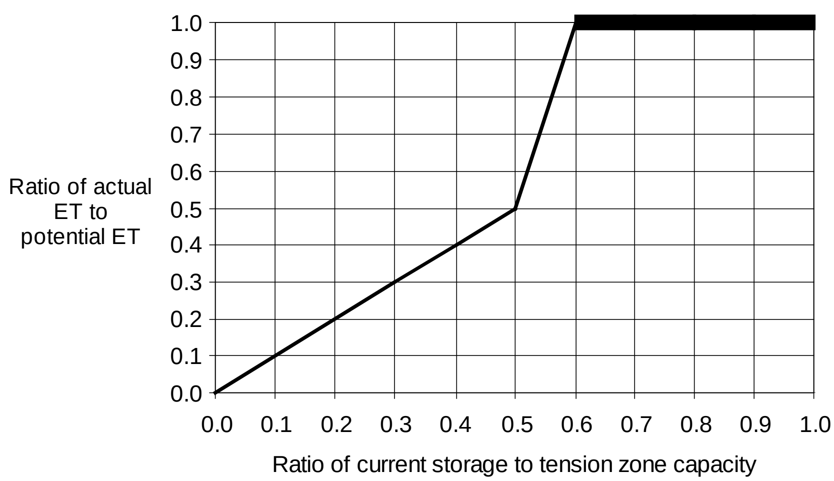

Figure 15. ET as a function of tension zone storage (Bennett, 1998)

- **Groundwater storage**. Groundwater layers in the SMA represent horizontal interflow processes. The SMA model can include either one or two such layers. Water percolates into groundwater storage from the soil profile. The percolation rate is a function of a user-specified maximum percolation rate and the current storage in the layers between which the water flows. Losses from a groundwater storage layer are due to *groundwater flow* or to percolation from one layer to another. Percolation from the soil profile enters the first layer. Stored water can then percolate from layer 1 to groundwater layer 2 or from groundwater layer 2 to deep percolation. In the latter case, this water is considered lost from the system; aquifer flow is not modeled in the SMA.

Flow Component

The SMA model computes flow into, out of, and between the storage volumes. This flow can take the form of:

- **Precipitation.** Precipitation is an input to the system of storages. Precipitation first contributes to the canopy interception storage. If the canopy storage fills, the excess amount is then available for infiltration.

- **Infiltration.** Infiltration is water that enters the soil profile from the ground surface. Water available for infiltration during a time step comes from precipitation that passes through canopy interception, plus water already in surface storage.

  The volume of infiltration during a time interval is a function of the volume of water available for infiltration, the state (fraction of capacity) of the soil profile, and the maximum infiltration rate specified by the model user. For each interval in the analysis, the SMA model computes the potential infiltration volume, *PotSoilInfl*, as:

$$PotSoilInfil = MaxSoilInfil - \frac{CurSoilStore}{MaxSoilStore} MaxSoilInfil \qquad (21)$$

  where *MaxSoilInfl* = the maximum infiltration rate; *CurSoilStore* = the volume in the soil storage at the beginning of the time step; and *MaxSoilStore* = the maximum volume of the soil storage. The actual infiltration rate, *ActInfil*, is the minimum of *PotSoilInfil* and the volume of water available for infiltration. If the water available for infiltration exceeds this calculated infiltration rate, the excess then contributes to surface interception storage.

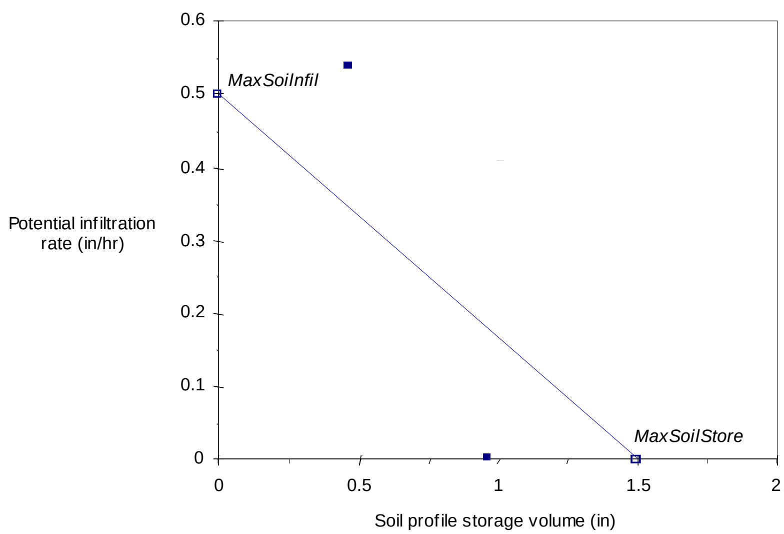

Figure 16. Potential infiltration rate versus beginning of time step soil profile storage.

Figure 16 illustrates the relationship of these, using an example with *MaxSoilInfil* = 0.5 in/hr and *MaxSoilStore* = 1.5 in. As illustrated, when the soil profile storage is empty, potential infiltration equals the maximum infiltration rate, and when the soil profile is full, potential infiltration is zero.

- **Percolation.** Percolation is the movement of water downward from the soil profile, through the groundwater layers, and into a deep aquifer.

  In the SMA model, the rate of percolation between the soil-profile storage and a groundwater layer or between two groundwater layers depends on the volume in the source and receiving layers. The rate is greatest when the source layer is nearly full and the receiving layer is nearly empty. Conversely, when the receiving layer is nearly full and the source layer is nearly empty, the percolation rate is less. In the SMA model, the percolation rate from the soil profile into groundwater layer 1 is computed as:

  $$PotSoilPerc = MaxSoilPerc \left(\frac{CurSoilStore}{MaxSoilStore}\right)\left(1-\frac{CurGwStore}{MaxGwStore}\right) \quad (22)$$

  where *PotSoilPerc* = the potential soil percolation rate; *MaxSoilPerc* = a user-specified maximum percolation rate; *CurSoilStore* = the calculated soil storage at the beginning of the time step; *MaxSoilStore* = a user-specified maximum storage for the soil profile; *CurGwStore* = the calculated groundwater storage for the upper groundwater layer at the beginning of the time step; and *MaxGwStore* = a user-specified maximum groundwater storage for groundwater layer 1.

  The potential percolation rate computed with Equation 22 is multiplied by the time step to compute a potential percolation volume. The available water for percolation is equal the initial soil storage plus infiltration. The minimum of the potential volume and the available volume percolates to groundwater layer 1.

  A similar equation is used to compute *PotGwPerc*, the potential percolation from groundwater layer 1 to layer 2:

  $$PotGwPerc = MaxPercGw\left(\frac{CurGwStore}{MaxGwStore}\right)\left(1-\frac{CurGwStore}{MaxGwStore}\right) \quad (23)$$

  where *MaxPercGw* = a user-specified maximum percolation rate; *CurGwStore* = the calculated groundwater storage for the groundwater layer 2; and *MaxGwStore* = a user-specified maximum groundwater storage for layer 2. The actual volume of percolation is computed as described above.

  For percolation directly from the soil profile to the deep aquifer in the absence of groundwater layers, for percolation from layer 1 when layer 2 is not used, or percolation from layer 2, the rate depends only on the storage volume in the source layer. In those cases, percolation rates are computed as

  $$PotSoilPerc = MaxSoilPerc \ \frac{CurSoilStore}{MaxSoilStore} \quad (24)$$

  and

  $$PotGwPerc = MaxPercGw \ \frac{CurGwStore}{MaxGwStore} \quad (25)$$

respectively, and actual percolation volumes are computed as described above.

- **Surface runoff and groundwater flow.** Surface runoff is the water that exceeds the infiltration rate and overflows the surface storage. This volume of water is direct runoff; the resulting runoff hydrograph is computed with one of the models described in Chapter 6.

  Groundwater flow is the sum of the volumes of groundwater flow from each groundwater layer at the end of the time interval. The rate of flow is computed as:

$$GwFlow_{t+1} = \frac{ActSoilPerc + CurGw_iStore - PotGw_iPerc - \frac{1}{2}GwFlow_t \cdot TimeStep}{RoutGw_iStore + \frac{1}{2}TimeStep} \quad (26)$$

  where $GwFlow_t$ and $GwFlow_{t+1}$ = groundwater flow rate at beginning of the time interval t and t+1, respectively; *ActSoilPerc* = actual percolation from the soil profile to the groundwater layer; $PotGw_iPerc$ = potential percolation from groundwater layer i; $RoutGw_iStore$ = groundwater flow routing coefficient from groundwater storage i; *TimeStep* = the simulation time step; and other terms are as defined previously. The volume of groundwater flow that the watershed releases, *GwVolume*, is the integral of the rate over the model time interval. This is computed as

$$GwVolume = \frac{1}{2}(GwFlow_{t+1} + GwFlow_t) \cdot TimeStep \quad (27)$$

  This volume may be treated as inflow to a linear reservoir model to simulate baseflow, as described in Chapter 7.

- **Evapotranspiration (ET).** ET is the loss of water from the canopy interception, surface depression, and soil profile storages. In the SMA model, potential ET demand currently is computed from monthly pan evaporation depths, multiplied by monthly-varying pan correction coefficients, and scaled to the time interval.

  The potential ET volume is satisfied first from canopy interception, then from surface interception, and finally from the soil profile. Within the soil profile, potential ET is first fulfilled from the upper zone, then the tension zone. If potential ET is not completely satisfied from one storage in a time interval, the unsatisfied potential ET volume is filled from the next available storage.

  When ET is from interception storage, surface storage, or the upper zone of the soil profile, actual ET is equivalent to potential ET. When potential ET is drawn from the tension zone, the actual ET is a percentage of the potential, computed as:

$$ActEvapSoil = PotEvapSoil \cdot f\left(CurSoilStore, MaxTenStore\right) \quad (28)$$

  where *ActEvapSoil* = the calculated ET from soil storage; *PotEvapSoil* = the calculated maximum potential ET; and *MaxTenStore* = the user specified maximum storage in the tension zone of soil storage. The function, *f*(•), in Equation 28 is defined as follows:

- o As long as the current storage in the soil profile exceeds the maximum tension zone storage (*CurSoilStore*/*MaxTenStore* > 1), water is removed from the upper zone at a one-to-one rate, the same as losses from canopy and surface interception.
- o Once the volume of water in the soil profile zone reaches the tension zone, *f*(•) is determined similar to percolation. This represents the decreasing rate of ET loss from the soil profile as the amount of water in storage (and therefore the capillary force) decreases, as illustrated in Figure 15.

Order of Model Computations

Flow into and out of storage layers is computed for each time step in the SMA model. (Appendix B describes how the time step is selected.) The order of computations in each time step depends upon occurrence of precipitation or ET, as follows:

- If precipitation occurs during the interval, ET is not modeled. Precipitation contributes first to canopy-interception storage. Precipitation in excess of canopy-interception storage, combined with water already in surface storage, is available for infiltration. If the volume available is greater than the available soil storage, or if the calculated potential infiltration rate is not sufficient to deplete this volume in the determined time step, the excess goes to surface-depression storage. When surface-depression storage is full, any excess is surface runoff.

  Infiltrated water enters soil storage, with the tension zone filling first. Water in the soil profile, but not in the tension zone, percolates to the first groundwater layer. Groundwater flow is routed from the groundwater layer 1, and then any remaining water may percolate to the groundwater layer 2. Percolation from layer 2 is to a deep aquifer and is lost to the model.

- If no precipitation occurs, ET is modeled. Potential ET is satisfied first from canopy storage, then from surface storage. Finally, if the potential ET is still not satisfied from surface sources, water is removed from the upper-soil profile storage. The model then continues as described above for the precipitation periods.

Estimating Model Parameters

SMA model parameters must be determined by calibration with observed data. In this iterative process, candidate parameter values are proposed, the model is exercised with these parameters and precipitation and evapotranspiration inputs. The resulting computed hydrograph is compared with an observed hydrograph for the same period. If the match is not satisfactory, the parameters are adjusted, and the search continues. Bennett (1998) and EM 1110-2-1417 offer guidance for this calibration. The automatic calibration algorithm described in Chapter 9 may be used to aid this search.

Applicability and Limitations of the Runoff-Volume Models

Selecting a loss model and estimating the model parameters are critical steps in developing program input. Not all loss models can be used with all transforms. For instance, the gridded loss methods can only be used with the ModClark transform. Table 13 lists some positive and negative aspects of the alternatives. However, these are only guidelines and should be supplemented by knowledge of, and experience with, the models and the watershed. League and Freeze (1985) point out that

> *In many ways, hydrologic modeling is more an art than a science, and it is likely to remain so. Predictive hydrologic modeling is normally carried out on a given catchment using a specific model under the supervision of an individual hydrologist. The usefulness of the results depends in large measure on the talents and experience of the hydrologist and …understanding of the mathematical nuances of the particular model and the hydrologic nuances of the particular catchment. It is unlikely that the results of an objective analysis of modeling methods…can ever be substituted for the subjective talents of an experienced modeler.*

Table 13. Positive and negative aspects of loss models.

Model	Positive	Negative
Initial and Constant	"Mature" model that has been used successfully in hundreds of studies throughout the US. Easy to set up and use. Model is parsimonious; it includes only a few parameters necessary to explain the variation of runoff volume (see EM 1110-2-1417).	Difficult to apply to ungaged areas due to lack of direct physical relationship of parameters and watershed properties. Model may be too simple to predict losses within event, even if it does predict total losses well.
Deficit and Constant	Similar to above Can be used for long-term simulations (for example, for period-of-record analyses.)	Similar to above
SCS CN	Simple, predictable, and stable method Relies on only one parameter, which varies as a function of soil group, land use and treatment, surface condition, and antecedent moisture condition. Features readily grasped and reasonable well-documented environmental inputs. Well established method, widely accepted for use in US and abroad. (From Ponce and Hawkins, 1996)	Predicted values not in accordance with classical unsaturated flow theory. Infiltration rate will approach zero during a storm of long duration, rather than constant rate as expected. Developed with data from small agricultural watersheds in midwestern US, so applicability elsewhere is uncertain. Default initial abstraction (0.2*S*) does not depend upon storm characteristics or timing. Thus, if used with design storm, abstraction will be same with 0.50-AEP storm and 0.01-AEP storm. Rainfall intensity not considered. (Same loss for 25 mm rainfall in 1 hour or 1 day.)
Green and Ampt	Parameters can be estimated for ungaged watersheds from information about soils	Not widely used, so less mature, not as much experience in professional community. Less parsimonious than simple empirical models.

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CHAPTER 6

Modeling Direct Runoff

This chapter describes the models that simulate the process of direct runoff of excess precipitation on a watershed. This process refers to the "transformation" of precipitation excess into point runoff. The program provides two options for these transform methods:

- **Empirical models (also referred to as system theoretic models)**. These are the traditional unit hydrograph (UH) models. The system theoretic models attempt to establish a causal linkage between runoff and excess precipitation without detailed consideration of the internal processes. The equations and the parameters of the model have limited physical significance. Instead, they are selected through optimization of some goodness-of-fit criterion.

- **A conceptual model**. The conceptual model included in the program is a kinematic-wave model of overland flow. It represents, to the extent possible, all physical mechanisms that govern the movement of the excess precipitation over the watershed land surface and in small collector channels in the watershed.

Basic Concepts of the Unit Hydrograph Model

The unit hydrograph is a well-known, commonly-used empirical model of the relationship of direct runoff to excess precipitation. As originally proposed by Sherman in 1932, it is "…the basin outflow resulting from one unit of direct runoff generated uniformly over the drainage area at a uniform rainfall rate during a specified period of rainfall duration." The underlying concept of the UH is that the runoff process is linear, so the runoff from greater or less than one unit is simply a multiple of the unit runoff hydrograph.

To compute the direct runoff hydrograph with a UH, the program uses a discrete representation of excess precipitation, in which a "pulse" of excess precipitation is known for each time interval. It then solves the discrete convolution equation for a linear system:

$$Q_n = \sum_{m=1}^{n \le M} P_m U_{n-m+1} \qquad (29)$$

where Q_n = storm hydrograph ordinate at time $n\Delta t$; P_m = rainfall excess depth in time interval $m\Delta t$ to $(m+1)\Delta t$; M = total number of discrete rainfall pulses; and U_{n-m+1} = UH ordinate at time $(n-m+1)\Delta t$. Q_n and P_m are expressed as flow rate and depth respectively, and U_{n-m+1} has dimensions of flow rate per unit depth. Use of this equation requires the implicit assumptions:

1. The excess precipitation is distributed uniformly spatially and is of constant intensity throughout a time interval Δt.

2. The ordinates of a direct-runoff hydrograph corresponding to excess precipitation of a given duration are directly proportional to the volume of excess. Thus, twice the excess produces a doubling of runoff hydrograph ordinates and half the excess produces a halving. This is the so-called *assumption of linearity*.

3. The direct runoff hydrograph resulting from a given increment of excess is independent of the time of occurrence of the excess and of the antecedent precipitation. This is the *assumption of time-invariance*.

4. Precipitation excesses of equal duration are assumed to produce hydrographs with equivalent time bases regardless of the intensity of the precipitation.

User-Specified Unit Hydrograph

A UH may be specified directly by entering all ordinates of the UH. That is, values of U_{n-m+1} in Equation 29 may be specified directly and used for runoff computation.

Estimating the Model Parameters

Because it is a system theoretic model, the UH for a watershed is properly derived from observed rainfall and runoff, using deconvolution—the inverse of solution of the convolution equation. To estimate a UH using this procedure:

1. Collect data for an appropriate observed storm runoff hydrograph and the causal precipitation. This storm selected should result in approximately one unit of excess, should be uniformly distributed over the watershed, should be uniform in intensity throughout its entire duration, and should be of duration sufficient to ensure that the entire watershed is responding. This duration, T, is the duration of the UH that will be found.

2. Estimate losses and subtract these from the precipitation. Estimate baseflow and separate this from the runoff.

3. Calculate the total volume of direct runoff and convert this to equivalent uniform depth over the watershed area.

4. Divide the direct runoff ordinates by the equivalent uniform depth. The result is the UH.

Chow, Maidment, and Mays (1988) present matrix algebra, linear regression, and linear programming alternatives to this approach.

With any of these approaches, the UH derived is appropriate only for analysis of other storms of duration T. To apply the UH to storms of different duration, the UH for these other durations must be derived. If the other durations are integral multiples of T, the new UH can be computed by lagging the original UH, summing the results, and dividing the ordinates to yield a hydrograph with volume equal one unit. Otherwise, the S-hydrograph method can be used. This is described in detail in texts by Chow, Maidment, and Mays (1988), Linsley, Kohler, and Paulhus (1982), Bedient and Huber (1992), and others.

Application of the User-Specified UH

In practice, direct runoff computation with a specified-UH is uncommon. The data necessary to derive the UH in the manner described herein are seldom available, so the UH ordinates are not easily found. Worse yet, streamflow data are not available for many watersheds of interest, so the procedure cannot be used at all. Even when the data are available, they are available for complex storms, with significant variations of precipitation depths within the storm. Thus, the UH-determination procedures described are difficult to apply. Finally, to provide information for many water resources development activities, a UH for alternative watershed land use or channel conditions is often needed—data necessary to derive a UH for these future conditions are never available.

Parametric and Synthetic Unit Hydrographs

What is a Parametric UH?

The alternative to specifying the entire set of UH ordinates is to use a parametric UH. A parametric UH defines all pertinent UH properties with one or more equations, each of which has one or more parameters. When the parameters are specified, the equations can be solved, yielding the UH ordinates.

For example, to approximate the UH with a triangle shape, all the ordinates can be described by specifying:

- Magnitude of the UH peak.
- Time of the UH peak.

The volume of the UH is known—it is one unit depth multiplied by the watershed drainage area. This knowledge allows us, in turn, to determine the time base of the UH. With the peak, time of peak, and time base, all the ordinates on the rising limb and falling limb of the UH can be computed through simple linear interpolation. Other parametric UH are more complex, but the concept is the same.

What is a Synthetic UH?

A synthetic UH relates the parameters of a parametric UH model to watershed characteristics. By using the relationships, it is possible to develop a UH for watersheds or conditions other than the watershed and conditions originally used as the source of data to derive the UH. For example, a synthetic UH model may relate the UH peak of the simple triangular UH to the drainage area of the watershed. With the relationship, an estimate of the UH peak for any watershed can be made given an estimate of the drainage area. If the time of UH peak and total time base of the UH is estimated in a similar manner, the UH can be defined "synthetically" for any watershed. That is, the UH can be defined in the absence of the precipitation and runoff data necessary to derive the UH.

Chow, Maidment, and Mays (1988) suggest that synthetic UH fall into three categories:

1. Those that relate UH characteristics (such as UH peak and peak time) to watershed characteristics. The Snyder UH is such a synthetic UH.
2. Those that are based upon a dimensionless UH. The SCS UH is such a synthetic UH.

3. Those that are based upon a quasi-conceptual accounting for watershed storage. The Clark UH and the ModClark model do so.

All of these synthetic UH models are included in the program.

Snyder Unit Hydrograph Model

Basic Concepts and Equations

In 1938, Snyder published a description of a parametric UH that he had developed for analysis of ungaged watersheds in the Appalachian Highlands in the US. More importantly, he provided relationships for estimating the UH parameters from watershed characteristics. The program includes an implementation of the Snyder UH.

For his work, Snyder selected the lag, peak flow, and total time base as the critical characteristics of a UH. He defined a *standard* UH as one whose rainfall duration, t_r, is related to the basin lag, t_p, by:

$$t_p = 5.5 t_r \qquad (30)$$

(Here lag is the difference in the time of the UH peak and the time associated with the centroid of the excess rainfall hyetograph, as illustrated in Figure 17.) Thus, if the duration is specified, the lag (and hence the time of UH peak) of Snyder's standard UH can be found. If the duration of the desired UH for the watershed of interest is significantly different from that specified by Equation 30, the following relationship can be used to define the relationship of UH peak time and UH duration:

$$t_{pR} = t_p - \frac{t_r - t_R}{4} \qquad (31)$$

in which t_R = duration of desired UH; and t_{pR} = lag of desired UH.

For the *standard* case, Snyder discovered that UH lag and peak per unit of excess precipitation per unit area of the watershed were related by:

$$\frac{U_p}{A} = C \frac{C_p}{t_p} \qquad (32)$$

where U_p = peak of standard UH; A = watershed drainage area; C_p = UH peaking coefficient; and C = conversion constant (2.75 for SI or 640 for foot-pound system).

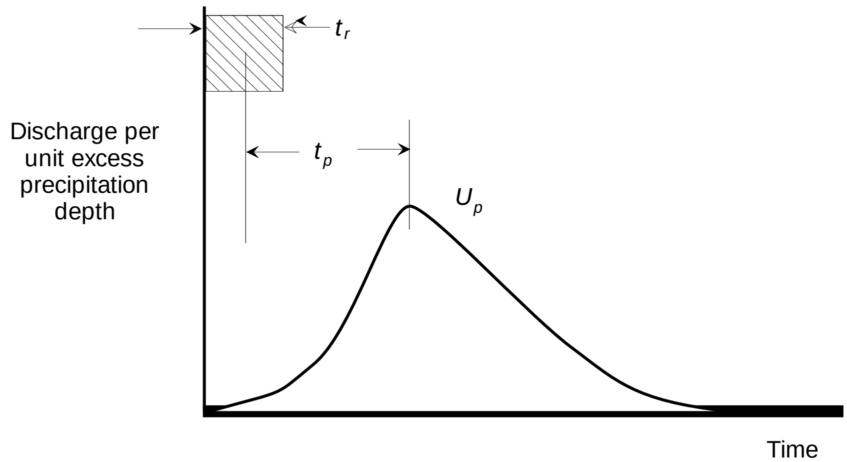


Figure 17. Snyder's unit hydrograph.

For other durations, the UH peak, Q_{pR}, is defined as:

$$\frac{U_{pR}}{A} = C\frac{C_p}{t_{pR}} \tag{33}$$

Snyder's UH model requires specifying the standard lag, t_p, and the coefficient, C_p. The program sets t_{pR} of Equation 31 equal the specified time interval, and solves Equation 31 to find the lag of the required UH. Finally, Equation 33 is solved to find the UH peak. Snyder proposed a relationship with which the total time base of the UH may be defined. I nstead of this relationship, the program uses the computed UH peak and time of peak to find an equivalent UH with Clark's model (see the next section). From that, it determines the time base and all ordinates other than the UH peak.

Estimating the Model Parameters

Snyder collected rainfall and runoff data from gaged watersheds, derived the UH as described earlier, parameterized these UH, and related the parameters to measurable watershed characteristics. For the UH lag, he proposed:

$$t_p = CC_t(LL_c)^{0.3} \tag{34}$$

where C_t = basin coefficient; L = length of the main stream from the outlet to the divide; L_c = length along the main stream from the outlet to a point nearest the watershed centroid; and C = a conversion constant (0.75 for SI and 1.00 for foot-pound system).

The parameter C_t of Equation 34 and C_p of Equation 32 are best found via calibration, as they are not physically-based parameters. Bedient and Huber (1992) report that C_t typically ranges from 1.8 to 2.2, although it has been found to vary from 0.4 in mountainous areas to 8.0 along the Gulf of Mexico. They report also that C_p ranges from 0.4 to 0.8, where larger values of C_p are associated with smaller values of C_t.

Alternative forms of the parameter predictive equations have been proposed. For example, the Los Angeles District, USACE (1944) has proposed to estimate t_p as:

$$t_p = CC_t\left(\frac{LL_c}{\sqrt{S}}\right)^N \quad (35)$$

where *S* = overall slope of longest watercourse from point of concentration to the boundary of drainage basin; and *N* = an exponent, commonly taken as 0.33.

Others have proposed estimating t_p as a function of t_C , the watershed time of concentration (Cudworth, 1989; USACE, 1987). Time of concentration is the time of flow from the most hydraulically remote point in the watershed to the watershed outlet, and may be estimated with simple models of the hydraulic processes, as described here in the section on the SCS UH model. Various studies estimate t_p as 50-75% of t_C.

SCS Unit Hydrograph Model

The Soil Conservation Service (SCS) proposed a parametric UH model; this model is included in the program. The model is based upon averages of UH derived from gaged rainfall and runoff for a large number of small agricultural watersheds throughout the US. SCS *Technical Report 55* (1986) and the *National Engineering Handbook* (1971) describe the UH in detail.

Basic Concepts and Equations

At the heart of the SCS UH model is a dimensionless, single-peaked UH. This dimensionless UH, which is shown in ZZ, expresses the UH discharge, U_t, as a ratio to the UH peak discharge, U_p, for any time *t*, a fraction of T_p, the time to UH peak.

Research by the SCS suggests that the UH peak and time of UH peak are related by:

$$U_P = C\frac{A}{T_P} \quad (36)$$

in which *A* = watershed area; and *C* = conversion constant (2.08 in SI and 484 in foot-pound system). The time of peak (also known as the time of rise) is related to the duration of the unit of excess precipitation as:

$$T_p = \frac{\Delta t}{2} + t_{lag} \quad (37)$$

in which Δt = the excess precipitation duration (which is also the computational interval in the run); and t_{lag} = the basin lag, defined as the time difference between the center of mass of rainfall excess and the peak of the UH. [Note that for adequate definition of the ordinates on the rising limb of the SCS UH, a computational interval, Δt , that is less than 29% of t_{lag} must be used (USACE, 1998).]

When the lag time is specified, the program solves Equation 37 to find the time of UH peak, and Equation 36 to find the UH peak. With U_p and T_p known, the UH can be found from the dimensionless form, which is built into the program, by multiplication.

Estimating the Model Parameters

The SCS UH lag can be estimated via calibration, using procedures described in Chapter 9, for gaged headwater subwatersheds.

For ungaged watersheds, the SCS suggests that the UH lag time may be related to time of concentration, t_c, as:

$$t_{lag} = 0.6\, t_c \tag{38}$$

Time of concentration is a quasi-physically based parameter that can be estimated as:

$$t_c = t_{sheet} + t_{shallow} + t_{channel} \tag{39}$$

where t_{sheet} = sum of travel time in sheet flow segments over the watershed land surface; $t_{shallow}$ = sum of travel time in shallow flow segments, down streets, in gutters, or in shallow rills and rivulets; and $t_{channel}$ = sum of travel time in channel segments.

Identify open channels where cross section information is available. Obtain cross sections from field surveys, maps, or aerial photographs. For these channels, estimate velocity by Manning's equation:

$$V = \frac{CR^{2/3}S^{1/2}}{n} \tag{40}$$

where V = average velocity; R = the hydraulic radius (defined as the ratio of channel cross-section area to wetted perimeter); S = slope of the energy grade line (often approximated as channel bed slope); and C = conversion constant (1.00 for SI and 1.49 for foot-pound system.) Values of n, which is commonly known as Manning's roughness coefficient, can be estimated from textbook tables, such as that in Chaudhry (1993). Once velocity is thus estimated, channel travel time is computed as:

$$t_{channel} = \frac{L}{V} \tag{41}$$

where L = channel length.

Sheet flow is flow over the watershed land surface, before water reaches a channel. Distances are short—on the order of 10-100 meters (30-300 feet). The SCS suggests that sheet-flow travel time can be estimated as:

$$t_{sheet} = \frac{0.007(NL)^{0.8}}{(P_2)^{0.5}\,S^{0.4}} \tag{42}$$

in which N = an overland-flow roughness coefficient; L = flow length; P_2 = 2-year, 24-hour rainfall depth, in inches; and S = slope of hydraulic grade line, which may be approximated by the land slope. (This estimate is based upon an approximate solution of the kinematic wave equations, which are described later in this chapter.) Table 14 shows values of N for various surfaces.

Sheet flow usually turns to shallow concentrated flow after 100 meters. The average velocity for shallow concentrated flow can be estimated as:

$$V = \left\{ \begin{array}{ll} 16.1345\sqrt{S} & \textit{for unpaved surface} \\ 20.3282\sqrt{S} & \textit{for paved surface} \end{array} \right\} \tag{43}$$

From this, the travel time can be estimated with Equation 41.

Table 14. Overland-flow roughness coefficients for sheet-flow modeling (USACE, 1998)

Surface Description	*N*
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses, including species such as weeping love grass, bluegrass, buffalo grass, blue grass, and native grass mixtures	0.24
Bermudagrass	0.41
Range	0.13
Woods [1]	
Light underbrush	0.40
Dense underbrush	0.80

Notes:

[1] When selecting *N*, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Clark Unit Hydrograph Model

Clark's model derives a watershed UH by explicitly representing two critical processes in the transformation of excess precipitation to runoff:

- **Translation** or movement of the excess from its origin throughout the drainage to the watershed outlet.
- **Attenuation** or reduction of the magnitude of the discharge as the excess is stored throughout the watershed.

Basic Concepts and Equations

Short-term storage of water throughout a watershed—in the soil, on the surface, and in the channels—plays an important role in the transformation of precipitation excess to runoff. The linear reservoir model is a common representation of the effects of this storage. That model begins with the continuity equation:

$$\frac{dS}{dt} = I_t - O_t \tag{44}$$

in which dS/dt = time rate of change of water in storage at time t; I_t = average inflow to storage at time t; and O_t = outflow from storage at time t.

With the linear reservoir model, storage at time t is related to outflow as:

$$S_t = RO_t \tag{45}$$

where *R* = a constant linear reservoir parameter. Combining and solving the equations using a simple finite difference approximation yields:

$$O_t = C_A I_t + C_B O_{t-1} \tag{46}$$

where C_A, C_B = routing coefficients. The coefficients are calculated from:

$$C_A = \frac{\Delta t}{R + 0.5\Delta t} \tag{47}$$

$$C_B = 1 - C_A \tag{48}$$

The average outflow during period *t* is:

$$\overline{O_t} = \frac{O_{t-1} + O_t}{2} \tag{49}$$

With Clark's model, the linear reservoir represents the aggregated impacts of all watershed storage. Thus, conceptually, the reservoir may be considered to be located at the watershed outlet.

In addition to this lumped model of storage, the Clark model accounts for the time required for water to move to the watershed outlet. It does that with a linear channel model (Dooge, 1959), in which water is "routed" from remote points to the linear reservoir at the outlet with delay (translation), but without attenuation. This delay is represented implicitly with a so-called time-area histogram. That specifies the watershed area contributing to flow at the outlet as a function of time. If the area is multiplied by unit depth and divided by Δt, the computation time step, the result is inflow, I_t, to the linear reservoir.

Solving Equation 46 and Equation 49 recursively, with the inflow thus defined, yields values of $\overline{O_t}$. However, if the inflow ordinates in Equation 46 are runoff from a unit of excess, these reservoir outflow ordinates are, in fact, U_t, the UH.

[Note that as the solution of the equations is recursive, outflow will theoretically continue for an infinite duration. The program continues computation of the UH ordinates until the volume of the outflow exceeds 0.995 inches or mm. The UH ordinates are then adjusted using a depth-weighted consideration to produce a UH with a volume exactly equal to one unit of depth.]

Estimating the Model Parameters

Application of the Clark model requires:

- Properties of the time-area histogram.
- The storage coefficient, *R.*

As noted, the linear routing model properties are defined implicitly by a time-area histogram. Studies at HEC have shown that, even though a watershed-specific relationship can be developed, a smooth function fitted to a typical time-area relationship represents the temporal distribution adequately for UH derivation for most watersheds. That typical time-area relationship, which is built into the program, is:

$$\frac{A_t}{A} = \begin{cases} 1.414\left(\dfrac{t}{t_c}\right)^{1.5} & for\ t \le \dfrac{t_c}{2} \\ 1 - 1.414\left(1 - \dfrac{t}{t_c}\right)^{1.5} & for\ t \ge \dfrac{t_c}{2} \end{cases} \qquad (50)$$

where A_t = cumulative watershed area contributing at time t; A = total watershed area; and t_c = time of concentration of watershed. Application of this implementation only requires the parameter t_c, the time of concentration. This can be estimated via calibration, as described in Chapter 9, or it can be estimated using the procedures described earlier in the SCS UH section of this chapter.

The basin storage coefficient, R, is a index of the temporary storage of precipitation excess in the watershed as it drains to the outlet point. It, too, can be estimated via calibration if gaged precipitation and streamflow data are available. Though R has units of time, there is only a qualitative meaning for it in the physical sense. Clark (1945) indicated that R can be computed as the flow at the inflection point on the falling limb of the hydrograph divided by the time derivative of flow.

ModClark Model

In Chapter 2, models are categorized as *lumped-parameter models* or *distributed-parameter models*. A distributed parameter model is one in which spatial variability of characteristics and processes are considered explicitly. The modified Clark (ModClark) model is such a model (Kull and Feldman, 1998; Peters and Easton, 1996). This model accounts explicitly for variations in travel time to the watershed outlet from all regions of a watershed.

Basic Concepts and Equations

As with the Clark UH model, runoff computations with the ModClark model explicitly account for translation and storage. Storage is accounted for with the same linear reservoir model incorporated in the Clark model. Translation is accounted for with a grid-based travel-time model.

With the ModClark method, a grid is superimposed on the watershed. For each cell of the grid representation of the watershed, the distance to the watershed outlet is specified. Translation time to the outlet is computed as:

$$t_{cell} = t_c \frac{d_{cell}}{d_{\max}} \qquad (51)$$

where t_{cell} = time of travel for a cell, t_c = time of concentration for the watershed, d_{cell} = travel distance from a cell to the outlet, and d_{max} = travel distance for the cell that is most distant from the outlet.

The area of each cell is specified, and from this, the volume of inflow to the linear reservoir for each time interval, Δt, is computed as the product of area and precipitation excess. The excess is the difference in MAP on the cell and losses in the cell. The inflows thus computed are routed through a linear reservoir, yielding an outflow hydrograph for each cell. The program combines these cell outflow hydrographs to determine the basin direct runoff hydrograph.

Setting Up and Using the ModClark Model

To use the ModClark model, a gridded representation of the watershed is defined. Information about this representation is stored in a grid-parameter file; Figure 18 shows the contents of such a file. The file may be based upon an HRAP grid or HEC's standard hydrologic grid, and it can be generated by any means. A geographic information system (GIS) will permit automated preparation of the file; guidance (*GridParm*; USACE, 1996) and software tools (HEC-*GeoHMS*; USACE, 1999) for this task are available from HEC.

```
Parameter Order: Xcoord YCoord TravelLength Area
End:
Subbasin: 85
Grid Cell:    633    359    88.38    3.76
Grid Cell:    634    359    84.51    0.18
Grid Cell:    633    358    85.55    16.13
Grid Cell:    632    358    82.55    12.76
Grid Cell:    625    348    13.75    12.07
Grid Cell:    626    348    17.12    0.09
Grid Cell:    622    347    21.19    3.26
Grid Cell:    623    347    15.56    9.96
End:
Subbasin: 86
Grid Cell:    637    361    59.13    6.79
Grid Cell:    638    361    59.04    6.95
Grid Cell:    636    361    56.68    1.17
Grid Cell:    636    360    55.08    16.38
Grid Cell:    636    347    67.96    2.45
Grid Cell:    637    347    71.72    7.41
Grid Cell:    638    347    72.57    8.78
Grid Cell:    639    347    73.32    0.04
End:
```

Figure 18. Contents of a grid-cell file.

Kinematic Wave Model

As an alternative to the empirical UH models, HEC-HMS includes a conceptual model of watershed response. This model represents a watershed as an open channel (a very wide, open channel), with inflow to the channel equal to the excess precipitation. Then it solves the equations that simulate unsteady shallow water flow in an open channel to compute the watershed runoff hydrograph. This model is referred to as the kinematic-wave model. Details of the kinematic-wave model implemented in the program are presented in HEC's *Training document No. 10* (USACE, 1979).

Basic Concepts and Equations

Figure 19(a) shows a simple watershed for which runoff is to be computed for design, planning, or regulating. For kinematic wave routing, the watershed and its channels are conceptualized as shown in Figure 19(b). This represents the watershed as two plane surfaces over which water runs until it reaches the channel. The water then flows down the channel to the outlet. At a cross section, the system would resemble an open book, with the water running parallel to the text on the page (down the shaded planes) and then into the channel that follows the book’s center binding.

The kinematic wave overland flow model represents behavior of overland flow on the plane surfaces. The model may also be used to simulate behavior of flow in the watershed channels.

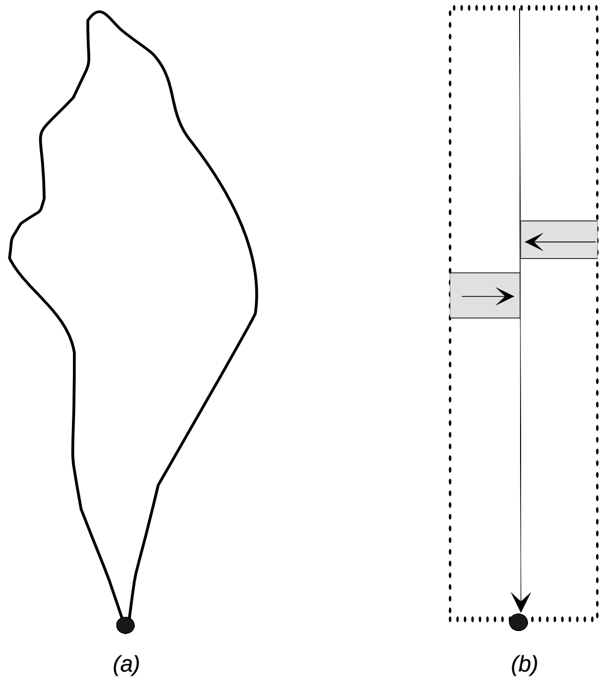


Figure 19. Simple watershed with kinematic-wave model representation.

Overland-flow model. At the heart of the overland model are the fundamental equations of open channel flow: the momentum equation and the continuity equation. Flow over the plane surfaces is primarily one-dimensional flow. In one dimension, the momentum equation is:

$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \quad (52)$$

where S_f = energy gradient (also known as the friction slope); S_0 = bottom slope; V = velocity; y = hydraulic depth; x = distance along the flow path, t = time; g = acceleration due to gravity; $(\partial y/\partial x)$ = pressure gradient; $(V/g)(\partial V/\partial x)$ = convective acceleration; and $(1/g)(\partial V/\partial t)$ = local acceleration. [This equation, these terms, and the basic concepts are described in detail in Chow (1959), Chaudhry (1993), and many other texts.]

The energy gradient can be estimated with Manning's equation (Equation 40), which can be written as:

$$Q = \frac{CR^{2/3}S_f^{1/2}}{N}A \quad (53)$$

where Q = flow, R = hydraulic radius, A = cross-sectional area, and N = a resistance factor that depends on the cover of the planes (note that this is not Manning's n). For shallow flow, bottom slope and the energy gradient are approximately equal and acceleration effects are negligible, so the momentum equation simplifies to:

$$S_f = S_o \tag{54}$$

Equation 53 can be simplified to:

$$Q = \alpha A^m \tag{55}$$

where α and *m* are parameters related to flow geometry and surface roughness.

The second critical equation, the one-dimensional representation of the continuity equation, is:

$$A\frac{\partial V}{\partial x} + VB\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} = q \tag{56}$$

where *B* = water surface width; *q* = lateral inflow per unit length of channel; $A(\partial V/\partial x)$ = prism storage; $VB(\partial y/\partial x)$ =wedge storage; and $B(\partial y/\partial t)$ =rate of rise. [Again, the equation, the terms, and the basic concepts are described in detail in Chow (1959), Chaudhry (1993), and other texts.] The lateral inflow represents the precipitation excess, computed as the difference in MAP and precipitation losses.

With simplification appropriate for shallow flow over a plane, the continuity equation reduces to:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{57}$$

Combining Equations 56 and 57 yields

$$\frac{\partial A}{\partial t} + \alpha m A^{(m-1)}\frac{\partial A}{\partial x} = q \tag{58}$$

This equation is a kinematic-wave approximation of the equations of motion. The program represents the overland flow element as a wide rectangular channel of unit width; α=1.486$S^{1/2}$/*N* and *m*=5/3. *N* is not Manning's *n*, but rather an overland flow roughness factor (Table 14).

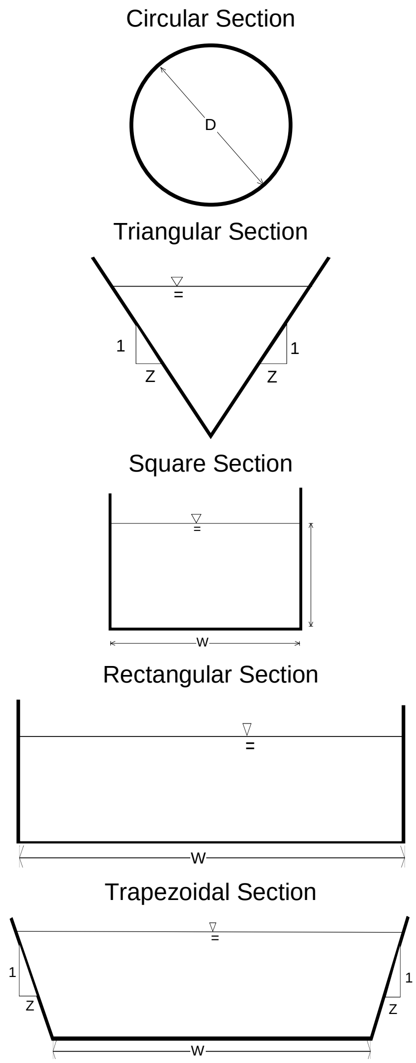
Channel-flow model. For certain classes of channel flow, conditions are such that the momentum equation can be simplified to the form shown as Equation 54. (These cases are defined in Chapter 8.) In those cases, the kinematic-wave approximation of Equation 58 is an appropriate model of channel flow. In the case of channel flow, the inflow in Equation 58 may be the runoff from watershed planes or the inflow from upstream channels.

Figure 20 shows values for α and *m* for various channel shapes used in the program. (The availability of a circular channel shape here does not imply that HEC-HMS can be used for analysis of pressure flow in a pipe system; it cannot. Note also that the circular channel shape only approximates the storage characteristics of a pipe or culvert. Because flow depths greater than the diameter of the circular channel shape can be computed with the kinematic-wave model, the user must verify that the results are appropriate.)

Solution of equations. The kinematic-wave approximation is solved in the same manner for either overland or channel flow:

- The partial differential equation is approximated with a finite-difference scheme.
- Initial and boundary conditions are assigned.
- The resulting algebraic equations are solved to find unknown hydrograph ordinates.

The overland-flow plane initial condition sets A, the area in Equation 58, equal to zero, with no inflow at the upstream boundary of the plane. The initial and boundary conditions for the kinematic wave channel model are based on the upstream hydrograph. Boundary conditions, either precipitation excess or lateral inflows, are constant within a time step and uniformly distributed along the element.



$$\alpha = \frac{0.804}{n} S^{\frac{1}{2}} D^{\frac{1}{6}}$$

m = 5/4

$$\alpha = \frac{0.94}{n} S^{\frac{1}{2}} \left(\frac{Z}{1+Z^2} \right)^{\frac{1}{3}}$$

m = 4/3

$$\alpha = \frac{0.72}{n} S^{\frac{1}{2}}$$

m = 4/4

$$\alpha = \frac{1.49}{n} S^{\frac{1}{2}} W^{-\frac{2}{3}}$$

m = 5/3

$$Q = \frac{1.49}{n} S^{\frac{1}{2}} A^{\frac{5}{3}} \left(\frac{1}{W + 2Y\sqrt{1+Z^2}} \right)^{\frac{2}{3}}$$

Figure 20. Kinematic wave parameters for various channel shapes (USACE, 1998)

In Equation 58, A is the only dependent variable, as α and m are constants, so solution requires only finding values of A at different times and locations. To do so, the finite difference scheme approximates $\partial A/\partial t$ as $\Delta A/\Delta t$, a difference in area in successive times, and it approximates $\partial A/\partial x$ as $\Delta A/\Delta x$, a difference in area at adjacent locations, using a scheme proposed by Leclerc and Schaake (1973). The resulting algebraic equation is:

$$\frac{A_i^j - A_i^{j-1}}{\Delta t} + \alpha m \left[\frac{A_i^{j-1} + A_{i-1}^{j-1}}{2}\right]^{m-1} \left[\frac{A_i^{j-1} - A_{i-1}^{j-1}}{\Delta x}\right] = \frac{q_i^j + q_i^{j-1}}{2} \qquad (59)$$

Equation 59 is the so-called standard form of the finite-difference approximation. The indices of the approximation refer to positions on a space-time grid, as shown in Figure 21. That grid provides a convenient way to visualize the manner in which the solution scheme solves for unknown values of A at various locations and times. The index i indicates the current location at which A is to be found along the length, L, of the channel or overland flow plane. The index j indicates the current time step of the solution scheme. Indices i-1, and j-1 indicate, respectively, positions and times removed a value Δx and Δt from the current location and time in the solution scheme.

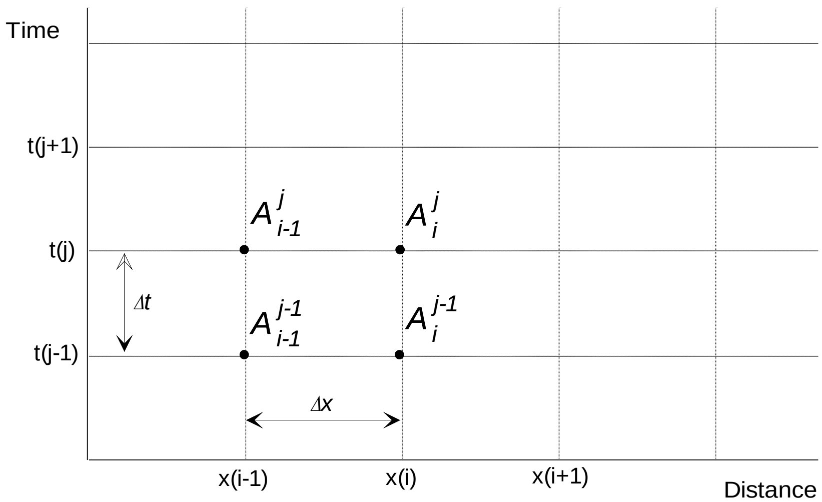


Figure 21. Finite difference method space-time grid.

With the solution scheme proposed, the only unknown value in Equation 59 is the current value at a given location, A_i^j. All other values of A are known from either a solution of the equation at a previous location and time, or from an initial or boundary condition. The program solves for the unknown as:

$$A_i^j = q_a \Delta t + A_i^{j-1} - \alpha m \left[\frac{\Delta t}{\Delta x}\right] \left[\frac{A_i^{j-1} + A_{i-1}^{j-1}}{2}\right]^{m-1} \left[A_i^{j-1} - A_{i-1}^{j-1}\right] \qquad (60)$$

The flow is computed as:

$$Q_i^j = \alpha \left[A_i^j\right]^m \qquad (61)$$

This standard form of the finite difference equation is applied when the following stability factor, R, is less than 1.00 (see Alley and Smith, 1987):

$$R = \frac{\alpha}{q_a \Delta x}\left[\left(q_a \Delta t + A_{i-1}^{j-1^m}\right) - A_{i-1}^{j-1^m}\right]; \ q_a > 0 \tag{62}$$

or

$$R = \alpha m A_{i-1}^{j-1^{m-1}} \frac{\Delta t}{\Delta x} q_a; \quad q_a = 0 \tag{63}$$

If R is greater than 1.00, then the following finite difference approximation is used:

$$\frac{Q_i^j - Q_{i-1}^j}{\Delta x} + \frac{A_{i-1}^j - A_{i-1}^{j-1}}{\Delta t} = q_a \tag{64}$$

where Q_i^j is the only unknown. This is referred to as the conservation form. Solving for the unknown yields:

$$Q_i^j = Q_{i-1}^j + q\Delta x - \frac{\Delta x}{\Delta t}\left[A_{i-1}^j - A_{i-1}^{j-1}\right] \tag{65}$$

When Q_i^j is found, the area is computed as

$$A_i^j = \left[\frac{Q_i^j}{\alpha}\right]^{\frac{1}{m}} \tag{66}$$

Accuracy and stability. HEC-HMS uses a finite difference scheme that ensures accuracy and stability. Accuracy refers to the ability of the solution procedure to reproduce the terms of the differential equation without introducing minor errors that affect the solution. For example, if the solution approximates $\partial A/\partial x$ as $\Delta A/\Delta x$, and a very large Δx is selected, then the solution will not be accurate. Using a large Δx introduces significant errors in the approximation of the partial derivative. Stability refers to the ability of the solution scheme to control errors, particularly numerical errors that lead to a worthless solution. For example, if by selecting a very small Δx, an instability may be introduced. With small Δx, many computations are required to simulate a long channel reach or overland flow plan. Each computation on a digital computer inherently is subject to some round-off error. The round-off error accumulates with the recursive solution scheme used by the program, so in the end, the accumulated error may be so great that a solution is not found.

An accurate solution can be found with a stable algorithm when $\Delta x/\Delta t \approx c$, where c = average kinematic-wave speed over a distance increment Δx. But the kinematic-wave speed is a function of flow depth, so it varies with time and location. The program must select Δx and Δt to account for this. To do so, it initially selects $\Delta x = c\Delta t_m$ where c = estimated maximum wave speed, depending on the lateral and upstream inflows; and Δt_m = time step equal to the minimum of:

1. One-third the plane or reach length divided by the wave speed.
2. One-sixth the upstream hydrograph rise time for a channel.
3. The specified computation interval.

Finally, Δx is chosen as: the minimum of this computed Δx and the reach, or plane length divided by the number of distance steps (segments) specified in the input form for the kinematic-wave models. The minimum default value is two segments.

When Δx is set, the finite difference scheme varies Δt when solving Equation 61 or Equation 66 to maintain the desired relationship between Δx, Δt and c. However, the program reports results at the specified constant time interval.

Setting Up and Using the Kinematic Wave Model

To estimate runoff with the kinematic-wave model, the watershed is described as a set of elements that include:

- **Overland flow planes**. Up to two planes that contribute runoff to channels within the watershed can be described. The combined flow from the planes is the total inflow to the watershed channels. Column 1 of Table 15 shows information that must be provided about each plane.

- **Subcollector channels**. These are small feeder pipes or channels, with principle dimension generally less that 18 inches, that convey water from street surfaces, rooftops, lawns, and so on. They might service a portion of a city block or housing tract, with area of 10 acres. Flow is assumed to enter the channel uniformly along its length. The average contributing area for each subcollector channel must be specified. Column 2 of Table 15 shows information that must be provided about the subcollector channels.

- **Collector channels**. These are channels, with principle dimension generally 18-24 inches, which collect flows from subcollector channels and convey it to the main channel. Collector channels might service an entire city block or a housing tract, with flow entering laterally along the length of the channel. As with the subcollectors, the average contributing area for each collector channel is required. Column 2 of Table 15 shows information that must be provided about the collector channels.

- **The main channel**. This channel conveys flow from upstream subwatersheds and flows that enter from the collector channels or overland flow planes. Column 3 of Table 15 shows information that must be provided about the main channel.

The choice of elements to describe any watershed depends upon the configuration of the drainage system. The minimum configuration is one overland flow plane and the main channel, while the most complex would include two planes, subcollectors, collectors, and the main channel.

The planes and channels are described by representative slopes, lengths, shapes, and contributing areas. Publications from HEC (USACE, 1979; USACE, 1998) provide guidance on how to choose values and give examples.

The roughness coefficients for both overland flow planes and channels commonly are estimated as a function of surface cover, using, for example, Table 14, for overland flow planes and the tables in Chow (1959) and other texts for channel n values.

Table 15. Information needs for kinematic wave modeling.

Overland Flow Planes	Collectors and Subcollectors	Main Channel
Typical length	Area drained by channel	Channel length
Representative slope	Representative channel length	Description of channel shape
Overland-flow roughness coefficient	Description of channel shape	Principle dimensions of channel cross section
Area represented by plane	Principle dimensions of representative channel cross section	Channel slope
Loss model parameters (see Chapter 5)	Representative channel slope	Representative Manning's roughness coefficient
	Representative Manning's roughness coefficient	Identification of upstream inflow hydrograph (if any)

Applicability and Limitations of Direct Runoff Models

Choice of a direct runoff model from amongst the included options depends upon:

- **Availability of information for calibration or parameter estimation**. Use of the parametric UH models requires specifying model parameters. Use one of the empirical parameter predictors, such as Equation 35, to compute parameters. However, the optimal source of these parameters is calibration, as described in Chapter 9. If the necessary data for such calibration in an urban watershed is not available, then the kinematic-wave model may be the best choice, as the parameters and information required to use that model are related to measurable and observable watershed properties.

- **Appropriateness of the assumptions inherent in the model**. Each of the models is based upon one or more basic assumptions; if these are violated, then avoid the use of the model. For example, the SCS UH model assumes that the watershed UH is a single-peaked hydrograph. If all available information indicates that the shape of the watershed and the configuration of the drainage network causes multiple peaks for even simple storms, then the SCS UH should not be used.

  Likewise, the kinematic wave model is not universally applicable: Ponce (1991) for example, argues that because of numerical properties of the solution algorithms, the method "…is intended primarily for small watersheds [those less than 1 sq mi (2.5 km^2)], particularly in the cases in which it is possible to resolve the physical detail without compromising the deterministic nature of the model." Thus, for a larger watershed, one of the UH models is perhaps a better choice.

- **User preference and experience**. A combination of experience and preference should guide the choice of models. As noted in Chapter 5, experience is a critical factor in the success of a modeling effort. However, be careful in using a particular model with a given parameter just because that seems to be the standard of practice. For example, do not automatically assume that t_{lag} = 0.6 t_c for the SCS UH method.

Instead, make best use of available data to confirm this parameter estimate.

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CHAPTER 7

Modeling Baseflow

Two distinguishable components of a streamflow hydrograph are (1) direct, quick runoff of precipitation, and (2) baseflow. Baseflow is the sustained or "fair-weather" runoff of prior precipitation that was stored temporarily in the watershed, plus the delayed subsurface runoff from the current storm. Some conceptual models of watershed processes account explicitly for this storage and for the subsurface movement. However this accounting is not necessary to provide the information for many water resources studies.

The program includes three alternative models of baseflow:

- Constant, monthly-varying value.
- Exponential recession model.
- Linear-reservoir volume accounting model.

Basic Concepts and Implementation

Constant , Monthly-Varying Baseflow

This is the simplest baseflow model included in the program. It represents baseflow as a constant flow; this may vary monthly. This user-specified flow is added to the direct runoff computed from rainfall for each time step of the simulation.

Exponential Recession Model

The program includes a exponential recession model to represent watershed baseflow (Chow, Maidment, and Mays, 1988). The recession model has been used often to explain the drainage from natural storage in a watershed (Linsley et al, 1982). It defines the relationship of Q_t, the baseflow at any time t, to an initial value as:

$$Q_t = Q_0 k^t \tag{67}$$

where Q_0 = initial baseflow (at time zero); and k = an exponential decay constant. The baseflow thus computed is illustrated in Figure 22. The shaded region represents baseflow in this figure; the contribution decays exponentially from the starting flow. Total flow is the sum of the baseflow and the direct surface runoff.

As implemented in the program, k is defined as the ratio of the baseflow at time t to the baseflow one day earlier. The starting baseflow value, Q_0, is an initial condition of the model. It may be specified as a flow rate (m^3/s or cfs), or it may be specified as a flow per unit area ($m^3/s/km^2$ or cfs/sq mi).

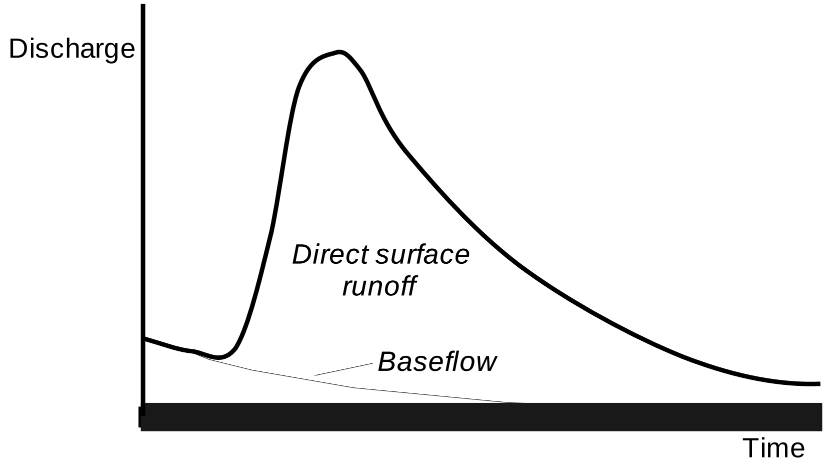

Figure 22. Initial baseflow recession.

The recession baseflow model is applied both at the start of simulation of a storm event, and later in the event as the delayed subsurface flow reaches the watershed channels, as illustrated in Figure 23. Here, after the peak of the direct runoff, a user-specified threshold flow defines the time at which the recession model of Equation 67 defines the total flow. That threshold may be specified as a flow rate or as a ratio to the computed peak flow. For example, if the threshold is specified as a ratio-to-peak of 0.10, and the computed peak is 1000 m^3/s, then the threshold flow is 100 m^3/s. Subsequent total flows are computed with Equation 67, with Q_0 = the specified threshold value.

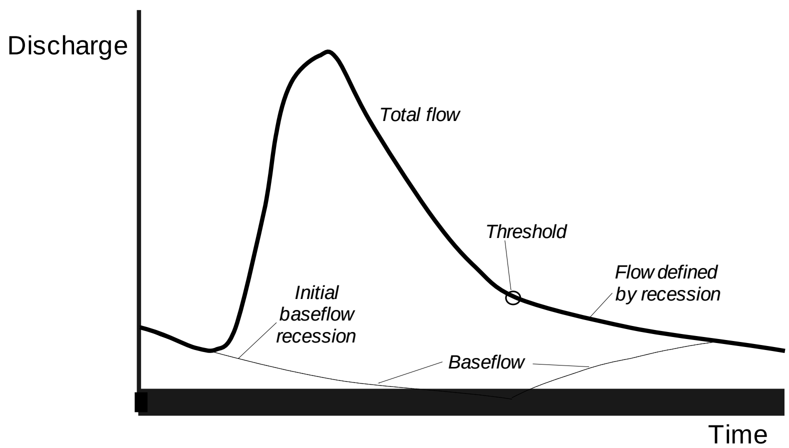

Figure 23. Baseflow model illustration.

At the threshold flow, baseflow is defined by the initial baseflow recession. Thereafter, baseflow is not computed directly, but is defined as the recession flow less the direct-surface-runoff. When the direct-surface runoff eventually reaches zero (all rainfall has run off the watershed), the total flow and baseflow are identical.

After the threshold flow occurs, the streamflow hydrograph ordinates are defined by the recession model alone, *unless* the direct runoff plus initial baseflow recession contribution exceeds the threshold. This may be the case if subsequent precipitation causes a second rise in the hydrograph, as illustrated in Figure 24. In that case, ordinates on the second rising limb are computed by adding direct runoff to the initial recession, as illustrated.

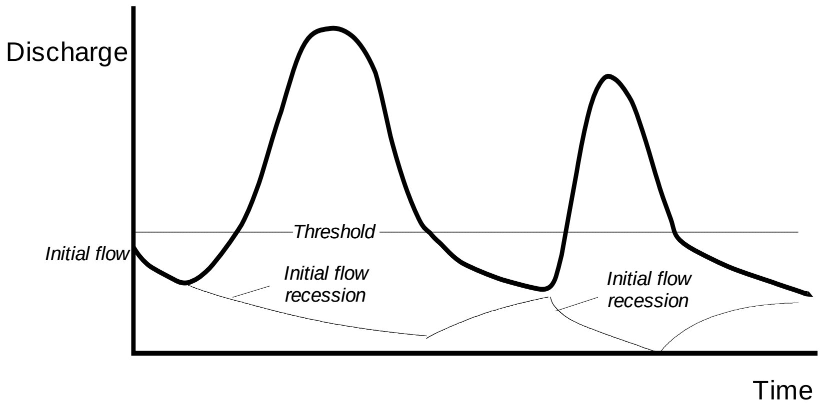

Figure 24. Recession with multiple runoff peaks.

Linear Reservoir Model

The linear-reservoir baseflow model is used in conjunction with the continuous soil-moisture accounting (SMA) model that is described in Chapter 5. This baseflow model simulates the storage and movement of subsurface flow as storage and movement of water through reservoirs. The reservoirs are linear: the outflow at each time step of the simulation is a linear function of the average storage during the time step. Mathematically, this is identical to the manner in which Clark's UH model represents watershed runoff, as described in Chapter 6.

The outflow from groundwater layer 1 of the SMA is inflow to one linear reservoir, and the outflow from groundwater layer 2 of the SMA is inflow to another. The outflow from the two linear reservoirs is combined to compute the total baseflow for the watershed.

Estimating Baseflow Model Parameters

Constant , Monthly-Varying Baseflow

The parameters of this model are the monthly baseflows. These are best estimated empirically, with measurements of channel flow when storm runoff is not occurring. In the absence of such records, field inspection may help establish the average flow. For large watersheds with contribution from groundwater flow and for watersheds with year-round precipitation, the contribution may be significant and should not be ignored. On the other hand, for most urban channels and for smaller streams in the western and southwestern US, the baseflow contribution may be negligible.

Exponential Recession Model

The parameters of this model include the initial flow, the recession ratio, and the threshold flow. As noted, the initial flow is an initial condition. For analysis of hypothetical storm runoff, initial flow should be selected as a likely average flow that would occur at the start of the storm runoff. For frequent events, the initial flow might be the average annual flow in the channel. Field inspection may help establish this. As with the constant, monthly-varying baseflow, for most urban channels and for smaller streams in the western and southwestern US, this may well be zero, as the baseflow contribution is negligible.

The recession constant, k, depends upon the source of baseflow. If k = 1.00, the baseflow contribution will be constant, with all $Q_t = Q_0$. Otherwise to model the exponential decay typical of natural undeveloped watersheds, *k* must be less than 1.00. Table 16 shows typical values proposed by Pilgrim and Cordery (1992) for basins ranging in size from 300 to 16,000 km^2 (120 to 6500 square miles) in the US, eastern Australia, and several other regions. Large watersheds may have k values at the upper end of the range, while smaller watersheds will have values at the lower end.

Table 16. Typical recession constant values.

Flow Component	Recession Constant, Daily
Groundwater	0.95
Interflow	0.8-0.9
Surface runoff	0.3-0.8

The recession constant can be estimated if gaged flow data are available. Flows prior to the start of direct runoff can be plotted, and an average of ratios of ordinates spaced one day apart can be computed. This is simplified if a logarithmic axis is used for the flows, as the recession model will plot as a straight line.

The threshold value can be estimated also from examination of a graph of observed flows versus time. The flow at which the recession limb is approximated well by a straight line defines the threshold value.

Linear Reservoir Model

The linear reservoir model is used with soil-moisture accounting model. It is best calibrated using procedures consistent with those used to calibrate that model.

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CHAPTER 8

Modeling Channel Flow

This section describes the models of channel flow that are included in the program; these are also known as routing models. The routing models available include:

- Lag
- Muskingum
- Modified Puls, also known as storage routing
- Kinematic-wave
- Muskingum Cunge

Each of these models computes a downstream hydrograph, given an upstream hydrograph as a boundary condition. Each does so by solving the continuity and momentum equations. This chapter presents a brief review of the fundamental equations, simplifications, and solutions to alternative models.

The routing models that are included are appropriate for many, but not all, flood runoff studies. The latter part of this chapter describes how to pick the proper model.

Open-Channel Flow Equations and Solution Techniques

Basic Equations of Open-Channel Flow

At the heart of the routing models included in the program are the fundamental equations of open channel flow: the momentum equation and the continuity equation. Together the two equations are known as the St. Venant equations or the dynamic wave equations.

The momentum equation accounts for forces that act on a body of water in an open channel. In simple terms, it equates the sum of gravitational force, pressure force, and friction force to the product of fluid mass and acceleration. In one dimension, the equation is written as:

$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \tag{68}$$

where S_f = energy gradient (also known as the friction slope); S_0 = bottom slope; V = velocity; y = hydraulic depth; x = distance along the flow path; t = time; g = acceleration due to gravity; $\partial y/\partial x$ = pressure gradient; $(V/g)(\partial V/\partial x)$ = convective acceleration; and $(1/g)(\partial V/\partial t)$ = local acceleration.

The continuity equation accounts for the volume of water in a reach of an open channel, including that flowing into the reach, that flowing out of the reach, and that stored in the reach. In one-dimension, the equation is:

$$A\frac{\partial V}{\partial x}+VB\frac{\partial y}{\partial x}+B\frac{\partial y}{\partial t}=q \quad (69)$$

where B = water surface width; and q = lateral inflow per unit length of channel. Each of the terms in this equation describes inflow to, outflow from, or storage in a reach of channel, a lake or pond, or a reservoir. Henderson (1966) described the terms as $A(\partial V/\partial x)$ = prism storage; $VB(\partial y/\partial x)$ = wedge storage; and $B(\partial y/\partial t)$ = rate of rise.

The momentum and continuity equations are derived from basic principles, assuming:

- Velocity is constant, and the water surface is horizontal across any channel section.
- All flow is gradually varied, with hydrostatic pressure prevailing at all points in the flow. Thus vertical accelerations can be neglected.
- No lateral, secondary circulation occurs.
- Channel boundaries are fixed; erosion and deposition do not alter the shape of a channel cross section.

Water is of uniform density, and resistance to flow can be described by empirical formulas, such as Manning's and Chezy's equation.

Approximations

Although the solution of the full equations is appropriate for all one-dimensional channel-flow problems, and necessary for many, approximations of the full equations are adequate for typical flood routing needs. These approximations typically combine the continuity equation (Equation 69) with a simplified momentum equation that includes only relevant and significant terms.

Henderson (1966) illustrates this with an example for a steep alluvial stream with an inflow hydrograph in which the flow increased from 10,000 cfs to 150,000 cfs and decreased again to 10,000 cfs within 24 hours. Table 17 shows the terms of the momentum equation and the approximate magnitudes that he found. The force associated with the stream bed slope is the most important. If the other terms are omitted from the momentum equation, any error in solution is likely to be insignificant. Thus, for this case, the following simplification of the momentum equation may be used:

$$S_f = S_0 \quad (70)$$

If this simplified momentum equation is combined with the continuity equation, the result is the kinematic wave approximation, which is described in Chapter 6.

Table 17. Relative magnitude of momentum equation terms for steep channel, rapidly-rising hydrograph (from Henderson, 1966)

Flow Component	Recession Constant, Daily
S_o (bottom slope)	26
$\frac{\partial y}{\partial x}$ (pressure gradient)	0.5
$\frac{V}{g}\frac{\partial V}{\partial X}$ (convective acceleration)	0.12 – 0.25
$\frac{1}{g}\frac{\partial V}{\partial t}$ (local acceleration)	0.05

Other common approximations of the momentum equation include:

- **Diffusion wave approximation**. This approximation is the basis of the Muskingum-Cunge routing model that is described subsequently in this chapter.

$$S_f = S_0 - \frac{\partial y}{\partial x} \tag{71}$$

- **Quasi-steady dynamic-wave approximation**. This approximation is often used for water-surface profile computations along a channel reach, given a steady flow. It is incorporated in computer programs HEC-2 (USACE, 1990) and HEC-RAS (USACE, 1998).

$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} \tag{72}$$

Solution Methods

In HEC-HMS, the various approximations of the continuity and momentum equations are solved using the finite difference method. In this method, finite difference equations are formulated from the original partial differential equations. For example, $\partial V/\partial t$ from the momentum equation is approximated as $\Delta V/\Delta t$, a difference in velocity in successive time steps Δt, and $\partial V/\partial x$ is approximated as $\Delta V/\Delta x$, a difference in velocity at successive locations spaced at Δx. Substituting these approximations into the partial differential equations yields a set of algebraic equations. Depending upon the manner in which the differences are computed, the algebraic equations may be solved with either an explicit or an implicit scheme. With an explicit scheme, the unknown values are found recursively for a constant time, moving from one location along the channel to another. The results of one computation are necessary for the next. With an implicit scheme, all the unknown values for a given time are found simultaneously.

Parameters, Initial Conditions, and Boundary Conditions

The basic information requirements for all routing models are:

- **A description of the channel**. All routing models that are included in the program require a description of the channel. In some of the models, this description is implicit in parameters of the model. In others, the description is provided in more common terms: channel width, bed slope, cross-section shape, or the equivalent.

- **Energy-loss model parameters**. All routing models incorporate some type of energy-loss model. The physically-based routing models, such as the kinematic-wave model and the Muskingum-Cunge model use Manning's equation and Manning's roughness coefficients (*n* values). Other models represent the energy loss empirically.
- **Initial conditions**. All routing models require initial conditions: the flow (or stage) at the downstream cross section of a channel prior to the first time period. For example, the initial downstream flow could be estimated as the baseflow within the channel at the start of the simulation, as the initial inflow, or as downstream flow likely to occur during a hypothetical event.
- **Boundary conditions**. The boundary conditions for routing models are the upstream inflow, lateral inflow, and tributary inflow hydrographs. These may be observed historical events, or they may be computed with the precipitation-runoff models included in the program.

Modified Puls Model

Basic Concepts and Equations

The Modified Puls routing method, also known as storage routing or level-pool routing, is based upon a finite difference approximation of the continuity equation, coupled with an empirical representation of the momentum equation (Chow, 1964; Henderson, 1966).

For the Modified Puls model, the continuity equation is written as

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (73)$$

This simplification assumes that the lateral inflow is insignificant, and it allows width to change with respect to location. Rearranging this equation and incorporating a finite-difference approximation for the partial derivatives yields:

$$\overline{I_t} - \overline{O_t} = \frac{\Delta S_t}{\Delta t} \qquad (74)$$

where $\overline{I_t}$ = average upstream flow (inflow to reach) during a period Δt; $\overline{O_t}$ = average downstream flow (outflow from reach) during the same period; and ΔS_t = change in storage in the reach during the period. Using a simple backward differencing scheme and rearranging the result to isolate the unknown values yields:

$$\left(\frac{S_t}{\Delta t} + \frac{O_t}{2}\right) = \left(\frac{I_{t-1} + I_t}{2}\right) + \left(\frac{S_{t-1}}{\Delta t} - \frac{O_{t-1}}{2}\right) \qquad (75)$$

in which I_{t-1} and I_t = inflow hydrograph ordinates at times t-1 and t, respectively; O_{t-1} and O_t = outflow hydrograph ordinates at times t-1 and t, respectively; and S_{t-1} and S_t = storage in reach at times t-1 and t, respectively. At time t, all terms on the right-hand side of this equation are known, and terms on the left-hand side are to be found. Thus, the equation has two unknowns at time t: S_t and O_t.

A functional relationship between storage and outflow is required to solve Equation 75. Once that function is established, it is substituted into Equation 75, reducing the equation to a nonlinear equation with a single unknown, O_t. This equation is solved recursively by the program, using a trial-and-error procedure. [Note that at the first time t, the outflow at time t-1 must be specified to permit recursive solution of the equation; this outflow is the initial outflow condition for the storage routing model.]

Defining the Storage-Outflow Relationship

The storage-outflow relationship required for the Modified Puls routing model can be determined with:

- **Water-surface profiles computed with a hydraulics model**. Steady-flow water surface profiles, computed for a range of discharges with programs like HEC-2 (USACE, 1990), HEC-RAS (USACE, 1998), or a similar model, define a relationship of storage to flow between two channel cross sections.

  Figure 25 illustrates this; it shows a set of water-surface profiles between cross section *A* and cross section *B* of a channel. These profiles were computed for a set of steady flows, Q_1, Q_2, Q_3, *and* Q_4.

  For each profile, the volume of water in the reach, S_i, can be computed, using solid geometry principles. In the simplest case, if the profile is approximately planar, the volume can be computed by multiplying the average cross-section area bounded by the water surface by the reach length. Otherwise, another numerical integration method can be used. If each computed volume is associated with the steady flow with which the profile is computed, the result is a set of points on the required storage-outflow relationship.

  This procedure can be used with existing or with proposed channel configurations. For example, to evaluate the impact of a proposed channel project, the channel cross sections can be modified, water surface profiles recalculated, and a revised storage-outflow relationship developed.

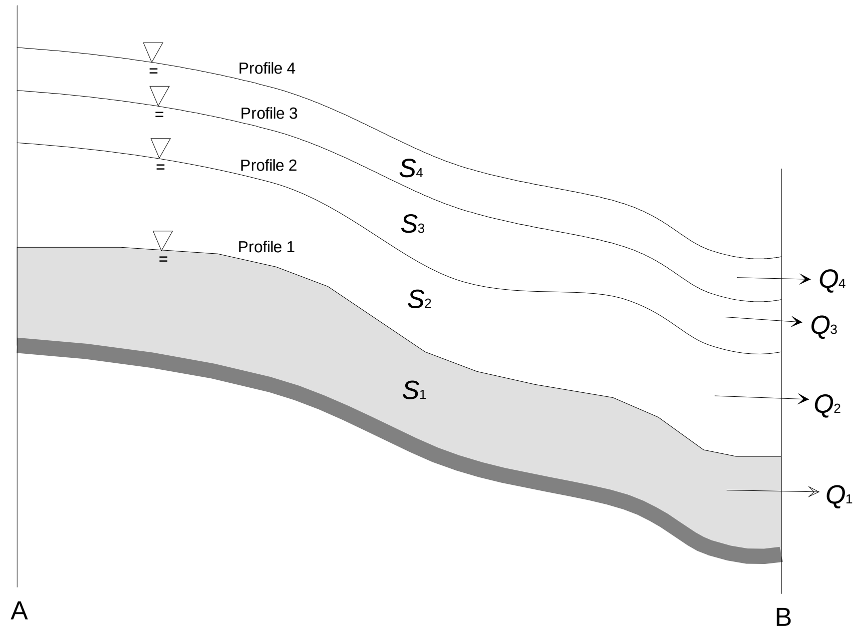

Figure 25. Steady-flow water-surface profiles and storage-outflow curve.

- **Historical observations of flow and stage**. Observed water surface profiles, obtained from high water marks, can be used to define the required storage-outflow relationships, in much the same manner that computed water-surface profiles are used. Each observed discharge-elevation pair provides information for establishing a point of the relationship.

  Sufficient stage data over a range of floods is required to establish the storage-outflow relationship in this manner. If only a limited set of observations is available, these may best be used to calibrate a water-surface profile-model for the channel reach of interest. Then that calibrated model can be exercised to establish the storage-outflow relationship as described above.

- **Calibration, using observed inflow and outflow hydrographs for the reach of interest**. Observed inflow and outflow hydrographs can be used to compute channel storage by an inverse process of flood routing. When both inflow and outflow are known, the change in storage can be computed using Equation 74. Then, the storage-outflow function can be developed empirically. Note that tributary inflow, if any, must also be accounted for in this calculation.

  Inflow and outflow hydrographs also can be used to find the storage-outflow function by trial-and-error. In that case, a candidate function is defined and used to route the inflow hydrograph. The outflow hydrograph thus computed is compared with the observed hydrograph. If the match is not adequate, the function is adjusted, and the process is repeated. Chapter 9 provides more information regarding this process, which is referred to as calibration.

Estimating Other Model Parameters

Chapter 6 of this manual describes how an accurate solution of the finite difference form of the kinematic-wave model requires careful selection of Δx and Δt; this is also true for solution of the storage-routing model equations. For the kinematic-wave model, an accurate solution can be found with a stable algorithm when $\Delta x/\Delta t \approx c$, where c = average wave speed over a distance increment Δx. This rule applies also with storage routing. As implemented in the program, Δx for the finite difference approximation of $\partial Q/\partial x$ is implicitly equal to the channel reach length, L, divided by an integer number of steps. The goal is to select the number of steps so that the travel time through the reach is approximately equal the time step Δt. This is given approximately by:

$$steps = \frac{L}{c\Delta t} \tag{76}$$

The number of steps affects the computed attenuation of the hydrograph. As the number of routing steps increases, the amount of attenuation decreases. The maximum attenuation corresponds to one step; this is used commonly for routing though ponds, lakes, wide, flat floodplains, and channels in which the flow is heavily controlled by downstream conditions. Strelkoff (1980) suggests that for locally-controlled flow, typical of steeper channels:

$$steps = 2L\frac{S_0}{y_0} \tag{77}$$

where y_o = normal depth associated with baseflow in the channel. EM 1110-2-1417 points out that this parameter, however, is best determined by calibration, using observed inflow and outflow hydrographs.

Muskingum Model

Basic Concepts and Equations

The Muskingum routing model, like the modified Puls model, uses a simple finite difference approximation of the continuity equation:

$$\left(\frac{I_{t-1}+I_t}{2}\right)-\left(\frac{O_{t-1}+O_t}{2}\right)=\left(\frac{S_t-S_{t-1}}{\Delta t}\right) \tag{78}$$

Storage in the reach is modeled as the sum of prism storage and wedge storage. As shown in Figure 26, prism storage is the volume defined by a steady-flow water surface profile, while wedge storage is the additional volume under the profile of the flood wave. During rising stages of the flood, wedge storage is positive and is added to the prism storage. During the falling stages of a flood, the wedge storage is negative and is subtracted from the prism storage.

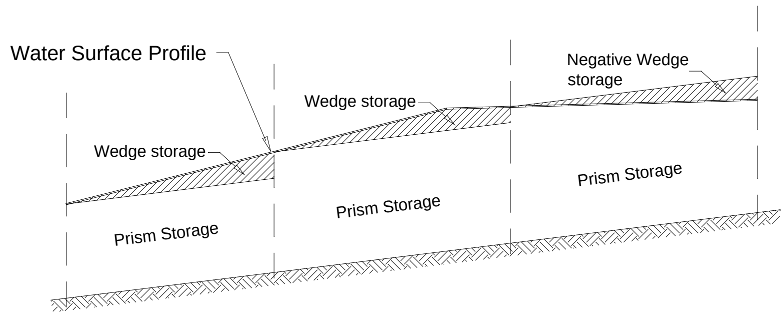

Figure 26. Wedge storage (from Linsley et al., 1982)

The volume of prism storage is the outflow rate, *O*, multiplied by the travel time through the reach, *K*. The volume of wedge storage is a weighted difference between inflow and outflow, multiplied by the travel time *K*. Thus, the Muskingum model defines the storage as:

$$S_t = KO_t + KX\left(I_t - O_t\right) = K[XI_t + (1-X)O_t] \qquad (79)$$

where *K* = travel time of the flood wave through routing reach; and *X* = dimensionless weight ($0 \leq X \leq 0.5$).

The quantity $X\, I_t + (1\text{-}X)\, O_t$ is a weighted discharge. If storage in the channel is controlled by downstream conditions, such that storage and outflow are highly correlated, then $X = 0.0$. In that case, Equation 82 resolves to $S = KO$; this is the linear reservoir model that was described in Chapter 6. If $X = 0.5$, equal weight is given to inflow and outflow, and the result is a uniformly progressive wave that does not attenuate as it moves through the reach.

If Equation 78 is substituted into Equation 79 and the result is rearranged to isolate the unknown values at time *t*, the result is:

$$O_t = \left(\frac{\Delta t - 2KX}{2K\left(1-X\right)+\Delta t}\right) I_t + \left(\frac{\Delta t + 2KX}{2K\left(1-X\right)+\Delta t}\right) I_{t-1} + \left(\frac{2K\left(1-X\right)-\Delta t}{2K\left(1-X\right)+\Delta t}\right) O_{t-1} \qquad (80)$$

The program solves Equation 80 recursively to compute ordinates of the outflow hydrograph, given the inflow hydrograph ordinates (I_t for all *t*), an initial condition ($O_{t=0}$), and the parameters, *K* and *X*.

Estimating the Model Parameters

Constraints on the parameters. As noted, the feasible range for the parameter *X* is (0, 0.5). However, these other constraints apply to selection of *X* and the parameter *K*:

- As with other routing models, an accurate solution requires selection of appropriate time steps, distance steps, and parameters to ensure accuracy and stability of the solution. With Muskingum routing, as with modified Puls routing, the distance step, Δx, is defined indirectly by the number of steps into which a reach is divided for routing. And as with other models, $\Delta x/\Delta t$ is selected to approximate c , where c = average wave speed over a distance increment Δx. With the Muskingum model, the wave speed is K/L, so the number of steps should be approximately $K/\Delta t$.

- The parameters K and X and the computational time step Δt also must be selected to ensure that the Muskingum model, as represented by Equations 8-15 and 8-16, is rational. That means that the parenthetical terms must be non-negative; the values of K and X must be chosen so that the combination falls within the shaded region shown in Figure 27.

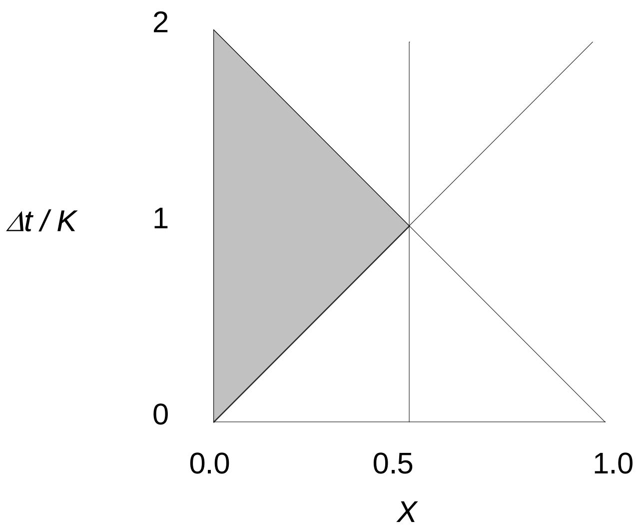

Figure 27. Feasible region for Muskingum model parameters.

Calibrating the model using observed flows. If observed inflow and outflow hydrographs are available, the Muskingum model parameter K can be estimated as the interval between similar points on the inflow and outflow hydrographs. For example, K can be estimated as the elapsed time between the centroid of areas of the two hydrographs, as the time between the hydrograph peaks, or as the time between midpoints of the rising limbs. Once K is estimated, X can be estimated by trial and error.

Chapter 9 describes the calibration capability of the program; this may be used with parameters of the Muskingum model. In that case, both K and X may be estimated by trial-and-error.

Estimating the parameters for ungaged watersheds. If gaged flows required for calibration are not available, K and X can be estimated from channel characteristics. For example, EM 1110-2-1417 proposes estimating K as follows:

- Estimate the flood wave velocity, V_w, using Seddon's law, as:

$$V_w = \frac{1}{B}\frac{dQ}{dy} \tag{81}$$

where B = top width of the water surface, and dQ/dy = slope of the discharge rating curve at a representative channel cross section. As an alternative, EM 1110-2-1417 suggests estimating the flood wave velocity as 1.33-1.67 times the average velocity, which may be estimated with Manning's equation and representative cross section geometric information.

- Estimate K as:

$$K = \frac{L}{V_w} \tag{82}$$

Experience has shown that for channels with mild slopes and over-bank flow, the parameter X will approach 0.0. For steeper streams, with well-defined channels that do not have flows going out of bank, X will be closer to 0.5. Most natural channels lie somewhere in between these two limits, leaving room for engineering judgement. Cunge (1969) estimated X as

$$X = \frac{1}{2}\left(1 - \frac{Q_o}{BS_o c\Delta x}\right) \tag{83}$$

where Q_o = a reference flow from the inflow hydrograph; B = top width of flow area; S_o = friction slope or bed slope; c = flood wave speed (celerity); and Δx = the length of reach. The reference flow is an average value for the hydrograph, midway between the base flow and the peak flow (Ponce, 1983).

Lag Model

Basic Concept

This is the simplest of the included routing models. With it, the outflow hydrograph is simply the inflow hydrograph, but with all ordinates translated (lagged in time) by a specified duration. The flows are not attenuated, so the shape is not changed. This model is widely used, especially in urban drainage channels (Pilgrim and Cordery, 1993).

Mathematically, the downstream ordinates are computed by:

$$O_t = \begin{Bmatrix} I_t & t < lag \\ I_{t-lag} & t \geq lag \end{Bmatrix} \tag{84}$$

where O_t = outflow hydrograph ordinate at time t; I_t = inflow hydrograph ordinate at time t; and lag = time by which the inflow ordinates are to be lagged.

Figure 28 illustrates the results of application of the lag model. In the figure, the upstream (inflow) hydrograph is the boundary condition. The downstream hydrograph is the computed outflow, with each ordinate equal to an earlier inflow ordinate, but lagged in time.

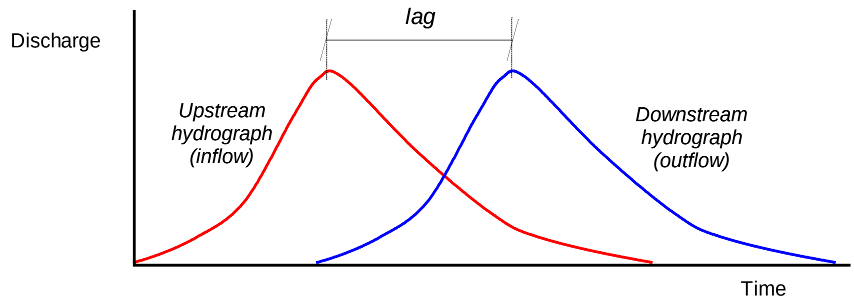

Figure 28. Lag example.

The lag model is a special case of other models, as its results can be duplicated if parameters of those other models are carefully chosen. For example, if X = 0.50 and $K = \Delta t$ in the Muskingum model, the computed outflow hydrograph will equal the inflow hydrograph lagged by K.

Estimating the Lag

If observed flow hydrographs are available, the lag can be estimated from these as the elapsed time between the time of the centroid of areas of the two hydrographs, between the time of hydrograph peaks, or between the time of the midpoints of the rising limbs.

Kinematic Wave Model

Basic Concepts and Equations

The kinematic-wave channel routing model is based upon a finite difference approximation of the continuity equation and a simplification of the momentum equation. This is described in detail in Chapter 6.

Setting Up the Model

Information required to used the kinematic-wave channel routing model is shown in Table 18. This information, for the most part, can be gathered from maps, surveys, and field inspection. Manning's n can be estimated using common procedures.

Table 18. Kinematic wave routing model information requirements.

Flow Component
Shape of the cross section: Is it trapezoidal, rectangular, or circular?
Principle dimension: bottom width of the channel, diameter of the conduit.
Side slope of trapezoidal shape.
Length of the reach.
Slope of the energy grade line.
Manning *n*, roughness coefficient for channel flow.

Muskingum-Cunge Model

Basic Concepts and Equations

Although popular and easy to use, the Muskingum model includes parameters that are not physically based and thus are difficult to estimate. Further, the model is based upon assumptions that often are violated in natural channels. An extension, the Muskingum-Cunge model, overcomes these limitations.

The model is based upon solution of the following form of the continuity equation, (with lateral inflow, q_L, included):

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_L \qquad (85)$$

and the diffusion form of the momentum equation:

$$S_f = S_o - \frac{\partial y}{\partial x} \qquad (86)$$

Combining these and using a linear approximation yields the convective diffusion equation (Miller and Cunge, 1975):

$$\frac{\partial Q}{\partial t} + c\frac{\partial Q}{\partial x} = \mu \frac{\partial^2 Q}{\partial x^2} + cq_L \qquad (87)$$

where c = wave celerity (speed); and μ = hydraulic diffusivity. The wave celerity and the hydraulic diffusivity are expressed as follows:

$$c = \frac{dQ}{dA} \qquad (88)$$

and

$$\mu = \frac{Q}{2BS_o} \qquad (89)$$

where B = top width of the water surface. A finite difference approximation of the partial derivatives, combined with Equation 80, yields:

$$O_t = C_1 I_{t-1} + C_2 I_t + C_3 O_{t-1} + C_4 (q_L \Delta x) \qquad (90)$$

The coefficients are:

$$C_1 = \frac{\frac{\Delta t}{K} + 2X}{\frac{\Delta t}{K} + 2(1 - X)} \qquad (91)$$

$$C_2 = \frac{\frac{\Delta t}{K} - 2X}{\frac{\Delta t}{K} + 2(1 - X)} \qquad (92)$$

$$C_3 = \frac{2(1 - X) - \frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2(1 - X)} \qquad (93)$$

$$C_4 = \frac{2\left(\frac{\Delta t}{K}\right)}{\frac{\Delta t}{K} + 2(1 - X)} \qquad (94)$$

The parameters *K* and *X* are (Cunge, 1969; Ponce, 1978):

$$K = \frac{\Delta x}{c} \qquad (95)$$

$$X = \frac{1}{2}\left(1 - \frac{Q}{BS_o c \Delta x}\right) \qquad (96)$$

But *c*, *Q*, and *B* change over time, so the coefficients C_1, C_2, C_3, and C_4 must also change. The program recomputes them at each time and distance step, Δt and Δx, using the algorithm proposed by Ponce (1986).

Again, the choice of these time and distance steps is critical. The steps are selected to ensure accuracy and stability. The Δt is selected as the minimum of the following: user time step from the control specifications; the travel time through the reach; or 1/20th the time to rise of the peak inflow with the steepest rising limb, rounded to the nearest multiple or divisor of the user time step. Once Δt is chosen, Δx is computed as:

$$\Delta x = c\Delta t \qquad (97)$$

The value is constrained so that:

$$\Delta x < \frac{1}{2}\left(c\Delta t + \frac{Q_o}{BS_o c}\right) \qquad (98)$$

Here Q_o = reference flow, computed from the inflow hydrograph as:

$$Q_o = Q_B + \frac{1}{2}\left(Q_{peak} - Q_B\right) \qquad (99)$$

where Q_B = baseflow; and Q_{peak} = inflow peak.

Setting Up the Model and Estimating Parameters

The Muskingum-Cunge model included in the program can be used in either of two configurations:

- **Standard configuration**. In this configuration, a simple description of a representative channel cross section is provided. Or, one of the alternative shapes shown in Figure 20 is selected. The principle dimensions of the section are specified, along with channel roughness, energy slope, and length. The length and roughness can be estimated from maps, aerial photographs, and field surveys. The energy slope can be estimated as the channel bed slope, in the absence of better information.

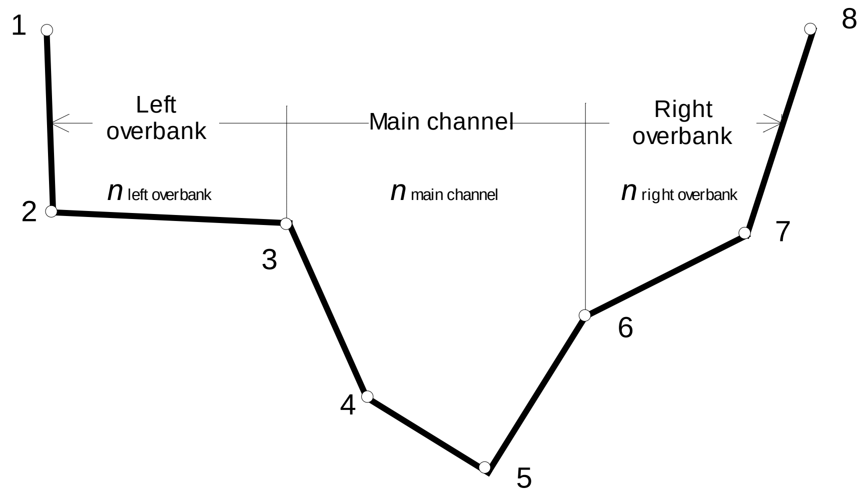

Figure 29. Format for describing channel geometry with 8 points.

- **8-point cross-section configuration**. If one of the standard cross-section shapes will not represent will the channel geometry, the alternative is to use the so-called 8-point cross section configuration. With this, a representative cross section is described for the routing reach, using 8 pairs of *x*, *y* (distance, elevation) values. These values are defined specifically as illustrated in Figure 29. Points labeled 3 and 6 represent the left and right banks of the channel at the representative cross section. Points 4 and 5 are within the channel. Points 1 and 2 represents the left overbank, and points 7 and 8 represent the right overbank.

The reach length, roughness coefficient(s), and energy grade also must be specified. As with the standard configuration, the length and roughness can be estimated from maps, aerial photographs, and field surveys, and the energy slope can be estimated as the channel bed slope, in the absence of better information.

With either configuration of the Muskingum-Cunge model, if the channel properties vary significantly along the routing reach, the reach may be subdivided and modeled as a series of linked subreaches, with the properties of each defined separately.

Applicability and Limitations of Routing Models

Each routing model that is included in the program solves the momentum and continuity equations. However, each omits or simplifies certain terms of those equations to arrive at a solution. To select a routing model, one must consider the routing method's assumptions and reject those models that fail to account for critical characteristics of the flow hydrographs and the channels through which they are routed. These include (but are not limited to) the following:

- **Backwater effects**. Tidal fluctuations, significant tributary inflows, dams, bridges, culverts, and channel constrictions can cause backwater effects. A flood wave that is subjected to the influences of backwater will be attenuated and delayed in time. The kinematic wave and Muskingum models cannot account for the influences of backwater on the flood wave, because these are based on uniform-flow assumptions. Only the modified Puls model can simulate backwater effects, and it can do so in only the case of time-invariant downstream conditions. To model this with the modified Puls model, the effects of the backwater must be determined and included when developing the storage-discharge relationship.

  Practically, none of the routing models that are included in the program will simulate channel flow well if the downstream conditions have a significant impact on upstream flows. The internal structure of the program is such that computations move from upstream watersheds and channels to those downstream. Thus downstream conditions are not yet known when routing computations begin. Only a complete hydraulic system model can accomplish this.

- **Floodplain storage**. If flood flows exceed the channel carrying capacity, water flows into overbank areas. Depending on the characteristics of the overbanks, that overbank flow can be slowed greatly, and often ponding will occur. This can be significant in terms of the translation and attenuation of a flood wave.

  To analyze the transition from main channel to overbank flows, the model must account for varying conveyance between the main channel and the overbank areas. For one-dimensional flow models, this is normally accomplished by calculating the hydraulic properties of the main channel and the overbank areas separately, then combining them to formulate a composite set of hydraulic relationships. This cannot be accomplished with the kinematic-wave and Muskingum models. The Muskingum model parameters are assumed constant. However, as flow spills from the channel, the velocity may change significantly, so K should change. While the Muskingum model can be calibrated to match the peak flow and timing of a specific flood magnitude, the parameters cannot easily be used to model a range of floods that may remain in bank or go out of bank. Similarly, the kinematic wave model assumes constant celerity, an incorrect assumption if flows spill into overbank areas.

  In fact, flood flows through extremely flat and wide flood plains may not be modeled adequately as one-dimensional flow. Velocity of the flow across the floodplain may be just as large as that of flow down the channel. If this occurs, a two-dimensional flow model will better simulate the physical processes. EM 1110-2-1416 (1993) provides more information on this complex subject.

- **Interaction of channel slope and hydrograph characteristics**. As channel slopes lessen, assumptions made to develop many of the models included in the program will be violated: momentum-equation terms that were omitted are more important if the channel slope is small.

  For example, the simplification for the kinematic-wave model is appropriate only if the channel slope exceeds 0.002. The Muskingum-Cunge model can be used to route slow-rising flood waves through reaches with flat slopes. However, it should not be used for rapidly-rising hydrographs in the same channels, because it omits acceleration terms of the momentum equation that are significant in that case. Ponce (1978) established a numerical criterion to judge the likely applicability of various routing models. He suggested that the error due to the use of the kinematic wave model is less than 5 percent if:

  $$\frac{TS_o u_o}{d_o} \geq 171 \qquad (100)$$

  where T = hydrograph duration; u_o = reference mean velocity, and d_o = reference flow depth. (These reference values are average flow conditions of the inflow hydrograph.) He suggested that the error with the Muskingum-Cunge model is less than 5 percent if:

  $$TS_o \left(\frac{g}{d_o}\right)^{1/2} \geq 30 \qquad (101)$$

  where g = acceleration of gravity.

- **Configuration of flow networks**. In a dendritic stream system, if the tributary flows or the main channel flows do not cause significant backwater at the confluence of the two streams, any of the hydraulic or hydrologic routing methods can be applied. However, if significant backwater does occur at confluences, then the models that can account for backwater must be applied. For full networks, where the flow divides and possibly changes direction during the event, none of the simplified models that are included in the program should be used.

- **Occurrence of subcritical and supercritical flow**. During a flood, flow may shift between subcritical and supercritical regimes. If the supercritical flow reaches are short, this shift will not have a noticeable impact on the discharge hydrograph. However, if the supercritical-flow reaches are long, these should be identified and treated as separate routing reaches. If the shifts are frequent and unpredictable, then none of the simplified models are appropriate.

- **Availability of data for calibration**. In general, if observed data are not available, the physically-based routing models will be easier to set up and apply with some confidence. Parameters such as the Muskingum *X* can be estimated, but the estimates can be verified only with observed flows. Thus these empirical models should be avoided if the watershed and channel are ungaged.

Table 19 summarizes the model selection criteria.

Table 19. Guidelines for selecting routing model.

If this is true…	…then consider this model
No observed hydrograph data available for calibration	Kinematic wave; Muskingum-Cunge
Significant backwater will influence discharge hydrograph	Modified Puls
Flood wave will go out of bank, into floodplain	Modified Puls, Muskingum-Cunge with 8-point cross section
Channel slope > 0.002 and $\frac{TS_o u_o}{d_o} \geq 171$	Any
Channel slopes from 0.002 to 0.0004 and $\frac{TS_o u_o}{d_o} \geq 171$	Muskingum-Cunge; modified Puls; Muskingum
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o}\right)^{1/2} \geq 30$	Muskingum-Cunge
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o}\right)^{1/2} < 30$	None

Modeling Confluences as Junctions

Basic Concepts and Equations

Figure 30 illustrates a simple stream confluence, also known as a stream junction. Here two channels intersect, flow is combined, and water travels downstream.

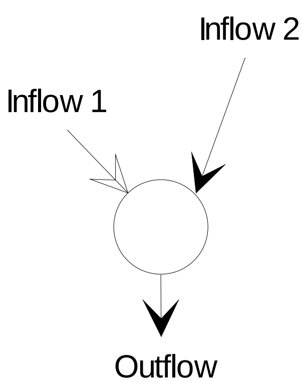

Figure 30. Stream confluence.

Such a confluence can be modeled with the program. To do so, the program uses the following simplification of the continuity equation, which is based upon an assumption that no water is stored at the confluence:

$$\sum_r I_t^r - O_t = 0 \qquad (102)$$

in which I_t^r = the flow in channel r at time t; and O_t = outflow from the confluence in period t. Rearranging yields:

$$O_t = \sum_r I_t^r \quad (103)$$

That is, the downstream flow at time t equals the sum of the upstream flows. This equation is solved repeatedly for all times t in the simulation duration.

Setting Up a Confluence Model

The confluence model included in the program requires the stream system configuration be specified using the graphical user interface. No parameters are required for the model.

Limitations of the Confluence Model

The confluence model is appropriate only if the fundamental assumption of no storage at the confluence is valid. This may not be true if backwater conditions exist at the confluence. In that case, the stream system can be represented well with an unsteady open-channel network model, such as UNET (USACE, 1996).

Modeling Bifurcations as Diversions

Basic Concepts and Equations

Figure 31 illustrates a bifurcation—a split in the flow in a channel. Such a bifurcation, in which the water flows downstream in one of two channels, can be modeled with the program.

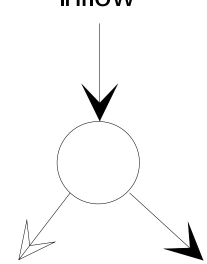

Figure 31. Stream bifurcation.

A bifurcation is modeled in the program with a simple one-dimensional approximation of the continuity equation. In that case:

$$I_t - O_t^{main} - O_t^{secondary} = 0 \quad (104)$$

in which O_t^{main} = average flow passing downstream in the main channel during time interval t; I_t = average channel flow just upstream of the bifurcation during the interval; and $O_t^{secondary}$ = average flow into the secondary channel during the interval. The distinction between main and secondary channels is arbitrary.

Setting Up a Bifurcation Model

The diversion model included in the program requires the secondary channel flow be specified as a function of the inflow upstream of the diversion. That is, Equation 104 must be represented as:

$$O_t^{main} = I_t - f(I_t) \qquad (105)$$

in which $f(I_t)$ = a functional relationship of main channel inflow and secondary channel flow. The relationship can be developed with historical measurements, a physical model constructed in a laboratory, or a mathematical model of the hydraulics of the channel.

Limitations of the Bifurcation Model

The diversion model included in the program is applicable to stream systems in which the necessary relationship between main channel inflow and secondary channel flow can be developed. Often this is impossible, because the secondary channel flow will not be a unique function of main channel inflow. Instead, it will depend upon downstream conditions in one or both channels, and upon the temporal distribution of the inflow hydrographs. In that case, an unsteady-flow network model, such as UNET (USACE, 1997), must be used instead to represent properly the complex hydraulic relationship.

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CHAPTER 9

Calibrating the Models

What is Calibration?

Each model that is included in the program has parameters. The value of each parameter must be specified to use the model for estimating runoff or routing hydrographs. Earlier chapters identified the parameters and described how they could be estimated from various watershed and channel properties. For example, the kinematic-wave direct runoff model described in Chapter 6 has a parameter N that represents overland roughness; this parameter can be estimated from knowledge of watershed land use.

However, as noted in Chapter 2, some of the models that are included have parameters that cannot be estimated by observation or measurement of channel or watershed characteristics. The parameter C_p in the Snyder UH model is an example; this parameter has no direct physical meaning. Likewise, the parameter x in the Muskingum routing model cannot be measured; it is simply a weight that indicates the relative importance of upstream and downstream flow in computing the storage in a channel reach. Equation 85 provides a method for estimating x from channel properties, but this is only approximate and is appropriate for limited cases.

How then can the appropriate values for the parameters be selected? If rainfall and streamflow observations are available, *calibration* is the answer. Calibration uses observed hydrometeorological data in a systematic search for parameters that yield the best fit of the computed results to the observed runoff. This search is often referred to as *optimization*.

Summary of the Calibration Procedure

In HEC-HMS, the systematic search for the best (optimal) parameter values follows the procedure illustrated in Figure 32. This procedure begins with data collection. For rainfall-runoff models, the required data are rainfall and flow time series. For routing models, observations of both inflow to and outflow from the routing reach are required. Table 20 and Table 21 offer some tips for collecting these data.

The next step is to select initial estimates of the parameters. As with any search, the better these initial estimates (the starting point of the search), the quicker the search will yield a solution. Tips for parameter estimation found in previous chapters may be useful here.

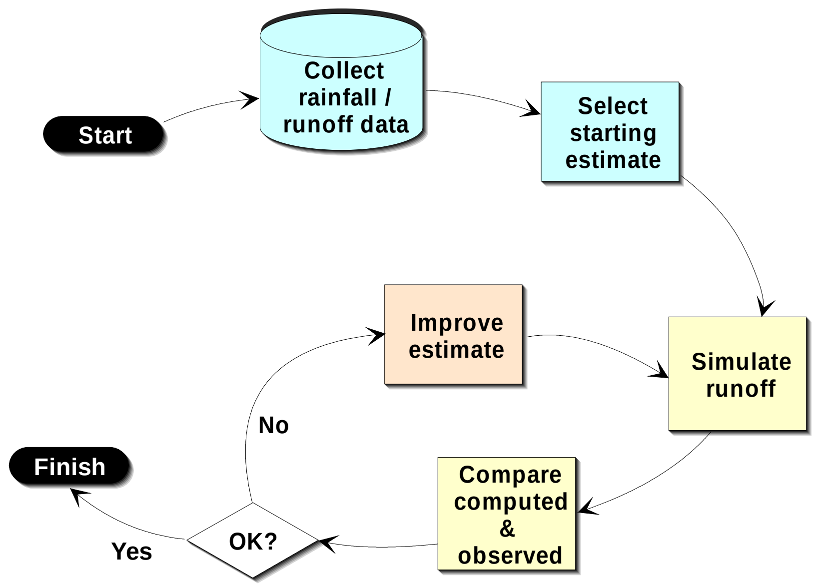


Figure 32. Schematic of calibration procedure.

Given these initial estimates of the parameters, the models included in the program can be used with the observed boundary conditions (rainfall or upstream flow) to compute the output, either the watershed runoff hydrograph or a channel outflow hydrograph.

At this point, the program compares the computed hydrograph to the observed hydrograph. For example, it computes the hydrograph represented with the dashed line in Figure 33 and compares it to the observed hydrograph represented with the solid line. The goal of this comparison is to judge how well the model "fits" the real hydrologic system. Methods of comparison are described later in this chapter.

If the fit is not satisfactory, the program systematically adjusts the parameters and reiterates. The algorithms for adjusting the parameters are described later in this chapter.

When the fit is satisfactory, the program will report the optimal parameter values. The presumption is that these parameter values then can be used for runoff or routing computations that are the goal of the flood runoff analyses.

Table 20. Tips for collecting data for rainfall-runoff model calibration.

Rainfall and runoff observations must be from the same storm. The runoff time series should represent all runoff due to the selected rainfall time series.
The rainfall data must provide adequate spatial coverage of the watershed, as these data will be used with the methods described in Chapter 4 to compute MAP for the storm.
The volume of the runoff hydrograph should approximately equal the volume of the rainfall hyetograph. If the runoff volume is slightly less, water is being lost to infiltration, as expected. But if the runoff volume is significantly less, this may indicate that flow is stored in natural or engineered ponds, or that water is diverted out of the stream. Similarly, if the runoff volume is slightly greater, baseflow is contributing to the total flow, as expected. However, if the runoff volume is much greater, this may indicate that flow is entering the system from other sources, or that the rainfall was not measured accurately.
The duration of the rainfall should exceed the time of concentration of the watershed to ensure that the entire watershed upstream of the concentration point is contributing to the observed runoff.
The size of the storm selected for calibration should approximately equal the size of the storm the calibrated model is intended to analyze. For example, if the goal is to predict runoff from a 1%-chance 24-hour storm of depth 7 inches, data from a storm of duration approximately 24 hours and depth approximately 7 inches should be used for calibration.

Table 21. Tips for collecting data for routing model calibration.

The upstream and downstream hydrograph time series must represent flow for the same period of time.
The volume of the upstream hydrograph should approximately equal the volume of the downstream hydrograph, with minimum lateral inflow. The lumped routing models in HEC-HMS assume that these volumes are equal.
The duration of the downstream hydrograph should be sufficiently long so that the total volume represented equals the volume of the upstream hydrograph.
The size of the event selected for calibration should approximately equal the size of the event the calibrated model is intended to analyze. For example, if the study requires prediction of downstream flows for an event with depths of 20 feet in a channel, historical data for a event of similar depth should be used for calibration.

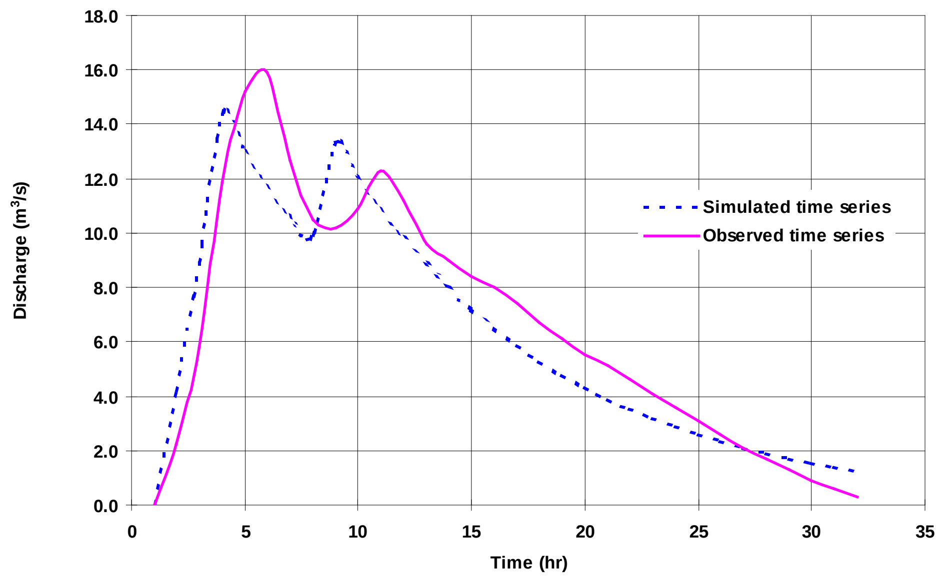


Figure 33. How well does the computed hydrograph "fit"?

Goodness-of-Fit Indices

To compare a computed hydrograph to an observed hydrograph, the program computes an index of the goodness-of-fit. Algorithms included in the program search for the model parameters that yield the best value of an index, also known as *objective function*. Only one of four objective functions included in the program can be used, depending upon the needs of the analysis. The goal of all four calibration schemes is to find reasonable parameters that yield the minimum value of the objective function. The objective function choices (shown in Table 22) are:

- **Sum of absolute errors**. This objective function compares each ordinate of the computed hydrograph with the observed, weighting each equally. The index of comparison, in this case, is the difference in the ordinates. However, as differences may be positive or negative, a simple sum would allow positive and negative differences to offset each other. In hydrologic modeling, both positive and negative differences are undesirable, as overestimates and underestimates as equally undesirable. To reflect this, the function sums the absolute differences. Thus, this function implicitly is a measure of fit of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs. If the value of this function equals zero, the fit is perfect: all computed hydrograph ordinates equal exactly the observed values. Of course, this is seldom the case.

- **Sum of squared residuals.** This is a commonly-used objective function for model calibration. It too compares all ordinates, but uses the squared differences as the measure of fit. Thus a difference of 10 m^3/sec "scores" 100 times worse than a difference of 1 m^3/sec. Squaring the differences also treats overestimates and underestimates as undesirable. This function too is implicitly a measure of the comparison of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs.

Table 22. Objective functions for calibration.

Criterion	Equation
Sum of absolute errors (Stephenson, 1979)	$Z = \sum_{i=1}^{NQ} \left\| q_O(i) - q_S(i) \right\|$
Sum of squared residuals (Diskin and Simon, 1977)	$Z = \sum_{i=1}^{NQ} \left[q_O(i) - q_S(i) \right]^2$
Percent error in peak	$Z = 100 \left\| \frac{q_S(peak) - q_O(peak)}{q_O(peak)} \right\|$
Peak-weighted root mean square error objective function (USACE, 1998)	$Z = \left\{ \frac{1}{NQ} \left[\sum_{i=1}^{NQ} \left(q_O(i) - q_S(i) \right)^2 \left(\frac{q_O(i) + q_O(mean)}{2q_O(mean)} \right) \right] \right\}^{1/2}$

Note:

[1] Z = objective function; NQ = number of computed hydrograph ordinates; $q_O(t)$ = observed flows; $q_S(t)$ = calculated flows, computed with a selected set of model parameters; $q_O(peak)$ = observed peak; $q_O(mean)$ = mean of observed flows; and $q_S(peak)$ = calculated peak

- **Percent error in peak.** This measures only the goodness-of-fit of the computed-hydrograph peak to the observed peak. It quantifies the fit as the absolute value of the difference, expressed as a percentage, thus treating overestimates and underestimates as equally undesirable. It does not reflect errors in volume or peak timing. This objective function is a logical choice if the information needed for designing or planning is limited to peak flow or peak stages. This might be the case for a floodplain management study that seeks to limit development in areas subject to inundation, with flow and stage uniquely related.

- **Peak-weighted root mean square error.** This function is identical to the calibration objective function included in computer program HEC-1 (USACE, 1998). It compares all ordinates, squaring differences, and it weights the squared differences. The weight assigned to each ordinate is proportional to the magnitude of the ordinate. Ordinates greater than the mean of the observed hydrograph are assigned a weight greater than 1.00, and those smaller, a weight less than 1.00. The peak observed ordinate is assigned the maximum weight. The sum of the weighted, squared differences is divided by the number of computed hydrograph ordinates; thus, yielding the mean squared error. Taking the square root yields the root mean squared error. This function is an implicit measure of comparison of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs.

In addition to the numerical measures of fit, the program also provides graphical comparisons that permit visualization of the fit of the model to the observations of the hydrologic system. A comparison of computed hydrographs can be displayed, much like that shown in Figure 33. In addition, the program displays a scatter plot, as shown in Figure 34. This is a plot of the calculated value for each time step against the observed flow for the same step. Inspection of this plot can assist in identifying model bias as a consequence of the parameters selected. The straight line on the plot represents equality of calculated and observed flows: If plotted points fall on the line, this indicates that the model with specified parameters has predicted exactly the observed ordinate. Points plotted above the line represents ordinates that are over-predicted by the model. Points below represent under-predictions. If all of the plotted values fall above the equality line, the model is biased; it always over-predicts. Similarly, if all points fall below the line, the model has consistently under-predicted. If points fall in equal numbers above and below the line, this indicates that the calibrated model is no more likely to over-predict than to under-predict.

The spread of points about the equality line also provides an indication of the fit of the model. If the spread is great, the model does not match well with the observations – random errors in the prediction are large relative to the magnitude of the flows. If the spread is small, the model and parameters fit better.

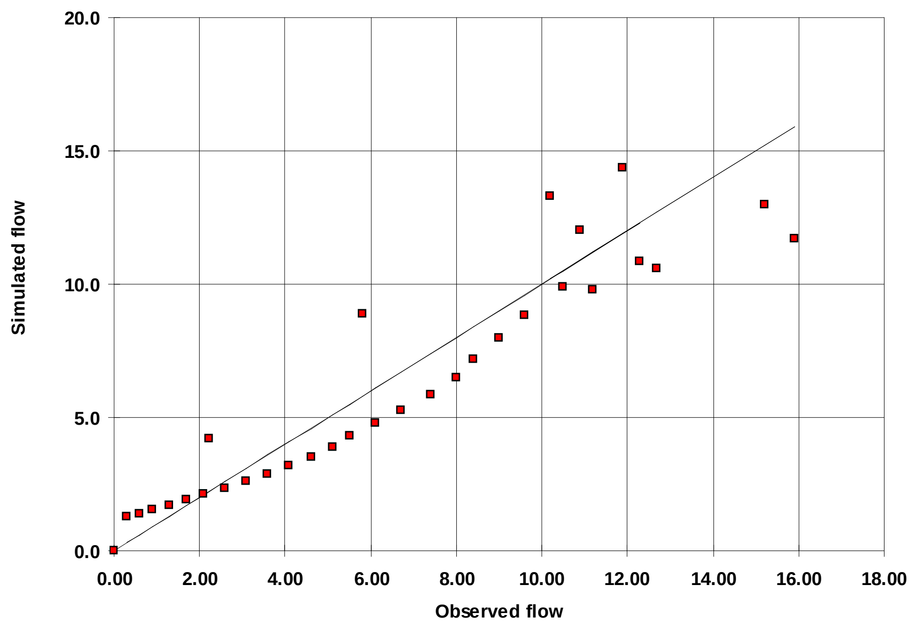


Figure 34. Scatter plot of optimization results.

The program also computes and plots a time series of residuals—differences between computed and observed flows. Figure 35 is an example of this. This plot indicates how prediction errors are distributed throughout the duration of the simulation. Inspection of the plot may help focus attention on parameters that require additional effort for estimation. For example, if the greatest residuals are grouped at the start of a runoff event, the initial loss parameter may have been poorly chosen.

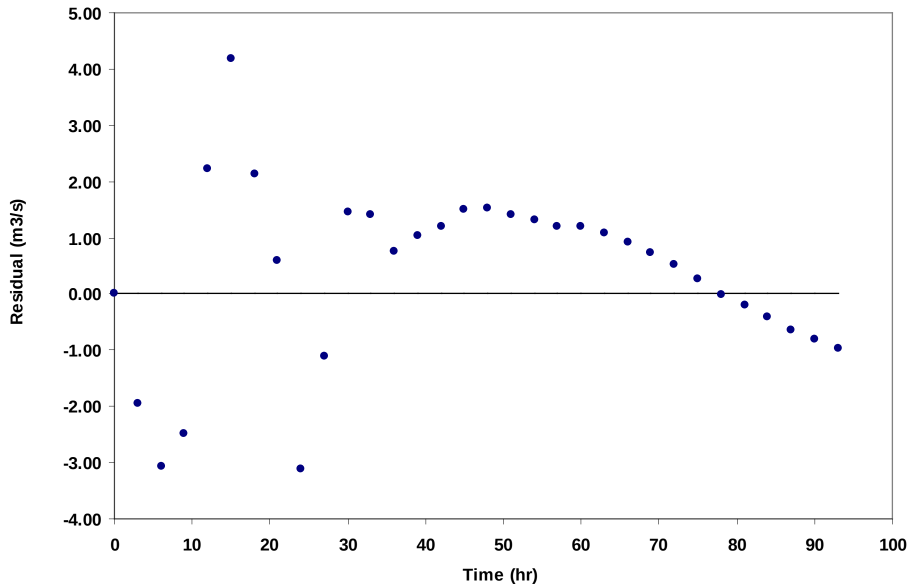


Figure 35. Residual plot of optimization results.

Search Methods

As noted earlier, the goal of calibration is to identify reasonable parameters that yield the best fit of computed to observed hydrograph, as measured by one of the objective functions. This corresponds mathematically to searching for the parameters that *minimize* the value of the objective function.

As shown in Figure 32, the search is a trial-and-error search. Trial parameters are selected, the models are exercised, and the error is computed. If the error is unacceptable, the program changes the trial parameters and reiterates. Decisions about the changes rely on the univariate gradient search algorithm or the Nelder and Mead simplex search algorithm.

Univariate-Gradient Algorithm

The univariate-gradient search algorithm makes successive corrections to the parameter estimate. That is, if x^k represents the parameter estimate with objective function $f(x^k)$ at iteration k, the search defines a new estimate x^{k+1} at iteration k+1 as:

$$x^{k+1} = x^k + \Delta x^k \qquad (106)$$

in which Δx^k = the correction to the parameter. The goal of the search is to select Δx^k so the estimates move toward the parameter that yields the minimum value of the objective function. One correction does not, in general, reach the minimum value, so this equation is applied recursively.

The gradient method, as used in the program, is based upon Newton's method. Newton's method uses the following strategy to define Δx^k:

- The objective function is approximated with the following Taylor series:

$$f(x^{k+1}) = f(x^k) + (x^{k+1} - x^k)\frac{df(x^k)}{dx} + \frac{(x^{k+1} - x^k)^2}{2}\frac{d^2 f(x^k)}{dx^2} \quad (107)$$

  in which $f(x^{k+1})$ = the objective function at iteration k; and $df(\bullet)/dx$ and $d^2f(\bullet)/dx^2$ = the first and second derivatives of the objective function, respectively.

- Ideally, x^{k+1} should be selected so $f(x^{k+1})$ is a minimum. That will be true if the derivative of $f(x^{k+1})$ is zero. To find this, the derivative of Equation 107 is found and set to zero, ignoring the higher order terms. That yields

$$0 = \frac{df(x^k)}{dx} + (x^{k+1} - x^k)\frac{d^2 f(x^k)}{dx^2} \quad (108)$$

  This equation is rearranged and combined with Equation 106, yielding

$$\Delta x^k = -\frac{\frac{df(x^k)}{dx}}{\frac{d^2 f(x^k)}{dx^2}} \quad (109)$$

The program uses a numerical approximation of the derivatives $df(\bullet)/dx$ and $d^2f(\bullet)/dx^2$ at each iteration k. These are computed as follows:

- Two alternative parameters in the neighborhood of x^k are defined as $x^k_1 = 0.99x^k$ and $x^k_2 = 0.98x^k$, and the objective function value is computed for each.

- Differences are computed, yielding $\Delta_1 = f(x^k_1) - f(x^k)$ and $\Delta_2 = f(x^k_2) - f(x^k_1)$

- The derivative $df(\bullet)/dx$ is approximated as Δ_1, and $d^2f(\bullet)/dx^2$ is approximated as $\Delta_2 - \Delta_1$. Strictly speaking, when these approximations are substituted in Equation 109, this yields the correction Δx^k in Newton's method.

As implemented in the program, the correction is modified slightly to incorporate HEC staff experience with calibrating the models included. Specifically, the correction is computed as:

$$\Delta x^k = 0.01\, C x^k \quad (110)$$

in which C is as shown in Table 23.

In addition to this modification, the program tests each value x^{k+1} to determine if, in fact, $f(x^{k+1}) < f(x^k)$. If not, a new trial value, x^{k+2} is defined as

$$x^{k+2} = 0.7\, x^k + 0.3\, x^{k+1} \quad (111)$$

If $f(x^{k+2}) > f(x^k)$, the search ends, as no improvement is indicated.

Table 23. Coefficients for correction in the univariant gradient search.

Δ2 - Δ1	**Δ1**	**C**
> 0	—	$\frac{\Delta_1}{\Delta_2} - 0.5$
< 0	> 0	50
	≤ 0	-33

If more than a single parameter is to be found via calibration, this procedure is applied successively to each parameter, holding all others constant. For example, if Snyder's C_p and t_p are sought, C_p, is adjusted while holding t_p at the initial estimate. Then, the algorithm will adjust t_p, holding C_p at its new, adjusted value. This successive adjustment is repeated four times. Then, the algorithm evaluates the last adjustment for all parameters to identify the parameter for which the adjustment yielded the greatest reduction in the objective function. That parameter is adjusted, using the procedure defined here. This process continues until additional adjustments will not decrease the objective function by at least 1%.

Nelder and Mead Algorithm

The Nelder and Mead algorithm searches for the optimal parameter value without using derivatives of the objective function to guide the search. Instead this algorithm relies on a simpler direct search. In this search, parameter estimates are selected with a strategy that uses knowledge gained in prior iterations to identify good estimates, to reject bad estimates, and to generate better estimates from the pattern established by the good.

The Nelder and Mead search uses a *simplex*—a set of alternative parameter values. For a model with *n* parameters, the simplex has *n*+1 different sets of parameters. For example, if the model has two parameters, a set of three estimates of each of the two parameters is included in the simplex. Geometrically, the *n* model parameters can be visualized as dimensions in space, the simplex as a polyhedron in the *n*-dimensional space, and each set of parameters as one of the *n*+1 vertices of the polyhedron. In the case of the two-parameter model, then, the simplex is a triangle in two-dimensional space, as illustrated in Figure 36.

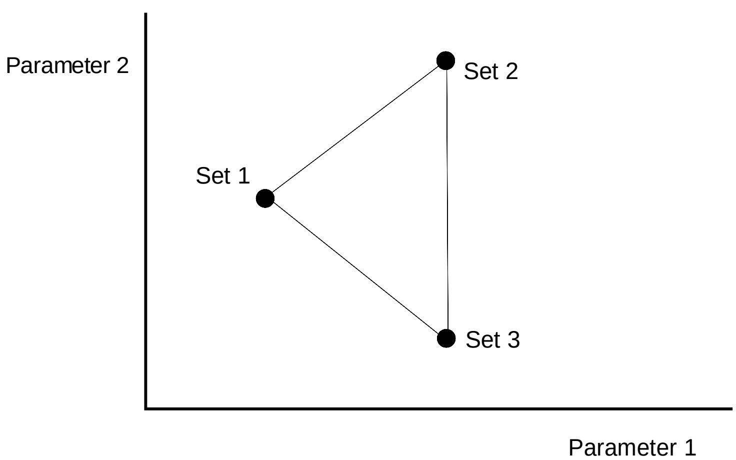
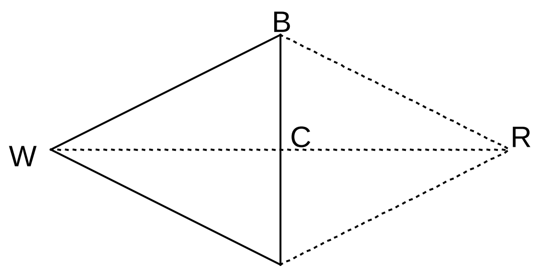


Figure 36. Initial simplex for a 2-parameter model.

The Nelder and Mead algorithm evolves the simplex to find a vertex at which the value of the objective function is a minimum. To do so, it uses the following operations:

- **Comparison**. The first step in the evolution is to find the vertex of the simplex that yields the worst (greatest) value of the objective function and the vertex that yields the best (least) value of the objective function. In Figure 37, these are labeled W and B, respectively.

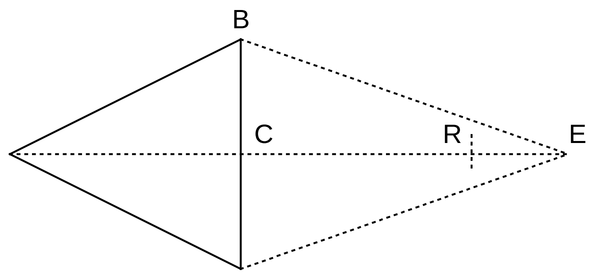
- **Reflection**. The next step is to find the centroid of all vertices, excluding vertex W; this centroid is labeled C in Figure 37. The algorithm then defines a line from W, through the centroid, and reflects a distance WC along the line to define a new vertex R, as illustrated Figure 37.



$x_i\,(reflected) = x_i\,(centroid) + 1.0\ [x_i\,(centroid) - x_i\,(worst)]$

Figure 37. Reflection of a simplex.

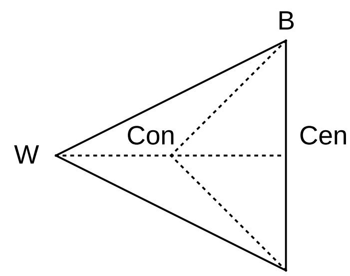
- **Expansion**. If the parameter set represented by vertex R is better than, or as good as, the best vertex, the algorithm further expands the simplex in the same direction, as illustrated in Figure 38. This defines an expanded vertex, labeled E in the figure. If the expanded vertex is better than the best, the worst vertex of the simplex is replaced with the expanded vertex. If the expanded vertex is not better than the best, the worst vertex is replaced with the reflected vertex.



$x_i\,(expanded) = x_i + 2.0\ [x_i\,(reflected) - x_i\,(centroid)]$

Figure 38. Expansion of a simplex.

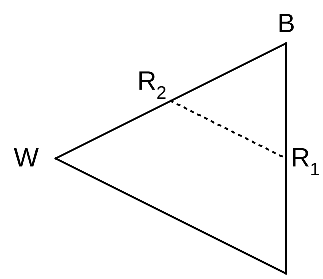
- **Contraction**. If the reflected vertex is worse than the best vertex, but better than some other vertex (excluding the worst), the simplex is contracted by replacing the worst vertex with the reflected vertex. If the reflected vertex is not better than any other, excluding the worst, the simplex is contracted. This is illustrated in Figure 39. To do so, the worst vertex is shifted along the line toward the centroid. If the objective function for this contracted vertex is better, the worst vertex is replaced with this vertex.



x_i *(contracted)* = x_i *(centroid)* - 0.5 [x_i *(centroid)* - x_i *(worst)*]

Figure 39. Contraction of a simplex.

- **Reduction**. If the contracted vertex is not an improvement, the simplex is reduced by moving all vertices toward the best vertex. This yields new vertices R_1 and R_2, as shown in Figure 40.



$x_{i,j}$ *(reduced)* = x_i *(best)* + 0.5 [$x_{i,j}$ - x_i *(best)*]

Figure 40. Reduction of a simplex.

The Nelder and Mead search terminates when either of the following criterion is satisfied:

- $$\sqrt{\sum_{j=1, j \| worst}^{n} \frac{\left(z_j - z_c\right)^2}{n-1}} < tolerance \qquad (112)$$

  in which n = number of parameters; j = index of a vertex, c = index of centroid vertex; and z_j and z_c = objective function values for vertices j and c, respectively.

- The number of iterations reaches 50 times the number of parameters.

The parameters represented by the best vertex when the search terminates are reported as the optimal parameter values.

Constraints on the Search

The mathematical problem of finding the best parameters for a selected model (or models) is what systems engineers refer to as a *constrained optimization* problem. That is, the range of feasible, acceptable parameters (which systems engineers would call the decision variables) is limited. For example, a Muskingum *x* parameter that is less than 0.0 or greater than 0.5 is unacceptable, no matter how good the resulting fit might be. Thus, searching outside that range is not necessary, and any value found outside that range is not be accepted. These limits on x, and others listed in Table 24, are incorporated in the search.

During the search with either the univariant gradient or Nelder and Mead algorithm, the program checks at each iteration to ascertain that the trial values of the parameters are within the feasible range. If they are not, the program increases the trial value to the minimum or decreases it to the maximum before it continues.

In addition to these inviolable constraints, the program will also consider user-specified *soft constraints*. These constraints define desired limits on the parameters. For example, the default range of feasible values of constant loss rate is 0-300 mm/hr. However, for a watershed with dense clay soils, the rate is likely to be less than 15 mm/hr—a much greater value would be suspect. A desired range, 0-15 mm/hr, could be specified as a soft constraint. Then if the search yields a candidate parameter outside the soft constraint range, the objective function is multiplied by a penalty factor. This penalty factor is defined as:

$$Penalty = 2\prod_{i=1}^{n}(\left|x_i - c_i\right| + 1) \qquad (113)$$

in which x_i = estimate of parameter i; c_i = maximum or minimum value for parameter i; and n = number of parameters. This "persuades" the search algorithm to select parameters that are nearer the soft-constraint range. For example, if the search for uniform loss rate leads to a value of 300 mm/hr when a 15 mm/hr soft constraint was specified, the objective function value would be multiplied by 2(│300-15+1│) = 572. Even if the fit was otherwise quite good, this penalty will cause either of the search algorithms to move away from this value and towards one that is nearer 15 mm/hr.

Table 24. Calibration parameter constraints.

Model	Parameter	Minimum	Maximum
Initial and constant-rate loss	Initial loss	0 mm	500 mm
	Constant loss rate	0 mm/hr	300 mm/hr
SCS loss	Initial abstraction	0 mm	500 mm
	Curve number	1	100
Green and Ampt loss	Moisture deficit	0	1
	Hydraulic conductivity	0 mm/mm	250 mm/mm
	Wetting front suction	0 mm	1000 mm
Deficit and constant-rate loss	Initial deficit	0 mm	500 mm
	Maximum deficit	0 mm	500 mm
	Deficit recovery factor	0.1	5
Clark's UH	Time of concentration	0.1 hr	500 hr
	Storage coefficient	0 hr	150 hr
Snyder's UH	Lag	0.1 hr	500 hr
	C_p	0.1	1.0
Kinematic wave	Lag	0.1 min	30000 min
Baseflow	Manning's n	0	1
	Initial baseflow	0 m^3/s	100000 m^3/s
	Recession factor	0.000011	-
Muskingum routing	K	0.1 hr	150 hr
	X	0	0.5
	Number of steps	1	100
Kinematic wave routing	N-value factor	0.01	10
Lag routing	Lag	0 min	30000 min

References

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C H A P T E R 1 0

Modeling Water-Control Facilities

This chapter describes how the program can be used for modeling two types of water-control facilities: diversions and detention ponds or reservoirs.

Diversion Modeling

Basic Concepts and Equations

Figure 41 is a sketch of a diversion. This diversion includes a bypass channel and a control structure (a broad-crested side-channel weir). When the water-surface elevation in the main channel exceeds the elevation of the weir crest, water flows over the weir from the main channel into the by-pass channel. The discharge rate in the diversion channel is controlled by the properties of the control structure. The discharge rate in the main channel downstream of the control is reduced by the volume that flows into the diversion channel.

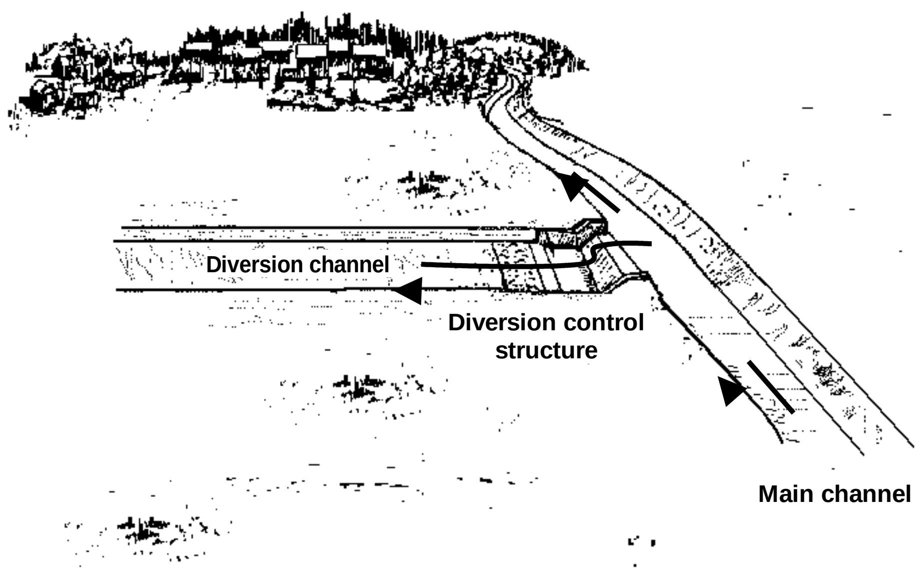

Figure 41. Illustration of a diversion structure.

A diversion is modeled in the same manner as a stream bifurcation by using a simple one-dimensional approximation of the continuity equation. In that case:

$$O_t^{main} = I_t - O_t^{bypass} \qquad (114)$$

in which O_t^{main}= average flow passing downstream in the main channel during time interval t ; I_t = average main channel flow just upstream of the diversion control structure during the interval; and O_t^{bypass}= average flow into the by-pass channel during the interval.

Setting Up a Diversion Model

The diversion model included in the program requires specifying the by-pass channel flow as a function of the main channel flow upstream of the diversion. That is, Equation 114 is represented as:

$$O_t^{main} = I_t - f(I_t) \tag{115}$$

in which $f(I_t)$ = the functional relationship of main channel flow and diversion channel flow. The relationship can be developed with historical measurements, a physical model constructed in a laboratory, or a mathematical model of the hydraulics of the structure. For example, flow over the weir in Figure 41 can be computed with the weir equation:

$$O = CLH^{1.5} \tag{116}$$

in which O = flow rate over the weir; C = dimensional discharge coefficient that depends upon the configuration of the weir; L = effective weir width; H = total energy head on crest. This head is the difference in the weir crest elevation and the water-surface elevation in the channel plus the velocity head, if appropriate. The channel water-surface elevation can be computed with a model of open channel flow, such as HEC-RAS (USACE, 1998a). For more accurate modeling, a two-dimensional flow model can be used to develop the relationship.

Return Flow from a Diversion

The bypass channel may be designed to return flow to the main channel downstream of the protected area, as illustrated in Figure 42. This is modeled in the program by linking a diversion/bifurcation model with channel routing models for the main and bypass channels and a confluence model at the downstream intersection of the bypass and main channels, as shown in Figure 42. Chapter 8 provides more information about modeling a confluence.

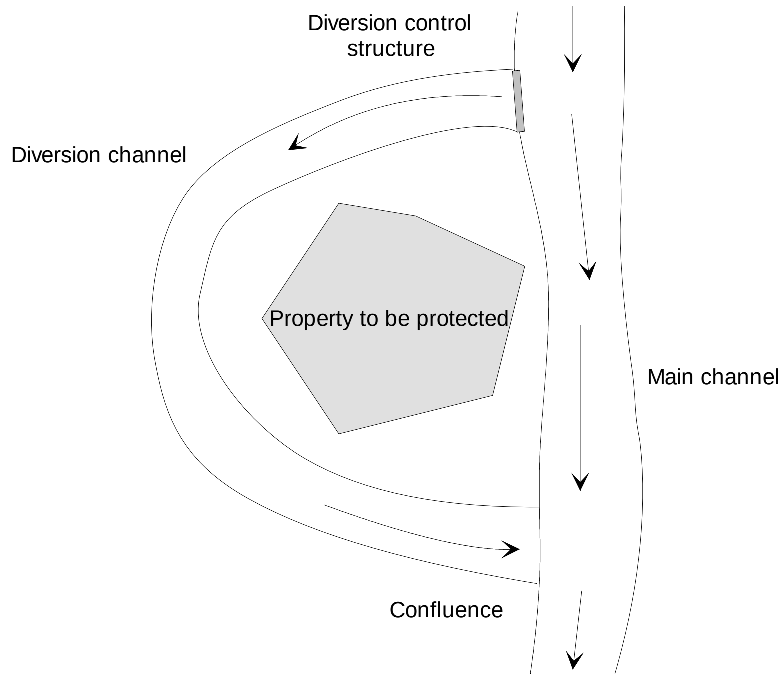

Figure 42. Illustration of a diversion return flow.

Applicability and Limitations of the Diversion Model

The diversion model is applicable to water-control systems in which the necessary relationship between main channel and bypass channel flow can be developed.

If a backwater condition can exist at the control structure (due to downstream conditions such as the confluence of the diversion and the main channel), then an unsteady-flow network model, such as UNET (USACE, 1997), must be used to properly represent the complex hydraulic relationship.

Reservoir and Detention Modeling

A reservoir or detention pond mitigates adverse impacts of excess water by holding that water and releasing it at a rate that will not cause damage downstream. This is illustrated by the hydrographs shown in Figure 43. In this figure, the target flow (release from detention pond) is 113 units. The inflow peak is as shown in the figure; 186 units. To reduce this peak to the target level, storage is provided. Thus the volume of water represented by the shaded area is stored and then released gradually. The total volume of the inflow hydrograph and the volume of the outflow hydrograph (the dotted line) are the same, but the time distribution of the runoff is altered by the storage facility.

Figure 44 is a sketch of a simple detention structure. The structure stores water temporarily and releases it, either through the outlet pipe or over the emergency spillway. The configuration of the outlet works and the embankment in this illustration serves two purposes. It limits the release of water during a flood event, thus protecting downstream property from high flow rates and stages, and it provides a method of emptying the pond after the event so that the pond can store future runoff. (Also, check that this change in timing of the peak does not adversely coincide with flows from other parts of the basin.)

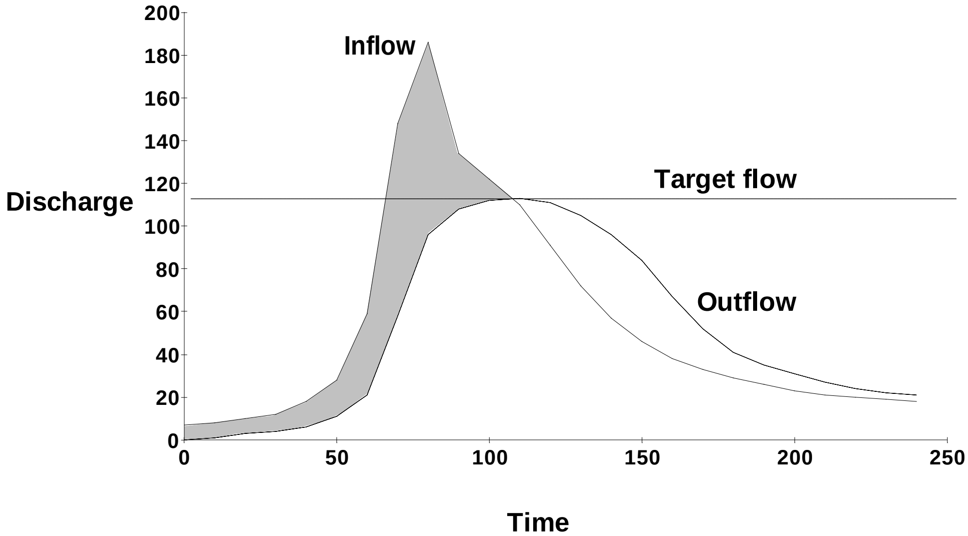

Figure 43.   Illustration of the impact of detention.

The reservoir outlet may consist of a single culvert, as shown in Figure 44. It may also consist of separate conduits of various sizes or several inlets to a chamber or manifold that leads to a single outlet pipe or conduit. The rate of release from the reservoir through the outlet and over the spillway depends on the characteristics of the outlet (in this case, a culvert), the geometric characteristics of the inlet, and the characteristics of the spillway.

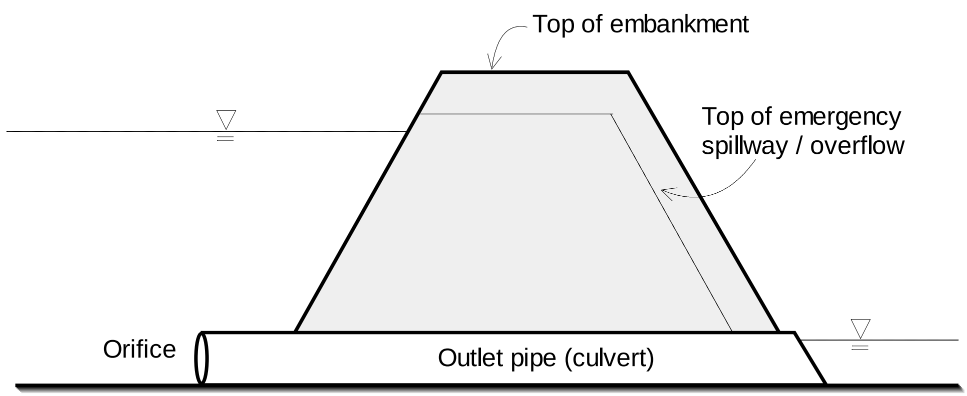

Figure 44.   Simple detention structure.

Basic Concepts and Equations

Outflow from an impoundment that has a horizontal water surface can be computed with the so-called level-pool routing model (also known as Modified Puls routing model). That model discretizes time, breaking the total analysis time into equal intervals of duration Δt. It then solves recursively the following one-dimensional approximation of the continuity equation:

$$I_{avg} - O_{avg} = \frac{\Delta S}{\Delta t} \qquad (117)$$

in which I_{avg} = average inflow during time interval; O_{avg} = average outflow during time interval; ΔS = storage change. With a finite difference approximation, this can be written as:

$$\frac{I_t + I_{t+1}}{2} - \frac{O_t + O_{t+1}}{2} = \frac{S_{t+1} - S_t}{\Delta t} \qquad (118)$$

in which t = index of time interval; I_t and I_{t+1} = the inflow values at the beginning and end of the t^{th} time interval, respectively; O_t and O_{t+1} = the corresponding outflow values; and S_t and S_{t+1} = corresponding storage values. This equation can be rearranged as follows:

$$\left(\frac{2S_{t+1}}{\Delta t} + O_{t+1}\right) = (I_t + I_{t+1}) + \left(\frac{2S_t}{\Delta t} - O_t\right) \qquad (119)$$

All terms on the right-hand side are known. The values of I_t and I_{t+1} are the inflow hydrograph ordinates, perhaps computed with models described earlier in the manual. The values of O_t and S_t are known at the t^{th} time interval. At t = 0, these are the initial conditions, and at each subsequent interval, they are known from calculation in the previous interval. Thus, the quantity ($2S_{t+1} / \Delta t + O_{t+1}$) can be calculated with Equation 119. For an impoundment, storage and outflow are related, and with this storage-outflow relationship, the corresponding values of O_{t+1} and S_{t+1} can be found. The computations can be repeated for successive intervals, yielding values O_{t+1} , O_{t+2}, ... O_{t+n} , the required outflow hydrograph ordinates.

Setting Up a Reservoir Model

To model detention with the program, the storage-outflow relationship for the existing or proposed reservoir must be specified. The storage-outflow relationship (or elevation-storage-outflow or elevation-area-outflow relationship) that is developed and provided will depend on the characteristics of the pond or reservoir, the outlet, and the spillway. Figure 45 illustrates how the relationship in a simple case might be developed. HEC-RAS or other hydraulics software can develop storage-outflow relationships for complex structures.

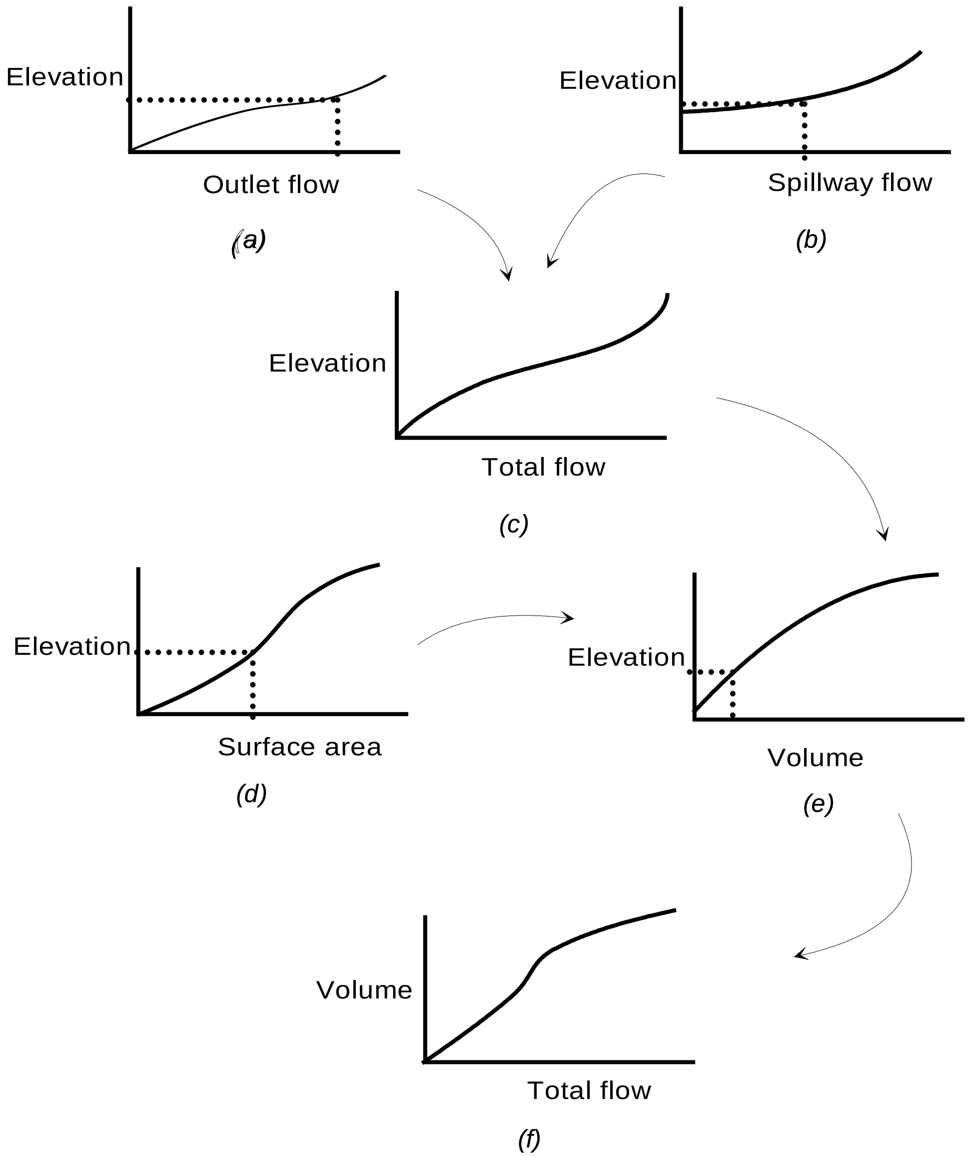

Figure 45. Illustration of the procedure for defining storage-outflow a relationship.

Figure 45*(a)* is the pond outlet-rating function; this relates outflow to the water-surface elevation in the pond. The relationship is determined with appropriate weir, orifice, or pipe formulas, depending on the design of the outlet. In the case of the configuration of Figure 44, the outflow is approximately equal to the inflow until the capacity of the culvert is exceeded. Then water is stored and the outflow depends on the head. When the outlet is fully submerged, the outflow can be computed with the orifice equations:

$$O = KA\sqrt{2gH} \qquad (120)$$

in which *O* = flow rate; *K* = dimensional discharge coefficient that depends upon the configuration of the opening to the culvert; *A*= the cross-sectional area of the culvert, normal to the direction of flow; *H* = total energy head on outlet. This head is the difference in the downstream water-surface elevation and the upstream (pond) water-surface elevation.

Figure 45*(b)* is the spillway rating function. In the simplest case, this function can be developed with the weir equation (Equation 116). For more complex spillways, refer to EM 1110-2-1603 (1965), to publications of the Soil Conservation Service (1985), and to publications of the Bureau of Reclamation (1977) for appropriate rating procedures.

Figure 45*(a)* and *(b)* are combined to yield *(c)*, which represents the total outflow when the reservoir reaches a selected elevation.

Figure 45*(d)* is relationship of reservoir surface area to water-surface elevation; the datum for the elevation here is arbitrary, but consistent throughout the figure. This relationship can be derived from topographic maps or grading plans. Figure 45*(e)* is developed from this with solid-geometry principles.

For an arbitrarily-selected elevation, the storage volume can be found in *(e)*, the total flow found in *(c)*, and the two plotted to yield the desired relationship, as shown in *(f)*. With this relationship, Equation 116 can be solved recursively to find the outflow hydrograph ordinates, given the inflow.

Applicability and Limitations of the Detention Model

The reservoir model that is included in the program is appropriate for simulating performance of any configuration of outlets and pond. However, the model assumes that outflow is inlet-controlled. That is, the outflow is a function of the upstream water-surface elevation. If the configuration of the reservoir and outlet works is such that the outflow is controlled by a backwater effect (perhaps due to a downstream confluence), then the reservoir model should not be used. Instead, an unsteady-flow network model, such as UNET (USACE, 1997) must be used to properly represent the complex relationship of storage, pond outflow, and downstream conditions. Further, if the reservoir is gated, and the gate operation is not uniquely a function of storage, then a reservoir system simulation model, such as HEC-5 (USACE, 1998b), should be used.

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APPENDIX A

CN Tables

The four pages in this section are reproduced from the SCS (now NRCS) report *Urban hydrology for small watersheds*. This report is commonly known as *TR-55*. The tables provide estimates of the curve number (CN) as a function of hydrologic soil group (HSG), cover type, treatment, hydrologic condition, antecedent runoff condition (ARC), and impervious area in the catchment.

TR-55 provides the following guidance for use of these tables:

- Soils are classified into four HSG's (A, B, C, and D) according to their minimum infiltration rate, which is obtained for bare soil after prolonged wetting. Appendix A [of TR-55] defines the four groups and provides a list of most of the soils in the United States and their group classification. The soils in the area of interest may be identified from a soil survey report, which can be obtained from local SCS offices or soil and water conservation district offices.

- There are a number of methods for determining cover type. The most common are field reconnaissance, aerial photographs, and land use maps.

- Treatment is a cover type modifier (used only in Table 2-2b) to describe the management of cultivated agricultural lands. It includes mechanical practices, such as contouring and terracing, and management practices, such as crop rotations and reduced or no tillage.

- Hydrologic condition indicates the effects of cover type and treatment on infiltration and runoff and is generally estimated from density of plant and residue cover on sample areas. Good hydrologic condition indicates that the soil usually has a low runoff potential for that specific hydrologic soil group, cover type and treatment. Some factors to consider in estimating the effect of cover on infiltration and runoff are: (a) canopy or density of lawns, crops, or other vegetative areas; (b) amount of year-round cover; (c) amount of grass or close-seeded legumes in rotations; (d) percent of residue cover; and (e) degree of surface roughness.

- The index of runoff potential before a storm event is the antecedent runoff condition (ARC). The CN for the average ARC at a site is the median value as taken from sample rainfall and runoff data. The curve numbers in table 2-2 are for the average ARC, which is used primarily for design applications.

- The percentage of impervious area and the means of conveying runoff from impervious areas to the drainage systems should be considered in computing CN for urban areas. An impervious area is considered connected if runoff from it flows directly into the drainage systems. It is also considered connected if runoff from it occurs as shallow concentrated shallow flow that runs over a pervious area and then into a drainage system. Runoff from unconnected impervious areas is spread over a pervious area as sheet flow.

SCS TR-55 Table 2-2a – Runoff curve numbers for urban areas[1]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4] . .		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acre	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c					

[1] Average runoff condition, and $I_a = 0.2S$.
[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4, based on the degree of development (imperviousness area percentage) and the CN's for the newly graded pervious areas.

SCS TR-55 Table 2-2b – Runoff curve numbers for cultivated agricultural lands[1]

Cover description			Curve numbers for hydrologic soil group			
Cover type	Treatment[2]	Hydrologic condition[3]	A	B	C	D
Fallow	Bare soil	–	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C & T)	Poor	66	74	80	82
		Good	62	71	78	81
	C & T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	838
	C & T	Poor	61	72	79	82
		Good	59	70	78	81
	C & T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C & T	Poor	63	73	80	83
		Good	51	67	76	80

1 Average runoff condition, and Ia = 0.2S.
2 *Crop residue cover* applies only if residue is on at least 5% of the surface throughout the year.
3 Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes in rotations, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.
Good: Factors impair infiltration and tend to increase runoff.
Poor: Factors encourage average and better than average infiltration and tend to decrease runoff.

SCS TR-55 Table 2-2c – Runoff curve numbers for other agricultural lands[1]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Hydrologic condition	A	B	C	D
Pasture, grassland, or range – continuous forage for graving.[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow – continuous grass, protected from grazing and generally mowed for hay.	–	30	58	71	78
Brush – brush-weed mixture with brush the major element.[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[4]	48	65	73
Woods – grass combination (orchard or tree farm).[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[4]	55	70	77
Farmsteads – buildings, lanes, driveways, and surrounding lots.	–	59	74	82	86

1 Average runoff condition, and $I_a = 0.2S$.

2 *Poor:* <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.

3 *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

4 Actual curve number is less than 30; use CN=30 for runoff computations.

5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

6 *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

SCS TR-55 Table 2-2d – Runoff curve numbers for arid and semiarid rangelands[1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition[2]	A[3]	B	C	D
Herbaceous – mixture of grass, weeds, and low-growing brush, with brush the minor element.	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen – mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper – pinyon, juniper, or both; grass understory.	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub – major plants include saltbrush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

1 Average runoff condition, and $I_a = 0.2S$.

2 *Poor:* <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: >70% ground cover.

3 Curve numbers for group A have been developed only for desert shrub.

APPENDIX B

Soil Moisture Accounting Model Details

This appendix includes additional description of features of the soil moisture accounting model.

Time Interval Selection

Models included in the program rely on the solution of differential equations to estimate watershed runoff. To solve the equations, the models use a finite-difference approximation, as described in Chapter 6. A discrete time interval (Δt) is selected for the approximation, and for this time interval, the program commonly uses the value defined by the user in the *control specifications*. So, for example, if the control specification calls for a 10-minute time interval, the curve number loss model is applied to compute infiltration for successive 10-minute intervals, and the unit hydrograph equations are solved to compute runoff hydrograph ordinates at 10-minute intervals. For these cases, the time interval is user-specified and is constant.

To ensure accuracy of solution of SMA model equations, the program determines and uses internally a *computational* time interval. This interval may be the user-specified interval, or it may be a fraction of that value. In either case, the program reports hydrograph ordinates at the user-specified interval. The time interval is selected as follows:

1. HEC-HMS finds a *minimum time interval* for each storage volume with potential to outflow, using procedures shown in Table 25.

2. HEC-HMS selects the minimum interval from Step 1. If the user-specified value is less, it is used instead.

3. If the time interval calculated in Step 2 is larger than one-quarter of the time required to fill the combined available canopy, surface and soil profile storage, the interval is reduced to that value.

4. If the interval from Step 3 is greater than the precipitation data interval, the computational interval is set equal the precipitation interval.

5. If the interval from Step 4 is greater than 12 hours, the computational interval is reduced to 12 hours. If the interval is less than 1 minute, the interval is increased to 1 minute.

6. If the interval from Step 5 is greater than the remaining time in the user-specified interval, the computational interval is set equal the remaining time.

7. If the interval from Step 6 is less than the remaining time in the user-specified interval, the computational interval is adjusted so it is an even divisor of the remaining time.

8. If the remaining time less the interval found in Step 7 is less than one minute, the computational interval is set equal to the time remaining in the user-specified interval.

The time required to fill or drain storages varies throughout the simulation period, so the program varies the computational time interval throughout the simulation. To do so, it repeats these steps for each user-specified interval. So, for example, during periods in which water is moving rapidly into and out of the storages in the SMA, the program may select and use ten 1-minute computational intervals to account for soil moisture fluxes during a 10-minute user-specified interval. However, as the movement slows, the program may select a longer computational interval—perhaps using two 5-minute computational intervals during the 10-minute user-specified interval.

Table 25. Minimum time step for storages.

Storage	Minimum Time Step
Canopy interception storage	$TimeStep = \frac{1}{4} \frac{CurCanStore}{PotEvapTrans}$ Calculated only if evapotranspiration losses can occur and when the current canopy interception storage at the beginning of the time step exceeds the nominal storage volume.
Surface interception storage	$TimeStep = \frac{1}{4} \frac{CurSurfStore}{PotSoilInfl + PotEvapTrans}$ Calculated when potential evapotranspiration or infiltration losses > 0, and *CurSurfStore* > 0.
Soil profile storage	$TimeStep = \frac{1}{4} \frac{CurSoilStore}{PotSoilPerc + PotEvapTrans}$ Calculated when percolation or evapotranspiration can occur from the soil profile, and *CurSoilStore* > 0.0001 inches.
Groundwater storage	$TimeStep = \frac{1}{4} \frac{CurGw1Store}{PotGw1Perc}$ Calculated when percolation (loss) can occur from a groundwater layer, and the current volume in a groundwater layer > 0
	$TimeStep = \frac{1}{16} RoutGw1Store$ Calculated when the groundwater storage volume divided by the linear reservoir routing coefficient > 0
Precipitation intensity	$TimeStep = \frac{1}{4} \frac{MaxCanStore + MaxSurfStore + MaxSoilStore}{PrecipTimeStep}$ Calculated when *PrecipTimeStep* > 0

Note:

1 TimeStep = time step for storage; CurCanStore = current canopy interception storage; CurSurfStore = current surface interception storage; CurSoilStore = current soil profile storage; MaxCanStore = maximum canopy interception storage; MaxSurfStore = maximum surface interception storage; MaxSoilStore = maximum soil profile storage; CurGw1Store = current groundwater storage; PotEvapTrans = potential ET; PotSoilInf = potential infiltration; PotSoilPerc = potential percolation from soil profile; PotGw1Perc = potential percolation from groundwater layer; RoutGw1Store = coefficient for groundwater linear reservoir model; PrecipTimeStep = time step for specification of precipitation data.

APPENDIX C

Glossary

This glossary is a collection of definitions from throughout the technical reference manual plus definitions of other pertinent terms. Many of the definitions herein are from the electronic glossary available from the USGS internet website at http://water.usgs.gov/wsc/wsc_glo.htm and the USBR website at http://www.usbr.gov/cdams/glossary.html.

Term Definitions

Annual Flood

The maximum peak discharge in a water year.

Annual Flood Series

A list of annual floods.

Antecedent Conditions

Watershed conditions prevailing prior to an event; normally used to characterize basin wetness, e.g., soil moisture. Also referred to as *initial conditions*.

Area-Capacity Curve

A graph showing the relation between the surface area of the water in a reservoir and the corresponding volume.

Attenuation

The reduction in the peak of a hydrograph resulting in a more broad, flat hydrograph.

Backwater

Water backed up or retarded in its course as compared with its normal or natural condition of flow. In stream gaging, a rise in stage produced by a temporary obstruction such as ice or weeds, or by the flooding of the stream below. The difference between the observed stage and that indicated by the stage-discharge relation, is reported as backwater.

Bank

The margins of a channel. Banks are called right or left as viewed facing in the direction of the flow.

Bank Storage

The water absorbed into the banks of a stream channel, when the stages rise above the water table in the bank formations, then returns to the channel as effluent seepage when the stages fall below the water table.

Bankfull Stage

Maximum stage of a stream before it overflows its banks. Bankfull stage is a hydraulic term, whereas flood stage implies damage. See also *flood stage*.

Base Discharge

In the US Geological Survey's annual reports on surface-water supply, the discharge above which peak discharge data are published. The base discharge at each station is selected so that an average of about three peaks a year will be presented. See also *partial-duration flood series*.

Baseflow

The sustained or fair weather flow in a channel due to subsurface runoff. In most streams, baseflow is composed largely of groundwater effluent. Also known as *base runoff*.

Basic Hydrologic Data

Includes inventories of features of land and water that vary spatially (topographic and geologic maps are examples), and records of processes that vary with both place and time. Examples include records of precipitation, streamflow, ground-water, and quality-of-water analyses.

Basic hydrologic information is a broader term that includes surveys of the water resources of particular areas and a study of their physical and related economic processes, interrelations and mechanisms.

Basic-Stage Flood Series

See *partial duration flood series.*

Bifurcation

The point where a stream channel splits into two distinct channels.

Boundary Condition

Known or hypothetical conditions at the boundary of a problem that govern its solution. For example, when solving a routing problem for a given reach, an upstream boundary condition is necessary to determine condition at the downstream boundary.

Calibration

Derivation of a set of model parameter values that produces the “best” fit to observed data.

Canopy Interception

Precipitation that falls on, and is stored in the leaf or trunk of vegetation. The term can refer to either the process or a volume.

Channel

An naturally or artificially created open conduit that may convey water. See also *watercourse*.

Channel Storage

The volume of water at a given time in the channel or over the flood plain of the streams in a drainage basin or river reach. Channel storage can be large during the progress of a flood event.

Computation Duration

The user-defined time window used in hydrologic modeling.

Computation Interval

The user-defined time step used by a hydrologic model for performing mathematical computations. For example, if the computation interval is 15 minutes and the starting time is 1200, hydrograph ordinates will be computed at 1200, 1215, 1230, 1245, and so on.

Concentration Time

See *time of concentration*.

Confluence

The point at which two streams converge.

Continuous Model

A model that tracks the periods between precipitation events, as well as the events themselves. Compare *event-based model.*

Correlation

The process of establishing a relation between a variable and one or more related variables. Correlation is simple if there is only one independent variable and multiple when there is more than one independent variable. For gaging station records, the usual variables are the short-term gaging-station record and one or more long-term gaging-station records.

Dendritic

Channel pattern of streams with tributaries that branch to form a tree-like pattern.

Depression Storage

The volume of water contained in natural depressions in the land surface, such as puddles.

Detention Basin

Storage, such as a small unregulated reservoir, which delays the conveyance of water downstream.

Diffusion

Dissipation of the energy associated with a flood wave; results in the attenuation of the flood wave.

Direct Runoff

The runoff entering stream channels promptly after rainfall or snowmelt. Superposed on base runoff, it forms the bulk of the hydrograph of a flood. The terms base runoff and direct runoff are time classifications of runoff. The terms groundwater runoff and surface runoff are classifications according to source. See also *surface runoff*

Discharge

The volume of water that passes through a given cross-section per unit time; commonly measured in cubic feet per second (cfs) or cubic meters per second (m^3/s). Also referred to as *flow*.

In its simplest concept discharge means outflow; therefore, the use of this term is not restricted as to course or location, and it can be applied to describe the flow of water from a pipe or from a drainage basin. If the discharge occurs in some course or channel, it is correct to speak of the discharge of a canal or of a river. It is also correct to speak of the discharge of a canal or stream into a lake, a stream, or an ocean.

Discharge data in US Geological Survey reports on surface water represent the total fluids measured. Thus, the terms *discharge*, *streamflow*, and *runoff* represent water with sediment and dissolved solids. Of these terms, *discharge* is the most comprehensive. The discharge of drainage basins is distinguished as follows:

- *Yield*. Total water runout or crop; includes runoff plus underflow.
- *Runoff*. That part of water yield that appears in streams.
- *Streamflow*. The actual flow in streams, whether or not subject to regulation, or underflow.

Each of these terms can be reported in total volumes or time rates. The differentiation between runoff as a volume and streamflow as a rate is not accepted. See also *streamflow* and *runoff.*

Discharge Rating Curve

See *stage discharge relation*.

Distribution Graph

A unit hydrograph of direct runoff modified to show the proportions of the volume of runoff that occurs during successive equal units of time.

Diversion

The taking of water from a stream or other body of water into a canal, pipe, or other conduit.

Drainage Area

The drainage area of a stream at a specified location is that area, measured in a horizontal plane, which is enclosed by a drainage divide.

Drainage Divide

The rim of a drainage basin. See also *watershed*.

Duration Curve

See *flow-duration curve* for one type.

ET

See *evapotranspiration*.

Effective Precipitation

That part of the precipitation that produces runoff. Also, a weighted average of current and antecedent precipitation that is "effective" in correlating with runoff.

Evaporation

The process by which water is changed from the liquid or the solid state into the vapor state. In hydrology, evaporation is vaporization and sublimation that takes place at a temperature below the boiling point. In a general sense, *evaporation* is often used interchangeably with *evapotranspiration* or *ET*. See also *total evaporation*.

Evaporation Demand

The maximum potential evaporation generally determined using an evaporation pan. For example, if there is sufficient water in the combination of canopy and surface storage, and in the soil profile, the actual evaporation will equal the evaporation demand. A *soil-water retention curve* describes the relationship between evaporation demand, and actual evaporation when the demand is greater than available water. See also *tension zone*.

Evaporation Pan

An open tank used to contain water for measuring the amount of evaporation. The US National Weather Service class A pan is 4 feet in diameter, 10 inches deep, set up on a timber grillage so that the top rim is about 16 inches from the ground. The water level in the pan during the course of observation is maintained between 2 and 3 inches below the rim.

Evapotranspiration

Water withdrawn from a land area by evaporation from water surfaces and moist soils and plant transpiration.

Event-Based Model

A model that simulates some hydrologic response to a precipitation event. Compare *continuous model*.

Exceedance Probability

Hydrologically, the probability that an event selected at random will exceed a specified magnitude.

Excess Precipitation

The precipitation in excess of infiltration capacity, evaporation, transpiration, and other losses. Also referred to as *effective precipitation*.

Excess Rainfall

The volume of rainfall available for direct runoff. It is equal to the total rainfall minus interception, depression storage, and absorption.

Falling Limb

The portion of a hydrograph where runoff is decreasing.

Field Capacity

The quantity of water which can be permanently retained in the soil in opposition to the downward pull of gravity. Also known as *field-moisture capacity.*

Field-Moisture Deficiency

The quantity of water, which would be required to restore the soil moisture to field-moisture capacity.

Flood

An overflow or inundation that comes from a river or other body of water, and causes or threatens damage. Any relatively high streamflow overtopping the natural or artificial banks in any reach of a stream. A relatively high flow as measured by either gage height or discharge quantity.

Flood Crest

See *flood peak*.

Flood Event

See *flood wave*.

Flood Peak

The highest value of the stage or discharge attained by a flood; thus, peak stage or peak discharge. *Flood crest* has nearly the same meaning, but since it connotes the top of the flood wave, it is properly used only in referring to stage—thus, *crest stage*, but not *crest discharge*.

Floodplain

A strip of relatively flat land bordering a stream, built of sediment carried by the stream and dropped in the slack water beyond the influence of the swiftest current. It is called a living flood plain if it is overflowed in times of highwater; but a fossil flood plain if it is beyond the reach of the highest flood. The lowland that borders a river, usually dry but subject to flooding. That land outside of a stream channel described by the perimeter of the maximum probable flood.

Flood Profile

A graph of elevation of the water surface of a river in flood, plotted as ordinate, against distance, measured in the downstream direction, plotted as abscissa. A flood profile may be drawn to show elevation at a given time, crests during a particular flood, or to show stages of concordant flows.

Flood Routing

The process of progressively determining the timing and shape of a flood wave at successive points along a river.

Flood Stage

The gage height of the lowest bank of the reach in which the gage is situated. The term "lowest bank" is, however, not to be taken to mean an unusually low place or break in the natural bank through which the water inundates an unimportant and small area. The stage at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured. See also *bankfull stage*.

Flood Wave

A distinct rise in stage culminating in a crest and followed by recession to lower stages.

Flood-Frequency Curve

A graph showing the number of times per year on the average, plotted as abscissa, that floods of magnitude, indicated by the ordinate, are equaled or exceeded. Also, a similar graph but with recurrence intervals of floods plotted as abscissa.

Floodway

A part of the floodplain otherwise leveed, reserved for emergency diversion of water during floods. A part of the floodplain which, to facilitate the passage of floodwater, is kept clear of encumbrances.

The channel of a river or stream and those parts of the floodplains adjoining the channel, which are reasonably required to carry and discharge the floodwater or floodflow of any river or stream.

Flow-Duration Curve

A cumulative frequency curve that shows the percentage of time that specified discharges are equaled or exceeded.

Gaging Station

A particular site on a stream, canal, lake, or reservoir where systematic observations of gage height or discharge are obtained. See also *stream-gaging station*.

Ground Water

Water in the ground that is in the zone of saturation, from which wells, springs, and groundwater runoff are supplied.

Groundwater Outflow

That part of the discharge from a drainage basin that occurs through the ground water. The term "underflow" is often used to describe the groundwater outflow that takes place in valley alluvium, instead of the surface channel, and thus is not measured at a gaging station.

Groundwater Runoff

That part of the runoff that has passed into the ground, has become ground water, and has been discharged into a stream channel as spring or seepage water. See also *base runoff* and *direct runoff*.

Hydraulic Radius

The flow area divided by the wetted perimeter. The wetted perimeter does not include the free surface.

Hydrograph

A graph showing stage, flow, velocity, or other property of water with respect to time.

Hydrologic Budget

An accounting of the inflow to, outflow from, and storage in, a hydrologic unit, such as a drainage basin, aquifer, soil zone, lake, reservoir, or irrigation project.

Hydrologic Cycle

The continuous process of water movement between the oceans, atmosphere, and land.

Hydrology

The study of water; generally focuses on the distribution of water and interaction with the land surface and underlying soils and rocks.

Hyetograph

Rainfall intensity versus time; often represented by a bar graph.

Index Precipitation

An index that can be used to adjust for bias in regional precipitation, often quantified as the expected annual precipitation.

Infiltration

The movement of water from the land surface into the soil.

Infiltration Capacity

The maximum rate at which the soil, when in a given condition, can absorb falling rain or melting snow.

Infiltration Index

An average rate of infiltration, in inches per hour, equal to the average rate of rainfall such that the volume of rain fall at greater rates equals the total direct runoff.

Inflection Point

Generally refers the point on a hydrograph separating the falling limb from the recession curve; any point on the hydrograph where the curve changes concavity.

Initial Conditions

The conditions prevailing prior to an event. See also to *antecedent conditions*.

Interception

The capture of precipitation above the ground surface, for example by vegetation or buildings.

Isohyet

Lines of equal rainfall intensity.

Isohyetal Line

A line drawn on a map or chart joining points that receive the same amount of precipitation.

Lag

Variously defined as time from beginning (or center of mass) of rainfall to peak (or center of mass) of runoff.

Lag Time

The time from the center of mass of excess rainfall to the hydrograph peak. Also referred to as *basin lag*.

Loss

The difference between the volume of rainfall and the volume of runoff. Losses include water absorbed by infiltration, water stored in surface depressions, and water intercepted by vegetation.

Mass Curve

A graph of the cumulative values of a hydrologic quantity (such as precipitation or runoff), generally as ordinate, plotted against time or date as abscissa. See also *double-mass curve* and *residual-mass curve*.

Maximum Probable Flood

See *probable maximum flood*.

Meander

The winding of a stream channel.

Model

A physical or mathematical representation of a process that can be used to predict some aspect of the process.

Moisture

Water diffused in the atmosphere or the ground.

Objective Function

A mathematical expression that allows comparison between a calculated result and a specified goal. In the program, the objective function correlates calculated discharge with observed discharge. The value of the objective function is the basis for *calibrating* model *parameters.*

Overland Flow

The flow of rainwater or snowmelt over the land surface toward stream channels. After it enters a stream, it becomes runoff.

Parameter

A variable, in a general model, whose value is adjusted to make the model specific to a given situation. A numerical measure of the properties of the real-world system.

Parameter Estimation

The selection of a parameter value based on the results of analysis and/or engineering judgement. Analysis techniques include calibration, regional analysis, estimating equations, and physically based methods. See also *calibration*.

Partial-Duration Flood Series

A list of all flood peaks that exceed a chosen base stage or discharge, regardless of the number of peaks occurring in a year. Also called floods above a base. See also *basic-stage flood series*.

Peak

The highest elevation reached by a flood wave. Also referred to as the *crest*.

Peak Flow

The point of the hydrograph that has the highest flow.

Peakedness

Describes the rate of rise and fall of a hydrograph.

Percolation

The movement, under hydrostatic pressure, of water through the interstices of a rock or soil.

Precipitation

As used in hydrology, precipitation is the discharge of water, in liquid or solid state, out of the atmosphere, generally upon a land or water surface. It is the common process by which atmospheric water becomes surface or subsurface water. The term *precipitation* is also commonly used to designate the quantity of water that is precipitated. Precipitation includes rainfall, snow, hail, and sleet, and is therefore a more general term than rainfall.

Probable Maximum Flood

The largest flood for which there is any reasonable expectancy in this climatic era.

Probable Maximum Precipitation

The largest precipitation for which there is any reasonable expectancy in this climatic era.

Rain

Liquid precipitation.

Rainfall

The quantity of water that falls as rain only. Not synonymous with *precipitation*.

Rainfall Excess

See *excess rainfall*.

Rating Curve

The relationship between stage and discharge.

Reach

A segment of a stream channel.

Recession Curve

The portion of the hydrograph where runoff is predominantly produced from basin storage (subsurface and small land depressions); it is separated from the falling limb of the hydrograph by an inflection point.

Recurrence Interval

The average interval of time within which the given flood will be equaled or exceeded once. When the recurrence interval is expressed in years, it is the reciprocal of the annual exceedance probability.

Regulation

The artificial manipulation of the flow of a stream.

Reservoir

A pond, lake, or basin, either natural or artificial, for the storage, regulation, and control of water.

Residual-Mass Curve

A graph of the cumulative departures from a given reference such as the arithmetic average, generally as ordinate, plotted against time or date, as abscissa. See also *mass curve*.

Retention Basin

Similar to detention basin but water in storage is permanently obstructed from flowing downstream.

Return Period

See *recurrence interval*.

Rising Limb

Portion of the hydrograph where runoff is increasing.

Runoff

That part of the precipitation that appears in surface streams. It is the same as streamflow unaffected by artificial diversions, storage, or other works of man in or on the stream channels.

Saturation Zone

The portion of the soil profile where available water storage is completely filled. The boundary between the *vadose zone* and the *saturation zone* is called the water table. Note, that under certain periods of infiltration, the uppermost layers of the soil profile can be saturated. See *vadose zone*.

SCS Curve Number

An empirically derived relationship between location, soil-type, land use, antecedent moisture conditions and runoff. A SCS curve number is used in many event-based models to establish the initial soil moisture condition, and the infiltration characteristics.

Snow

A form of precipitation composed of ice crystals.

Soil Moisture Accounting

A modeling process that accounts for continuous fluxes to and from the soil profile. Models can be *event-based* or *continuous*. When using a continuous simulation, a *soil moisture accounting* method is used to account for changes in soil moisture between precipitation events.

Soil Moisture

Water diffused in the soil, the upper part of the zone of aeration from which water is discharged by the transpiration of plants or by soil evaporation. See also *field-moisture capacity* and *field-moisture deficiency*.

Soil Profile

A description of the uppermost layers of the ground down to bedrock. In a hydrologic context, the portion of the ground subject to infiltration, evaporation and percolation fluxes.

Soil Water

See *soil moisture*.

Stage

The height of a water surface in relation to a datum.

Stage-Capacity Curve

A graph showing the relation between the surface elevation of the water in a reservoir usually plotted as ordinate, against the volume below that elevation plotted as abscissa.

Stage-Discharge Curve

A graph showing the relation between the water height, usually plotted as ordinate, and the amount of water flowing in a channel, expressed as volume per unit of time, plotted as abscissa. See also *rating curve*.

Stage-Discharge Relation

The relation expressed by the *stage-discharge curve*.

Stemflow

Rainfall or snowmelt led to the ground down the trunks or stems of plants.

Storage

Water artificially or naturally impounded in surface or underground reservoirs. The term *regulation* refers to the action of this storage in modifying downstream streamflow.

Also, water naturally detained in a drainage basin, such as ground water, channel storage, and depression storage. The term drainage basin storage or simply basin storage is sometimes used to refer collectively to the amount of water in natural storage in a drainage basin.

Storm

A disturbance of the ordinary average conditions of the atmosphere which, unless specifically qualified, may include any or all meteorological disturbances, such as wind, rain, snow, hail, or thunder.

Stream

A general term for a body of flowing water. In hydrology the term is generally applied to the water flowing in a natural channel as distinct from a canal. More generally as in the term stream gaging, it is applied to the water flowing in any channel, natural or artificial.

Stream Gaging

The process and art of measuring the depths, areas, velocities, and rates of flow in natural or artificial channels.

Streamflow

The discharge that occurs in a natural channel. Although the term discharge can be applied to the flow of a canal, the word streamflow uniquely describes the discharge in a surface stream course. The term *streamflow* is more general than runoff, as streamflow may be applied to discharge whether or not it is affected by diversion or regulation.

Stream-Gaging Station

A gaging station where a record of discharge of a stream is obtained. Within the US Geological Survey this term is used only for those gaging stations where a continuous record of discharge is obtained.

Sublimation

The process of transformation directly between a solid and a gas.

Surface Runoff

That part of the runoff that travels over the soil surface to the nearest stream channel. It is also defined as that part of the runoff of a drainage basin that has not passed beneath the surface since precipitation. The term is misused when applied in the sense of direct runoff. See also *runoff*, *overland flow*, *direct runoff*, *groundwater runoff*, and *surface water*.

Surface Water

Water on the surface of the earth.

Tension Zone

In the context of the program, the portion of the soil profile that will lose water only to evapotranspiration. This designation allows modeling water held in the interstices of the soil. See also *soil profile.*

Time of Concentration

The travel time from the hydraulically furthermost point in a watershed to the outlet. Also defined as the time from the end of rainfall excess to the inflection point on the recession curve.

Time of Rise

The time from the start of rainfall excess to the peak of the hydrograph.

Time to Peak

The time from the center of mass of the rainfall excess to the peak of the hydrograph. See also to *lag time*.

Total Evaporation

The sum of water lost from a given land area during any specific time by transpiration from vegetation and building of plant tissue; by evaporation from water surfaces, moist soil, and snow; and by interception. It has been variously termed *evaporation*, *evaporation from land areas*, *evapotranspiration*, *total loss*, *water losses*, and *fly off*.

Transpiration

The quantity of water absorbed and transpired and used directly in the building of plant tissue, in a specified time. It does not include soil evaporation. The process by which water vapor escapes from the living plant, principally the leaves, and enters the atmosphere.

Underflow

The downstream flow of water through the permeable deposits that underlie a stream and that are more or less limited by rocks of low permeability.

Unit Hydrograph

A direct runoff hydrograph produced by one unit of excess precipitation over a specified duration. For example, a one-hour unit hydrograph is the direct runoff from one unit of excess precipitation occurring uniformly over one hour.

Vadose Zone

The portion of the soil profile above the saturation zone.

Water Year

In US Geological Survey reports dealing with surface-water supply, the 12-month period, October 1 through September 30. The water year is designated by the calendar year in which it ends and which includes 9 of the 12 months. Thus, the year ended September 30, 1959, is called the *1959 water year*.

Watercourse

An open conduit either naturally or artificially created which periodically or continuously contains moving water, or which forms a connecting link between two bodies of water. River, creek, run, branch, anabranch, and tributary are some of the terms used to describe natural channels. Natural channels may be single or braided. *Canal* and *floodway* are terms used to describe artificial channels.

Watershed

An area characterized by all direct runoff being conveyed to the same outlet. Similar terms include *basin*, *drainage basin*, *catchment*, and *catch basin*.

A part of the surface of the earth that is occupied by a drainage system, which consists of a surface stream or a body of impounded surface water together with all tributary surface streams and bodies of impounded surface water.

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