HEC-HMS Technical Reference Manual

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1 Introduction

The Hydrologic Modeling System is designed to simulate the precipitation-runoff processes of dendritic watershed systems. It is designed to be applicable in a wide range of geographic areas for solving the widest possible range of problems. This includes large river basin water supply and flood hydrology, and small urban or natural watershed runoff. Hydrographs produced by the program are used directly or in conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation.

1.1 Contents of 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 channel 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, continuous simulation, and interior flooding analysis.

This technical reference manual describes the various methods included in the program, for example, the Green and Ampt loss method. The development of the mathematical algorithm underlying each method is presented. Information and guidance are provided to assist the user in selecting a method for application and estimating parameters. Details on the solution of the algorithm are also provided.

The presentation is aimed at an engineer or scientist who has studied hydrology in a university-level course. Thus, examples of common methods 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.

1.2 Program Overview

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

- Precipitation methods which can describe an observed (historical) precipitation event, a frequency-based hypothetical precipitation event, or an event that represents the upper limit of precipitation possible at a given location.
- Snow melt methods which can partition precipitation into rainfall and snowfall and then account for accumulation and melt of the snowpack. When a snow method is not used, all precipitation is assumed to be rain.
- Evapotranspiration methods which are used in continuous simulation for computing the amount of infiltrated soil water that is removed back to the atmosphere through evaporation and plant use.
- Loss methods which can estimate the amount of precipitation that infiltrates from the land surface into the soil. By implication, the precipitation that does not infiltrate becomes surface runoff.
- Direct runoff methods that describe overland flow, storage, and energy losses as water runs off a watershed and into the stream channels. These are generally called transform methods because the "transform" uninfiltrated precipitation into watershed outflow.

- Baseflow methods that estimate the amount of infiltrated water returning to the channel. Some of the included methods conserve mass through the infiltration process to baseflow; others do not have the same conserving properties.
- Hydrologic routing methods that account for storage and energy flux as water moves through stream channels.
- Models of naturally occurring confluences (junctions) and bifurcations (diversions).
- Models of water-control measures, including diversions and reservoirs.

These methods are similar to those options included in HEC-1. Significant methods not included in HEC-1 include:

- A distributed transform model for use with distributed precipitation data, such as the data available from weather radar.
- A simple one-layer and more complex five-layer soil-moisture-accounting model for use in continuous simulation. They can be used to simulate the long-term response of a watershed to wetting and drying.

The program also includes a number of tools to help process parameter data and computed results, including:

- An automatic calibration tool that can be used to estimate parameter values and initial conditions for most methods, given observations of hydrometeorological conditions.
- An analysis tool to assist in developing frequency curves throughout a watershed on the basis of storms with an associated exceedance probability.

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

1.3 Other Program References

Two references are available in addition to this technical reference 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 Hydrologic Modeling System HEC-HMS Applications Guide (USACE, 2002) describes how to apply the program to completing a hydrology study. A number of different types of studies are described, including typical goals, required information, and needed output data. The steps of performing the study are illustrated with a case study.

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*[1].

1 http://www.hec.usace.army.mil

1.4 Organization of this Manual

Table 1 shows how this manual is organized. Chapters 4-8 and 10 present the equations of the various methods, define the terms of the equations that make up the underlying algorithm, and explain the solution techniques used in the program. In addition, parameters of the methods and techniques for estimating the parameter values are also 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 the program and the technical reference manual
2	Primer on models	Defines terms used throughout the manual and describes basic concepts and components or the program
3	Program components	Describes how hydrologic processes are represented and identifies the methods that are included in the program
4	Precipitation	Identifies each type of precipitation event that may be analyzed, describes the format of the data for each, and presents the precipitation processing algorithms
5	Snow accumulation and melt	Summarizes the processes at work in a snow pack and describes the methods included simulating snow melt
6	Evaporation and transpiration	Describes the processes of evaporation and transpiration and describes how they are modeled in the program
7	Infiltration and runoff volume	Summarizes the methods that are included for estimating runoff volume, given precipitation
8	Surface runoff	Summarizes the methods available for computing runoff hydrographs, given runoff volume

9	Baseflow	Describes the methods for considering sub-surface flow
10	Channel flow	Describes the alternative methods for open channel flow that are available and provides guidance for usage
11	Water-control facilities	Describes the diversion and reservoir elements
12	Automatic parameter estimation	Describes how parameters may be calibrated with historical precipitation and runoff data

1.5 References

US Army Corps of Engineers, USACE (1998) HEC-1 flood hydrograph package user's manual. Hydrologic Engineering Center, Davis, CA.
USACE (2000) Hydrologic Modeling System HEC-HMS User's Manual. Hydrologic Engineering Center, Davis, CA.
USACE (2002) Hydrologic Modeling System HEC-HMS Applications Guide. Hydrologic Engineering Center, Davis, CA.CHAPTER 2

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.

2.1 What is a Model?

Hydrologic engineers are called upon to provide information 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 and/or droughts.
- Regulating floodplain activities.
- Developing plans that use water to enhance environmental function.

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, temperature, and perhaps other meteorologic data. The unknown output is usually runoff. For applications other than watershed runoff estimation, the known input is upstream flow and the unknown output is downstream flow.

2.2 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 San Francisco District maintains and operates the Bay-Delta Model to provide information for answering questions about complex hydraulic flow in the San Francisco Bay and upstream watershed.

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)

Researchers have also 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 HEC-HMS program includes models in a third category—mathematical models. In this manual, that term defines an equation or a set of equations that represent 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.
Mathematical models, including those that are included in the program, can be classified using a number of different criteria. 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.

2.2.1 Event or Continuous

This distinction applies primarily to models of infiltration, surface runoff, and baseflow. An event model simulates a single storm. The duration of the storm may range from a few hours to a few days. The key identifying feature is that the model is only capable of representing watershed response during and immediately after a storm. Event infiltration models do not include redistribution of the wetting front between storms and do not account for drying of the soil through evaporation and transpiration. Often but not always, event infiltration models use a function of cumulative loss to compute infiltration capacity. Unit hydrograph models of surface runoff are all classified as event models because the respond to excess precipitation. Excess precipitation is the actual precipitation minus any infiltrated loss. By definition, excess precipitation can only occur during a storm so these are also event models. Some baseflow models are classified as event because they compute receding flow based on the peak flow rate computed during a storm event. These same methods must include a capability to reset between storms to begin the next recession; nevertheless they remain event models.

A continuous model simulates a longer period, ranging from several days to many years. In order to do so, it must be capable of predicting watershed response both during and between precipitation events. For infiltration models, this requires consideration of the drying processes that occur in the soil between precipitation events. Surface runoff models must be able to account for dry surface conditions with no runoff, wet surface conditions that produce runoff during and after a storm, and the transition between the two states. Baseflow methods become increasing important in continuous simulation because the vast majority of the hydrograph is defined by inter-storm flow characteristics. Most of the models included in HEC-HMS are event models.

2.2.2 Spatially-Averaged or Distributed

This distinction applies mostly to models of infiltration and surface runoff. A distributed model is one in which the spatial (geographic) variations of characteristics and processes are considered explicitly, while in a spatially-averaged model, these spatial variations are averaged or ignored. While not always true, it is often the case that distributed models represent the watershed as a set of grid cells. Calculations are carried out separately for each grid cell. Depending on the complexity of the model, a grid cell may interact with its neighbor cells by exchanging water either above or below the ground surface.
It is important that note that even distributed models perform spatial averaging. As we will see later in detail, most of the models included in HEC-HMS are based on differential equations. These equations are written at the so-called point scale. By point scale we mean that the equation applies over a length Δx that is very small (differential) compared to the size of the watershed. In a spatially-averaged model, the equation is assumed to apply at the scale of a subbasin. Conversely, in a distributed model the equation is typically assumed to apply at the scale of a grid cell. Therefore it is accurate to say that distributed models also perform spatial averaging but generally do so over a much smaller scale than typical spatially-averaged models. HEC-HMS includes primarily spatially-averaged models.

2.2.3 Empirical 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. Many conceptual models are said to be based on "first principles." This usually means that a control volume is established and equations for the conservation of mass and either momentum or energy are written for the control volume. Conservation is a basic principle of physics that cannot be broken. Through the writing of the equations, a model of the process will emerge. In other cases, conceptual models are developed through a mechanistic view instead of first principles. A mechanistic view attempts to represent the dynamics of a process explicitly. For example, water has been observed to move through soil in very predictable ways. A mechanistic view attempts to determine what processes cause water to move as it is observed. If the processes can be described by one or more mathematical equations, then a model can be developed to directly describe the observed behavior.

An empirical model, on the other hand, is built upon observation of input and output, without seeking to represent explicitly the process of conversion. These types of models are sometimes called "black box" models because they convert input to output without any details of the actual physical process involved. A common way to develop empirical models is to collect field data with observations of input and resulting output. The data is analyzed statistically and a mathematical relationship is sought between input and output. Once the relationship is established, output can be predicted for an observed input. For example, observations of inflow to a river reach and resulting flow at a downstream location could be used to develop a relationship for travel time and attenuation of a flood peak through the reach. These empirical models can be very effective so long as they are applied under the same conditions for which they were originally developed. HEC-HMS includes both empirical and conceptual models.

2.2.4 Deterministic or Stochastic

A deterministic model assumes that the input is exactly known. Further, it assumes that the process described by the model is free from random variation. In reality there is always some variation. For example, you could collect a large sample of soil in the field and take it into a laboratory. Next you could divide the large sample into 10 equal small samples and estimate the porosity of each one. You would find a slightly different value for the porosity of each small sample even though the large sample was collected from a single hole dug in the field. This is one example of natural variation in model input. Process variation is somewhat different. Suppose a flood with a specific peak flow enters a section of river. The flood will move down through the reach and the resulting outflow hydrograph will show evidence of translation and attenuation. However, the bed of the river is constantly moving in response to both floods and inter-flood channel flows. The movement of the bed means that the exact same flood with the same specific peak flow could happen again, but the outflow hydrograph would be slightly different. While you might try to describe the reach carefully enough to eliminate the natural variation in the process, it is not practically possible to do so.

Deterministic models essentially ignore variation in input by assuming fixed input. The input may be changed for different scenarios or historical periods, but the input still takes on a single value. Such an assumption may seem too significant for the resulting model to produce meaningful results. However, deterministic models nevertheless are valuable tools because of the difficulty of characterizing watersheds and the hydrologic environment in the first place. Stochastic models, on the other hand, embrace random variation by attempting to explicitly describe it. For example, many floods in a particular river reach may be examined to determine the bed slope during each flood. Given enough floods to examine, you could estimate the mean bed slope, its standard deviation, and perhaps infer a complete probability distribution. Instead of using a single input like deterministic models, stochastic models include the statistics of variation both of the input and process. All models included in HEC-HMS are deterministic.

2.2.5 Measured-Parameter or Fitted-Parameter

This distinction between measured and fitted parameters 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. The Green and Ampt infiltration model is an example of a measured parameter model. It includes hydraulic conductivity and wetting front suction as parameters. Both parameters can be measured directly using appropriate instruments imbedded in the soil during a wetting-drying cycle. Many other parameters used in infiltration models can be reliably estimated if the soil texture is known; texture can be determined by direct visual examination of the soil.

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. The Muskingum routing model is an example of a fitted parameter model. The K parameter can be directly estimated as the travel time of the reach. However, the X parameter is a qualitative estimate of the amount of attenuation in the reach. Low values of X indicate significant attenuation while high values indicate pure translation. The only way to estimate the value of X for a particular reach is to examine the upstream hydrograph and the resulting outflow hydrograph. HEC-HMS includes both measured-parameter models and fitted-parameter models.

2.3 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 same components in common.

2.3.1 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.

2.3.2 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 curve number that is a constituent of the SCS curve number runoff model described in Chapter 5. This parameter, 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 increase. If the number decreases, the runoff volume will decrease.

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.

2.3.3 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.

2.3.4 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. Solving differential equations that involve time always requires knowledge about the state of the system at the beginning of the simulation.

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:

$$Q_t = Q_{t-\Delta t} + (\frac{\Delta Q}{\Delta t})$$

In this equation, $Q_{t-\Delta t}$ 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.

2.4 Models and Computer Programs

Many engineers and scientists will use the term "model" to refer to a variety of different things. An equation can properly be called a model because it does "model" or describe the behavior of a physical system. "Model" can also be used to refer to a primary equation and additional supporting equations or graphs that are used to estimate parameters for the primary equation. A good example of this second use of model is the Snyder unit hydrograph model, described in Chapter 6. An input "model" describes how data required to solve equations is acquired, processed, and prepared for use. Finally, a computer program may also be called a "model" because is solves equations that describe a physical system. For clarity, this manual minimizes the use of the term "model" in an attempt to reduce ambiguity. These terms are used:

2.4.1 Method

As noted above, a mathematical model is the equations that represent the behavior of hydrologic system components. This manual uses the term method in this context. For example, the Muskingum-Cunge channel routing method described in Chapter 8 encapsulates equations for continuity and momentum to form a mathematical model of open-channel flow for routing. All of the details of the equations, initial conditions, state variables, boundary conditions, and technique of solving the equations are contained within the method.

2.4.2 Input

When the equations of a mathematical model are solved with site-specific conditions and parameters, the equations describe the processes and predict 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, input to the program is necessary. The input encapsulates the site-specified conditions and parameters. With HEC-HMS, the information is supplied by completing forms in the graphical user interface. The input may also include time-series data, paired data functions, or grid data from an HEC-DSS database (USACE, 1995).

2.4.3 Program

If the equations of a mathematical model are too numerous or too complex to solve with pencil, paper, and calculator, they can be translated into computer code. Techniques from a branch of mathematics called numerical analysis are used to solve the equations within the constraints of performing calculations with a computer. The result is a computer program. The term model is often applied to a computer program because the particular program only solves one mathematical model. However, HEC-HMS includes a variety of methods for modeling hydrologic components. Thus it does not make sense to call it a model; it is a computer program.

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—generalized so that such fundamental changes are not required. Instead, these programs 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. Also, not all methods can be applied in all circumstances and selections must be made carefully. Criteria to assist in selecting a method for a particular application are listed at the end of each chapter.

2.5 References0

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3 Program Components

This chapter describes how the methods included in the program conceptually represent watershed behavior. It also identifies and categorizes these methods on the basis of the underlying mathematical models.

3.1 Watershed 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. The precipitation may be rainfall or could optionally include snowfall as well. In the simple conceptualization shown, the precipitation can fall on the watershed's vegetation, land surface, and water bodies such as streams and lakes.

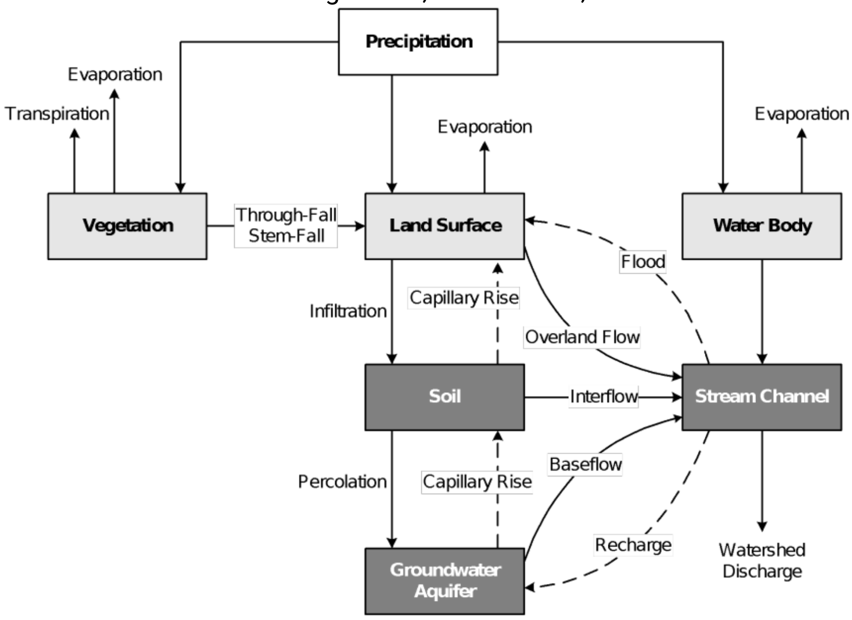


Figure 1.Systems diagram of the runoff process at local scale (after Ward, 1975).

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. The limitation occurs because the meteorologic conditions that result in precipitation often reduce evaporation nearly to zero by decreasing solar radiation and increasing relative humidity. Transpiration from vegetation may also be reduced during a storm because of decreased temperature. Finally, the short time window of a storm event does not allow the evaporation and transpiration processes to make a significant difference on the total water balance. However, evaporation and transpiration are almost always major components of the total water balance over long time windows, often returning a majority of the precipitation back to the atmosphere.
Some precipitation on vegetation falls between the leaves or runs down stems, branches, and trunks to the land surface, where it joins the precipitation that fell directly onto the surface. This is called through-fall and stem-flow, respectively. Once on the land surface, 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 may rise to the surface again by capillary action. When enough water has infiltrated to create saturation zones, it begins to

move vertically, and perhaps horizontally. The saturation point at which this occurs is called the field capacity. The presence of interflow (horizontal subsurface flow) is greatly enhanced by impeding layers such as clay. A saturated zone may develop above the impeding layer and horizontally just above it. The interflow eventually moves into the stream channel. Soil water above the field capacity also moves vertically as gravity drainage in a process called percolation. Percolation water eventually enters the groundwater aquifer beneath the watershed. Water in the aquifer moves slowly, but eventually, some returns to the channels as baseflow. Under some conditions, water in the stream channel may move to the groundwater aquifer as recharge.

Water that does not pond on the land surface or infiltrate into the soil 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.

3.2 Representation of Watershed Processes

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 modifications of watershed land use, it may be appropriate 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 loss method must be selected that is detailed and includes all of the necessary components. The program includes such a method.

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 exceedance probability, a detailed accounting and reporting of the amount of water stored in the upper soil layers is not needed. Instead, only an accurate report the peak, or the volume, or the hydrograph of watershed runoff is required. In this and similar cases, the "view" of the hydrologic process can be simpler. Then, as illustrated in Figure 2, only those components necessary to predict runoff are represented in detail, and the other components are simplified or omitted entirely. For example, in a common application, detailed accounting of movement of water within the soil can be omitted. In this "reductionist" view, the program is configured to include an infiltration method for 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 as direct runoff. It does not include a detailed model of interflow or flow to the groundwater aquifer, instead representing only the combined outflow as baseflow.

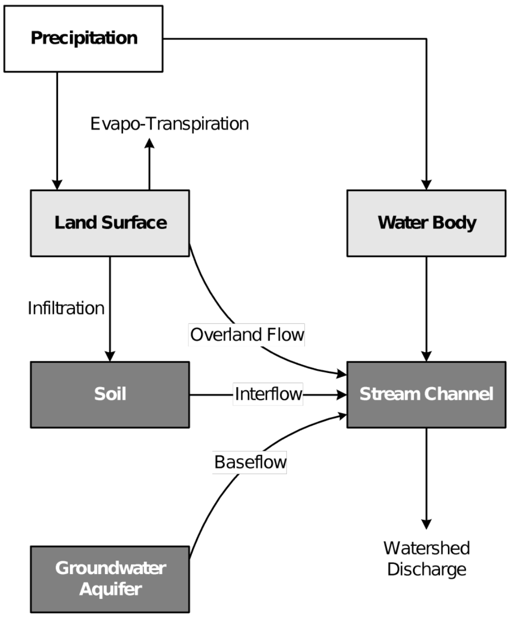


Figure 2.Typical representation of watershed runoff.

3.3 Synopsis of Program Methods

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

- Methods that process precipitation, snow, and potential evapotranspiration meteorologic data.
- Methods that compute infiltration and the resulting runoff volume.
- Methods that represent direct runoff, including overland flow and interflow.
- Methods for describing baseflow.
- Methods for computing channel flow.

The methods that compute infiltration and the resulting runoff volume are listed in Table 3. These methods 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 methods for surface runoff are listed in Table 4. These methods describe what happens as water that has not infiltrated or been stored on the watershed moves over or just beneath the watershed surface. Table 5 lists the models of baseflow. These simulate the slow subsurface drainage of water from the system into the channels.
The choices for modeling channel flow within the program are listed in Table 6. These so-called hydrologic routing models simulate one-dimensional open channel flow. An exception is the kinematic wave method which is a simplified hydraulic routing model.
Table 3.Loss methods for computing infiltration.

Model	**Categorization**
Deficit and constant	continuous, spatially averaged, conceptual, measured parameter

Exponential	event, spatially averaged, empirical, fitted parameter
Green and Ampt	event, spatially averaged, conceptual, measured parameter
Gridded deficit and constant	continuous, distributed, conceptual, measured parameter
Gridded SCS curve number	event, distributed, empirical, fitted parameter
Gridded SMA	continuous, distributed, empirical, fitted parameter
Initial and constant	event, spatially averaged, conceptual, fitted and measured parameter
SCS curve number	event, spatially averaged, empirical, fitted parameter
Smith Parlange	event, spatially averaged, conceptual, measured parameter
Soil moisture accounting (SMA)	continuous, spatially averaged, conceptual, fitted and measured parameter

Table 4. Transform methods for computing surface runoff.

Model	**Categorization**
User-specified unit hydrograph (UH)	event, spatially averaged, empirical, fitted parameter
User-specified s-graph	event, spatially averaged, empirical, fitted parameter
Clark's UH	event, spatially averaged, empirical, fitted parameter
Snyder's UH	event, spatially averaged, empirical, fitted and measured parameter
SCS UH	event, spatially averaged, empirical, fitted parameter
ModClark	event, distributed, empirical, fitted parameter

Kinematic wave	continuous, spatially averaged, conceptual, measured parameter

Table 5.Baseflow methods for computing subsurface flow.

Model	**Categorization**
Bounded recession	event, spatially averaged, empirical, fitted parameter
Constant monthly	continuous, spatially averaged, empirical, fitted parameter
Exponential recession	event, spatially averaged, empirical, fitted parameter
Linear reservoir	continuous, spatially averaged, empirical, fitted parameter
Nonlinear Boussinesq	event, spatially averaged, conceptual, measured parameter

Table 6.Routing methods for computing open channel flow..

Model	**Categorization**
Kinematic wave	continuous, spatially averaged, conceptual, measured parameter
Lag	continuous, spatially averaged, empirical, fitted parameter
Modified Puls	continuous, spatially averaged, empirical, fitted parameter
Muskingum	event, spatially averaged, empirical, fitted parameter
Muskingum-Cunge	continuous, spatially averaged, quasi-conceptual, measured parameter
Straddle-stagger	continuous, spatially averaged, empirical, fitted parameter
Confluence	continuous, conceptual, measured parameter

Bifurcation	continuous, conceptual, measured parameter
Reservoir	Continuous, empirical or conceptual, measured parameter

3.4 Program Setup and Application

The program has been designed to be as flexible as possible in how the hydrologic system is defined. For example, you could choose to combine the deficit and constant loss method with the Clark transform method. The program will do the work of connecting the infiltration and excess precipitation from the loss method to the transform method for computing surface runoff. Most methods can be successfully combined with any other method to compute watershed discharge. However, the applicability of any particular method depends on the characteristics of the watershed and some methods may not be appropriate for some hydrologic-engineering studies.
The program has also been designed to be flexible in the order in which setup is performed. However, some components must exist before others can be created. For example, infiltration calculations cannot be performed until precipitation data has been defined. To analyze a hydrologic system, the following steps work best:

1. Create a new project.
2. Enter shared component data.
3. Define the physical characteristics of the watershed by creating and editing a basin model.
4. Describe the meteorology by creating a meteorologic model.
5. Enter simulation time windows by creating control specifications.
6. Create a simulation by combining a basin model, meteorologic model, and control specifications and view results.
7. Create or modify data.
8. Make additional simulations and compare results.

Create a New Project
Create a new project by selecting the **File New…** menu command. After you press the button a window will open where you can name, choose a location on your computer or a network computer to save the new project, and enter a description for the new project. If the description is long, you can press the button to the right of the description field to open an editor. You should also select the default unit system; you can always change the unit system for any component after it is created but the default provides convenience. Press the **Create** button when you are satisfied with the name, location, and description. You cannot press the **Create** button if no name or location is specified for the new project. If you change your mind and do not want to create a new meteorologic model, press the **Cancel** button or the **X** button in the upper right of the *Create a New Project* window.
Enter Shared Project Data
Shared data includes time-series data, paired data, and grid data. Shared data is often required by basin and meteorologic models. For example, a reach element using the Modified-Puls routing method requires a storage-discharge relationship for the program to calculate flow through the reach. **Error! Reference source not found.** contains a complete list of shared data types used by the program.
Open a component manager to add shared data to a project. Go to the **Components** menu and select **Time-Series Data Manager**, **Paired Data Manager**, or **Grid Data Manager** command. Each one of these component

managers contains a menu for selecting the type of data to create or manage. The **Paired Data Manager** with the **Storage-Discharge** data type selected is shown in Figure 3. Once the data type is selected, you can use the buttons on the right side of the component manager to add a New, Copy, Rename, and Delete a data type. In the case or time-series data, the manager contains two extra buttons to add or delete time windows. A time window is needed for entering or viewing time-series data.

Table 7.Different kinds of shared component data that may be required.

Time-Series Data	**Paired Data**	**Grid Data**
Precipitation	Storage-discharge	Precipitation
Discharge	Elevation-storage	Temperature
Stage	Elevation-area	Solar radiation
Temperature	Elevation-discharge	Crop coefficient
Solar radiation	Inflow-diversion	Storage capacity
Windspeed	Diameter-percentage	Percolation rate
Crop coefficient	Cross sections	Storage coefficients
Snow water equivalent	Unit hydrograph curves	Moisture deficit
Sediment Load	Percentage curves	Impervious area
Concentration	ATI-meltrate functions	SCS curve number
	ATI-coldrate functions	Elevation
	Groundmelt patterns	Cold content
	Meltrate patterns	Cold content ATI
		Meltrate ATI
		Liquid water content

		Snow water equivalent

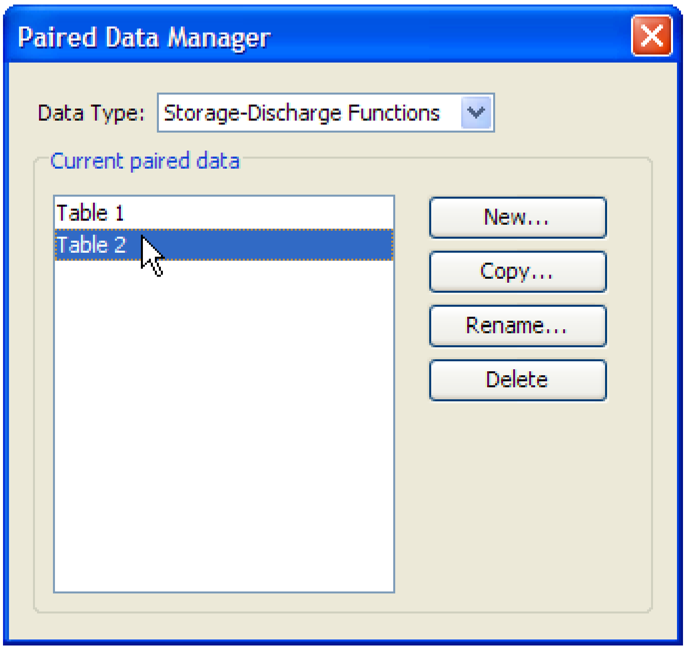


Figure 3.The paired data manager set to work with storage-discharge functions.

Describe the Physical Watershed

The physical watershed is represented in the basin model. Hydrologic elements are added and connected to one another to model the real-world flow of water in a natural watershed. A description of each hydrologic element is given in Table 8.

The basin model manager can be used to add a new basin model to the project. Open the basin model manager by selecting the **Components Basin Model Manager** command. The basin model manager can be used to copy, rename, or delete an existing basin model.

Once a basin model is created, hydrologic elements can be added to the basin map. Select the basin model in the *Watershed Explorer* to open the basin map in the *Desktop*. If background map layers are available, add them to the basin model before adding hydrologic elements. Add a hydrologic element by selecting one of the tools from the toolbar, and clicking the left mouse button on the desired location in the basin map. Connect a hydrologic element to a downstream element by placing the pointer tool over the upstream element icon and clicking the right mouse button to access the **Connect Downstream** menu item.

Most hydrologic elements require parameter data so that the program can model the hydrologic processes represented by the element. In the case of the subbasin element, many mathematical models are available for determining precipitation losses (Table 3), transforming excess precipitation to stream flow at the subbasin outlet (Table 4), and adding baseflow (Table 5). Models are also available for computing open channel flow (Table 6). In this document the different mathematical models will be referred to as methods.

Parameter data is entered in the *Component Editor*. Select a hydrologic element in the basin map or *Watershed Explorer* to open the correct *Component Editor*. An example is shown in . Global parameter editors can also be used to enter or view parameter data for many hydrologic elements in one table. Global parameter editors are opened by selecting the **Parameters** menu. An example of a global editor is shown in Figure 5.

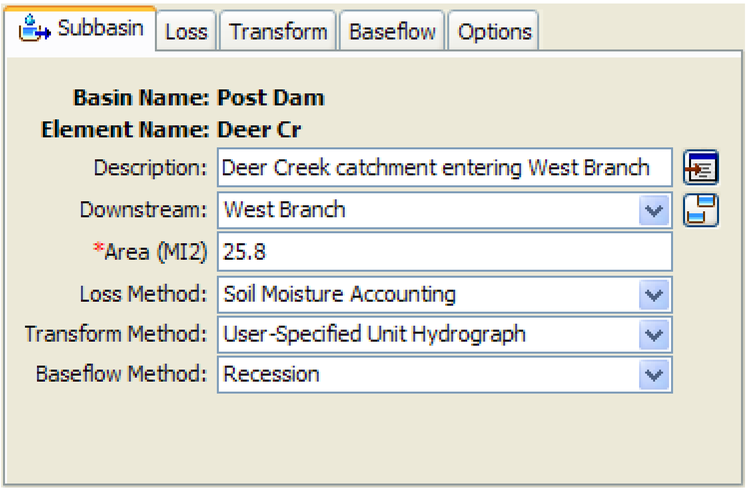


Figure 4.Subbasin component editor including data for loss, transform, and baseflow methods. Area is required.

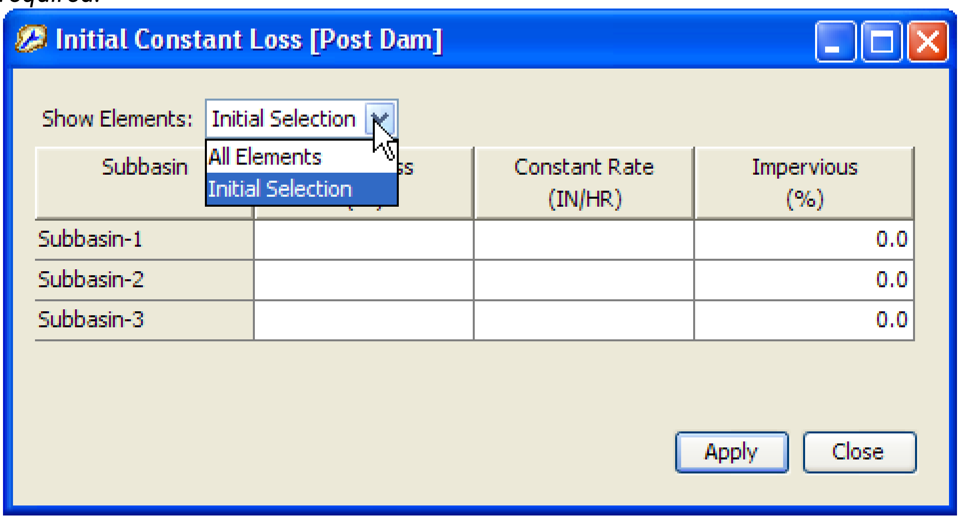


Figure 5.Global editor for the initial and constant loss method.

Table 8.Different kinds of hydrologic elements in a basin model.

Hydrologic Element	**Description**
Subbasin 	The subbasin is used to represent the physical watershed. Given precipitation, outflow from the subbasin element is calculated by subtracting precipitation losses, calculating surface runoff, and adding baseflow.
Reach 	The reach is used to convey streamflow in the basin model. Inflow to the reach can come from one or many upstream elements. Outflow from the reach is calculated by accounting for translation and attenuation. Channel losses can optionally be included in the routing.

Junction 	The junction is used to combine streamflow from elements located upstream of the junction. Inflow to the junction can come from one or many upstream elements. Outflow is calculated by summing all inflows.
Source 	The source element is used to introduce flow into the basin model. The source element has no inflow. Outflow from the source element is defined by the user.
Sink 	The sink is used to represent the outlet of the physical watershed. Inflow to the sink can come from one or many upstream elements. There is no outflow from the sink.
Reservoir 	The reservoir is used to model the detention and attenuation of a hydrograph caused by a reservoir or detention pond. Inflow to the reservoir element can come from one or many upstream elements. Outflow from the reservoir can be calculated using one of three routing methods.
Diversion 	The diversion is used for modeling streamflow leaving the main channel. Inflow to the diversion can come from one or many upstream elements. Outflow from the diversion element consists of diverted flow and non-diverted flow. Diverted flow is calculated using input from the user. Both diverted and non-diverted flows can be connected to hydrologic elements downstream of the diversion element.

Describe the Meteorology
The meteorologic model calculates the precipitation input required by a subbasin element. The meteorologic model can utilize both point and gridded precipitation and has the capability to model frozen and liquid precipitation along with evapotranspiration. The snowmelt methods model the accumulation and melt of the snow pack. The evapotranspiration methods include the constant monthly method and the new Priestly Taylor and gridded Priestly Taylor methods. An evapotranspiration method is only required when simulating the continuous or long term hydrologic response in a watershed. A brief description of the methods available for calculating basin average precipitation or grid cell precipitation is included in Table 9.
Use the meteorologic model manager to add a new meteorologic model to the project. Go to the **Components** menu and select the correct option from the menu list. The meteorologic model manager can also be used to copy, rename, and delete an existing meteorologic model.
Table 9.Precipitation methods available for describing meteorology.

Precipitation Methods	**Description**
Frequency Storm	Used to develop a precipitation event where depths for various durations within the storm have a consistent exceedance probability.

Gage Weights	User specified weights applied to precipitation gages.
Gridded Precipitation	Allows the use of gridded precipitation products, such as NEXRAD radar.
Inverse Distance	Calculates subbasin average precipitation by applying an inverse distance squared weighting with gages.
SCS Storm	Applies a user specified SCS time distribution to a 24-hour total storm depth.
Specified Hyetograph	Applies a user defined hyetograph to a specified subbasin element.
Standard Project Storm	Uses a time distribution to an index precipitation depth.

Enter Simulation Time Windows
A simulation time window sets the time span and time interval of a simulation run. A simulation time window is created by adding a control specifications to the project. This can be done using the control specifications manager. Go to the **Components** menu and select the correct option from the menu list. Besides creating a new simulation time window, the control specifications manager can be used to copy, rename, and delete an existing window.
Once a new control specifications has been added to the project, use the mouse pointer and select it in the *Watershed Explorer*. This will open the *Component Editor* for the control specifications as shown in Figure 6. Information that must be defined includes a starting date and time, ending date and time, and computation time step.

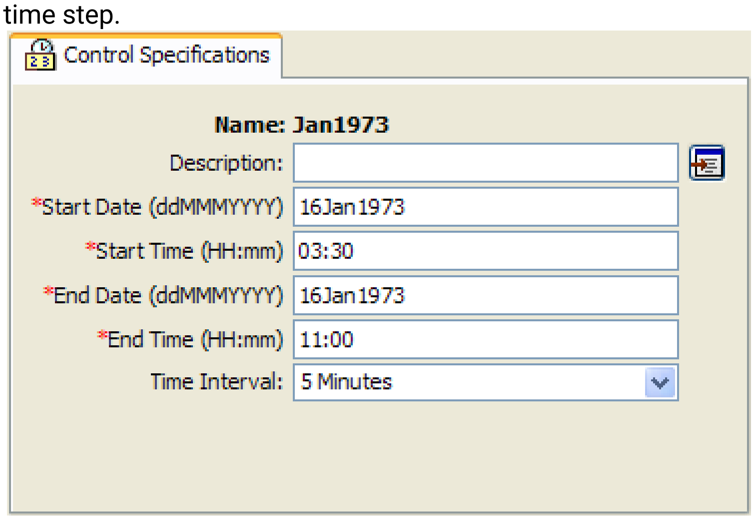


Figure 6.Control specifications Component Editor.
Simulate and View Results
A simulation run calculates the precipitation-runoff response in the basin model given input from the meteorologic model. The control specifications define the time period and time interval. All three

components are required for a simulation run to compute.
Create a new simulation run by selecting the **Compute Create Simulation Run** menu option. A wizard will open to step you through the process of creating a simulation run. First, enter a name for the simulation. Then, choose a basin model, meteorologic model, and control specifications. After the simulation run has been created, select the run. Go to the **Compute Select Run** menu option. When the mouse moves on top of **Select Run** a list of available runs will open. Choose the correct simulation. To compute the simulation, reselect the **Compute** menu and choose the **Compute Run** option at the bottom of the menu.
Results can be accessed from the basin map and the *Watershed Explorer*, "Results" tab. Results are available as long as a simulation run has been successfully computed and no edits have been made after the compute to any component used by the simulation run. For example, if the time of concentration parameter was changed for a subbasin element after the simulation run was computed, then results are no longer available for any hydrologic element in the basin model. The simulation run must be recomputed for results to become available.
The simulation must be selected (from the **Compute** menu or *Watershed Explorer*) before results can be accessed from the basin map. After the simulation run is selected, select the hydrologic element where you want to view results. While the mouse is located on top of the element icon, click the right mouse button. In the menu that opens, select the **View Results** option. Three result types are available: **Graph**, **Summary Table**, and **Time-Series Table** (Figure 12). These results can also be accessed through the toolbar and the **Results** menu. A hydrologic element must be selected before the toolbar buttons and options from the Results menu are active. A global summary table is available from the toolbar and **Results** menu. The global summary table contains peak flows and time of peak flows for each hydrologic element in the basin model.
Results can also be viewed from the *Watershed Explorer*, "Results" tab. Select the simulation run and the *Watershed Explorer* will expand to show all hydrologic elements in the basin model. If you select one of the hydrologic elements, the *Watershed Explorer* expands again to show all result types as shown in Figure 13. For a subbasin element, you might see outflow, incremental precipitation, excess precipitation, precipitation losses, direct runoff, and baseflow as the output results. Select one of these results to open a preview graph. Multiple results can be selected and viewed by holding down the Control or Shift buttons. Results from multiple hydrologic elements can be viewed together. Also, results from different simulation runs can be selected and viewed. Once output types are selected in the *Watershed Explorer*, a larger graph or time-series table can be opened in the *Desktop* by selecting the **Graph** and **Time-Series** buttons on the toolbar.

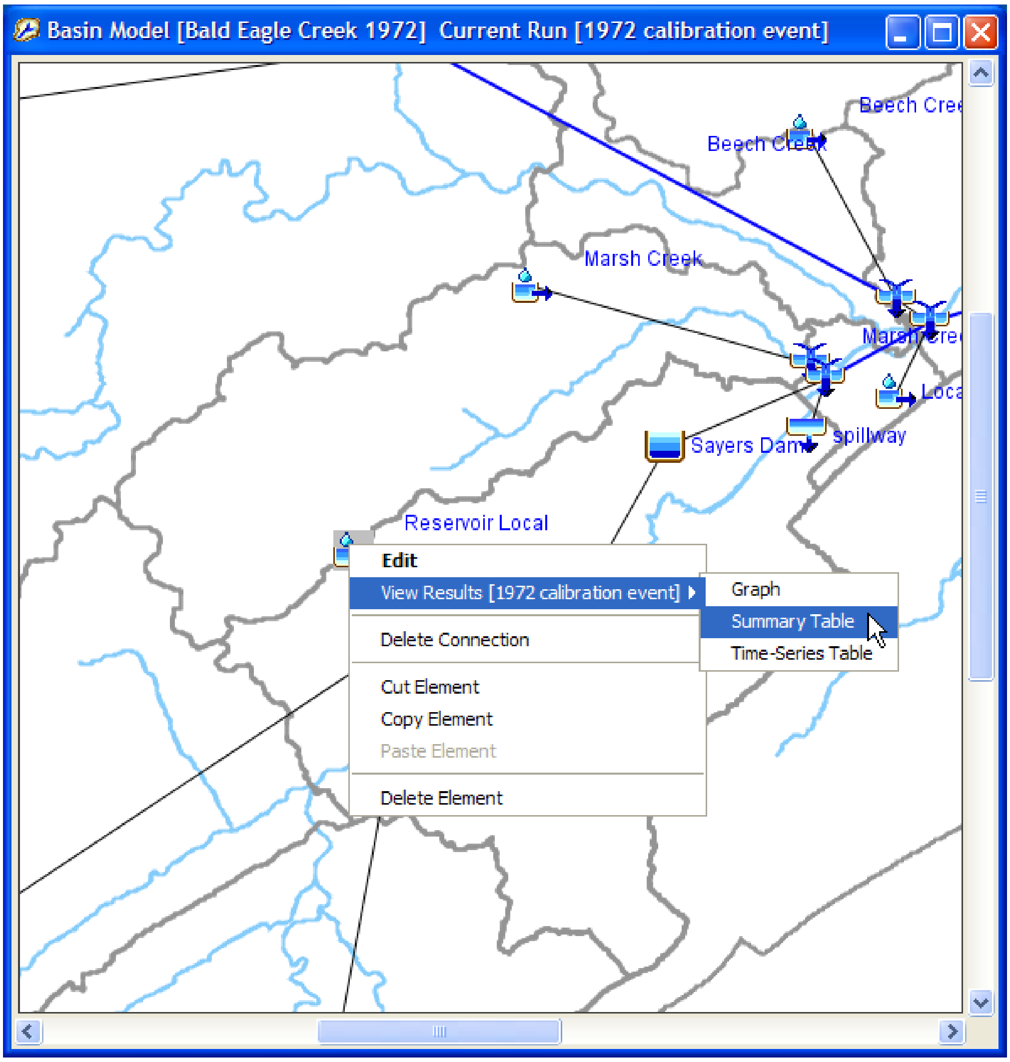


Figure 7.Accessing results for the current simulation run using the basin map.

Create or Modify Data

Many hydrologic studies are carried out to estimate the change in runoff given some change in the watershed. For example, a residential area is planned in a watershed. The change in flow at some point downstream of the new residential area is required to determine if flooding will occur as a result of the residential area. If this is the case, then two basin models can be developed. One is developed to model the current rainfall-runoff response given predevelopment conditions and another is developed to reflect future development.

An existing basin model can be copied using the basin model manager or the right mouse menu in the *Watershed Explorer*. In the *Watershed Explorer*, "Components" tab, select the basin model. Keep the mouse over the selected basin model and click the right mouse button. Select the **Create Copy…** menu item to copy the selected basin model. The copied basin model can be used to model the future development in the watershed.

To reflect future changes in the watershed, method parameters can be changed. For example, the percent impervious area can be increased for a subbasin element to reflect the increase in impervious area from development. Routing parameters can also be adjusted to reflect changes to the routing reach.

Make Additional simulations and Compare Results

Additional simulations can be created using new or modified model components. Results from each simulation run can be compared to one another in the same graph or time-series table. Select the "Results" tab in the *Watershed Explorer*. Select each simulation run that contains results you want to compare. The

Watershed Explorer will expand to show all hydrologic elements in the basin models. Select the hydrologic element in all simulation runs where results are needed. This will expand the *Watershed Explorer* even more to show available result types. Press the Control key and select each output result from the different simulation runs. When a result type is selected the result is added to the preview graph. Once all the results have been selected, a larger graph or time-series table can be opened by selecting the **Graph** and **Time-Series** buttons on the toolbar as seen in Figure 8.

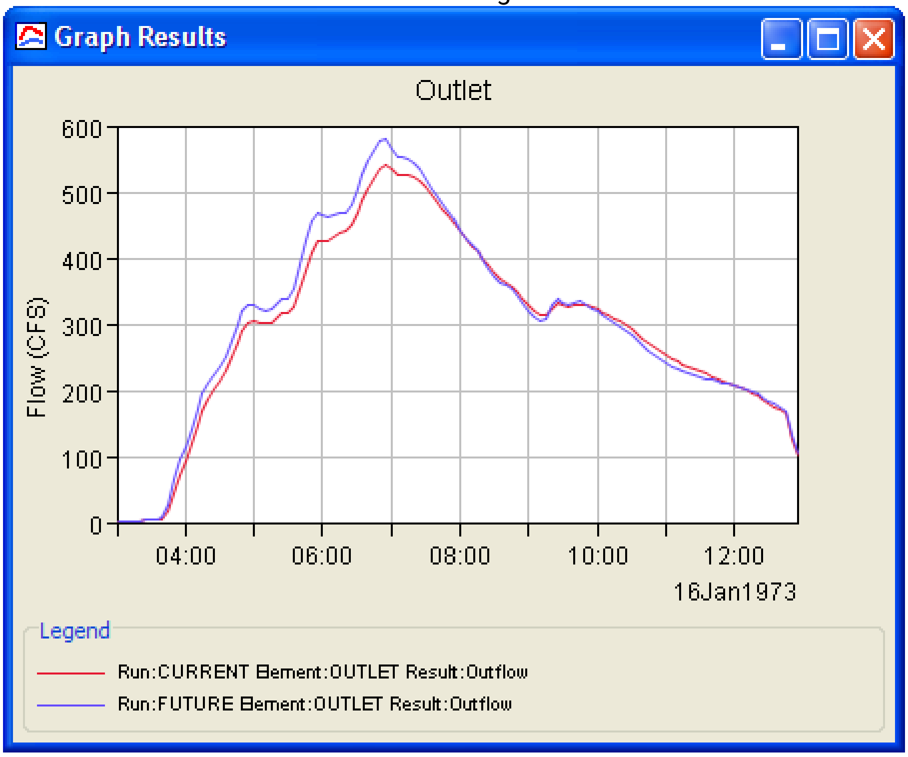


Figure 8.Graph comparing outflow in the same element in two different simulation runs. The correct time-series were selected in the Watershed Explorer and then the graph toolbar button was pressed.

3.5 References1

USACE (2005) 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

4 Geographic Information System (GIS)

4.1 GIS Basic Concepts

GIS, or geographic information systems, are computerized tools used to store, visualize, analyze, and interpret geographic data. Geographic data (also called spatial, or geospatial data) identifies the geographic location of features making these features "spatially-aware."

Spatial-awareness is not a requirement for solving problems in hydrology but since hydrology is inherently geographic, spatial-awareness can help solve some hydrologic problems more efficiently.

4.2 Basin Characteristics

4.2.1 Subbasin Characteristics

4.2.2 Reach Characteristics

4.3 GIS References

5 Meteorology

5.1 General

5.2 Precipitation

In watershed hydrology, the response of a watershed is driven by precipitation that falls on the watershed and evapotranspiration from the watershed. The precipitation may be observed as 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 that is possible at a given location at a specific time. 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. This chapter describes methods of specifying and analyzing historical or hypothetical-storm precipitation.

5.2.1 Precipitation Basic Concepts

Precipitation is the driving force behind all hydrologic processes. In the strict sense, precipitation includes rain, snow, ice, and every other form of water falling from the atmosphere and reaching the earth's surface. Without precipitation, there can be no infiltration, transpiration, surface runoff, baseflow, channel flow, or any other hydrologic process.

5.2.1.1 Mechanisms of Precipitation

While all precipitation involves some form of water falling from the atmosphere to the land surface, precipitation forms for a variety of reasons. Two principal mechanisms of precipitation are coalescence and cooling.

Coalescence

Under the coalescence mechanism, a water droplet forms around a nuclei when the temperature is below the dew point. The nuclei could be a dust particle, carbon dioxide, salt particle, or any other airborne non-water particle. As the amount of water coalesced in the droplet increases, the droplet falls at an increased velocity. The droplet will break apart when its diameter reaches approximately 7 mm. The pieces of the broken droplet can then form the nuclei of more droplets. Depending on wind conditions in the atmosphere, a droplet may grow and break apart many times before it finally reaches the ground.

Cooling

Under the cooling mechanism, precipitation occurs when the amount of moisture in the atmosphere exceeds the saturation capacity of air. Warm air can hold more water than cold air. If warm, moist air is cooled sufficiently, water in excess of the saturation capacity will fall as precipitation. Adiabatic cooling occurs when an air mass at a low elevation is lifted to a higher elevation. Frontal cooling happens along the border between a warm weather front and a cold front. Contact cooling is the result of warm air blowing across a cold lake. Finally, radiation cooling occurs when air is heated during the day and absorbs evaporated water, but then cools during the night. Any of the various cooling processes can lead to precipitation. Certain cooling processes may be more likely at some times of the year, and not all processes occur over every watershed.

5.2.1.2 Types of Precipitation

There are three different types of precipitation that are classified by the producing mechanism. The different types are closely related to weather patterns.

Convective

Convective precipitation occurs when warm, moist air rises in the atmosphere. Pressure decreases as elevation increases, which causes the temperature to fall. If the moist air mass rises to a sufficiently high elevation, precipitation will condense and fall. The tremendous energy associated with convection processes often leads to very intense precipitation rates. However, a convective storm usually has a small area and a short duration. Summer thunderstorms are the principal example of this type of precipitation.

Cyclonic

Cyclonic precipitation occurs when warm, moist air is drawn into a low-pressure, cold front. The warm air rises as it is drawn into the low-pressure zone and is subjected to adiabatic cooling. The intensity of the precipitation is determined by the magnitude of the low-pressure system and the presence of a warm and moist air mass. Cyclonic storms tend to be large and have a light to medium-intensity precipitation rate. Because of their large size, they tend to have a long duration. Most precipitation is the result of cyclonic activity.

Orographic

Orographic precipitation results when an air mass is lifted as it encounters topographic obstacles. A cold front is usually the driving force that pushes the air mass towards the obstacle, usually a mountain range. The moist air mass is mechanically lifted when the cold front forces it against a mountain range. The air mass is forced up in elevation where it cools adiabatically and precipitation results. It is important to realize that precipitation is not automatically orographic because the precipitation occurs over mountains. Orographic precipitation only results when the air mass moves perpendicular to the mountains. Within the United States, these storms are often found in the Cascade, Sierra Nevada, and Rocky mountains.

5.2.1.3 Measuring Precipitation

Accurately measuring precipitation is one of the greatest challenges in water resources engineering; the lack of measured precipitation may be a significant hurdle to hydrologic modeling. The complexity, accuracy, and robustness of a hydrologic model are meaningless if the precipitation boundary condition is incorrect. Precipitation can be measured at a point using some type of gage, or it can be measured spatially using a tool such as radar.

Point Measurements from Gages

Each of the precipitation measuring devices described in Table 10 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 10.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 9. 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, 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 10 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.

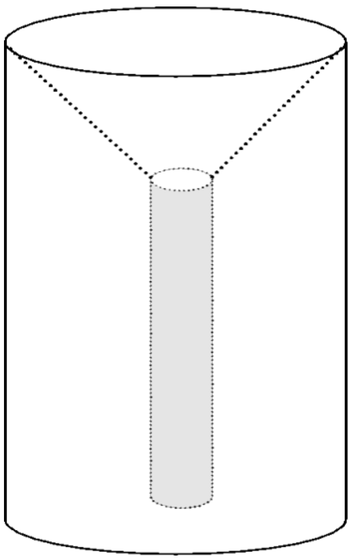


Figure 9.Manual precipitation gage.

1 Figure 10. Telemetering precipitation observation gage

Areal Measurements from Radar

Figure 11 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.

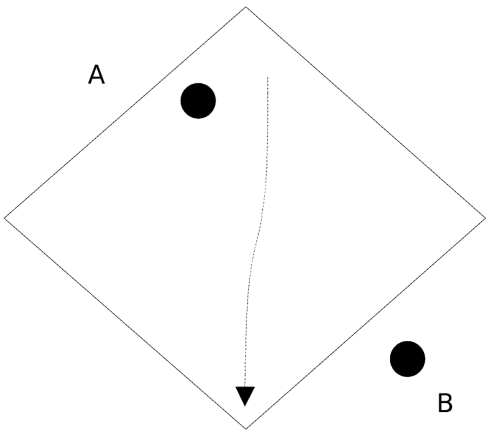


Figure 11.Mean areal precipitation can be computed as a weighted-average of depths at gages A and B.

From the gaged data, one might estimate mean areal precipitation (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 watershed

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:

$$N = A^{0.33}$$

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 measured. Precipitation gages such as those illustrated in Figure 9 and Figure 10 are typically 8-12 inches (20-30 cm) in diameter. Thus, in a one sq-mi (2.6 km^2) watershed, the catch surface of the gage 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 12. Figure 12(a) shows the watershed from Figure 11, 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 12(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.

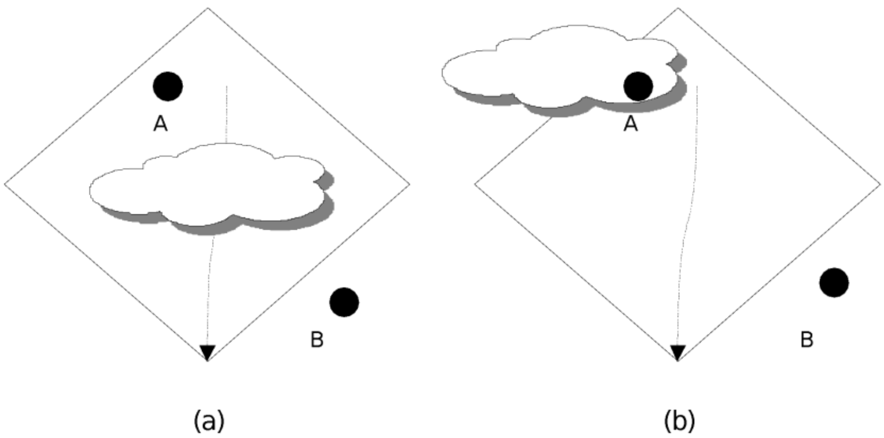


Figure 12. Lack of coverage can complicate MAP estimation.

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 be improperly estimated. A second solution is to use of rainfall depth estimates from weather radar.

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 much of the United States. 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 360º azimuthal scans, centered at the radar unit. Over a 5- to 10-minute period, successive scans are made with 0.5º 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. Varying levels of analysis may be performed to check and correct inconsistencies in the measured data. The final data products are distributed in a variety of digital formats. Grid cells are typically on the order of 4 km by 4 km.

5.2.1.4 Program Data Requirements

Chapter 6 provides details of the models for computing surface runoff from precipitation: the alternatives are various forms of the unit-hydrograph methods, and the kinematic-wave method. Inherent in methods 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.

Most of the precipitation methods included in the program compute a precipitation hyetograph for all of the subbasins in a basin model. The hyetograph is a time-series of precipitation, so it includes both the depth and timing of the precipitation. Note that the hyetograph represents a mean precipitation condition over the subbasin. The gridded precipitation method is an exception; it computes grids of precipitation over each subbasin instead of a hyetograph. Even so, the gridded precipitation is essentially a hyetograph for each grid cell.

5.2.2 User-Specified Hyetograph

The program includes a rich variety of methods for processing raw precipitation data into a hyetograph for each subbasin. However, there are so many methods for processing precipitation data that it is not feasible to include all of them within the program. This method allows you to process your own data and provide a hyetograph for the program to use. You may also choose to subdivide a watershed into many subbasins so that it becomes reasonable to use only one precipitation gage for each subbasin. In either case, the program makes no assumptions about the source of the precipitation data, it only applies the hyetograph to each subbasin as specified by the user. The hyetograph is entered in the program as if it were gage data.

While all of the major processing of the raw data must be performed external to the program, some "convenience" processing is done. The hyetograph entered by the user will be at some interval, for example, 15 minutes. However, you may wish to use control specifications with a time step different from the original data. Instead of re-entering the data manually, the program will automatically interpolate the data to the requested time step. While the data may be entered as incremental or cumulative, it will be converted to cumulative before interpolation is performed. The interpolation process does not affect the original data, it is performed "on-the-fly" during a compute so that the data agrees with the time step.

The hyetograph entered by the user inherently includes both volume and timing information. In most cases the data should be used exactly as entered, with the possibility of interpolation described above. In some cases it is convenient to prepare a hyetograph that should be treated as a pattern. In this case, the volume implicit in the hyetograph is not as important and the timing pattern it represents. You may optionally enter a total storm depth when selecting a hyetograph for each subbasin. This can be useful when examining the impact of the same storm occurring with a different precipitation depth.

5.2.3 User-Specified Gage Weights

Many watersheds are large enough that they contain multiple precipitation gages, especially in urban areas. An important question is immediately presented: How should the information at each gage be used to compute MAP over the watershed? A common approach is to take a fraction of the precipitation that occurs at each gage in order to compute MAP. The user-specified gage weights method provides a great deal of flexibility for the user to specify fractions using a generalized weighting scheme.

5.2.3.1 Basic Concepts and Equations

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

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

where P_{MAP} = total storm MAP over the subbasin; $p_i(t)$ = precipitation depth measured at time t at gage i; and w_i = weighting factor assigned to gage i. If gage i is not a recording device, the quantity $\sum p_i(t)$ is replaced by the total storm depth entered by the user. Many techniques have been developed for computing the gage weighting factors for a subbasin; some of them are described in the next section on estimating parameters. Problems can occur when interpolating precipitation over a large subbasin. For many reasons, the mean annual precipitation is likely to vary as a result of regional meteorological trends. When the variation is significant, the techniques presented previously for estimating depth factors must be modified. Consider the case where a precipitation gage has a mean annual precipitation depth of 76 cm. A subbasin in a study may be closer to that gage than to any other, but the subbasin may have an estimated annual precipitation of 88 cm. This suggests that on average, if 1 cm of precipitation is measured at the gage, that slightly more precipitation should be applied over the subbasin. The index precipitation can be used to correct for this situation.

The index precipitation for the gage and subbasin are applied together, by adjusting the gage data before it is used with the MAP factors to calculate the MAP for the subbasin. The precipitation gage data is then computed as:

$$P_{MAP} = \frac{\sum_i \left(\frac{I_{sub}}{I_i} w_i \sum_t p_i(t)\right)}{\sum_i w_i}$$

where P_{MAP} = total storm MAP for the subbasin; $p_i(t)$ = precipitation depth measured at time t at gage i; I_{sub} is the index precipitation for the subbasin; I_i is the index precipitation for the gage. As before, if gage i is not a recording device, the quantity $\sum p_i(t)$ is replaced by the total storm depth entered by the user. The index precipitation for a gage is usually estimated as the mean annual precipitation computed from the historical records at the gage. Another logical choice is to use the mean spring precipitation if the study goal is to produce a watershed model that works well only in spring months. However, there is no rule that requires the index precipitation to be the mean precipitation, whether for a year, a season, or a month. The index precipitation can be used carefully to apply a user-selected ratio of the measured precipitation to each subbasin.

While the mean annual precipitation can be estimated for a gage using the historical record, it can be difficult to estimate for a subbasin. Typically regional information on precipitation patterns must be used. One example of generally available data for estimating the annual precipitation for a subbasin is the PRISM data set (Daly, Neilson, and Phillips, 1994).

The time-series data recorded at each gage implicitly includes both volume and timing of the precipitation. In some cases it may be desirable to change the volume for a gage without changing the timing. This may be necessary if high winds during the storm cause the gage to under-catch precipitation and consequently under estimate the actual precipitation. Specifying the total storm depth for a recording gage is always optional. When a total storm depth is included, the precipitation gage data is then computed as:

$$P_{MAP} = \frac{\sum_i \left(\frac{D_{m \text{ measured}}}{D_{\text{option}}} \frac{I_{\text{sub}}}{I_i} w_i \sum_t p_i(t) \right)}{\sum_i w_i}$$

where P_{MAP} = total storm MAP for the subbasin; $p_i(t)$ = precipitation depth measured at time t at gage i; I_{sub} is the index precipitation for the subbasin; I_i is the index precipitation for the gage, $D_{measure}$ is the total depth measured at the gage, D_{option} is the optionally specified total storm depth. For a non-recording gage, $D_{measure}$ is automatically equal to D_{option}.

After the mean areal precipitation is computed, it must be distributed in time to create the hyetograph for a subbasin. A second weighting process is used to establish the temporal pattern using only recording gages. The weighting process includes a scheme to bias in favor of gages reporting precipitation when more than once gage is used to establish the temporal pattern. The hyetograph is computed as:

$$p(t) = \frac{P_{MP}}{\sum_i \left(w_i \sum_t p_i(t) \right)} \sum_i w_i p_i(t)$$

where P_{MAP} = total storm MAP for the subbasin; $p_i(t)$ = precipitation depth measured at time t at gage i; w_i is the temporal weight for gage i.

5.2.3.2 Parameter Estimation

This method requires a MAP weighting factor for each gage that will be used to compute a hyetograph for a subbasin. A separate temporal weighting factor is also required. The weights are determined by the user and entered; the program is not able to automatically estimate the weighting factors. The use of the index precipitation is optional. The following methods could be considered for estimating the weighting factors.

Arithmetic Mean MAP Factors

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 MAP Factors

This is an area-based weighting scheme, predicated on an assumption that the precipitation depth at any point within a watershed is best estimated as the precipitation depth at the nearest gage to that point. Thus, it assigns a weight to each gage in proportion to the area of the watershed that is closest to each gage.

As illustrated in Figure 13(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 textbooks.

Isohyetal Line MAP Factors

This is also an area-based weighting scheme. Contour lines of equal precipitation are estimated from the point measurements, as illustrated by Figure 13(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 textbooks.

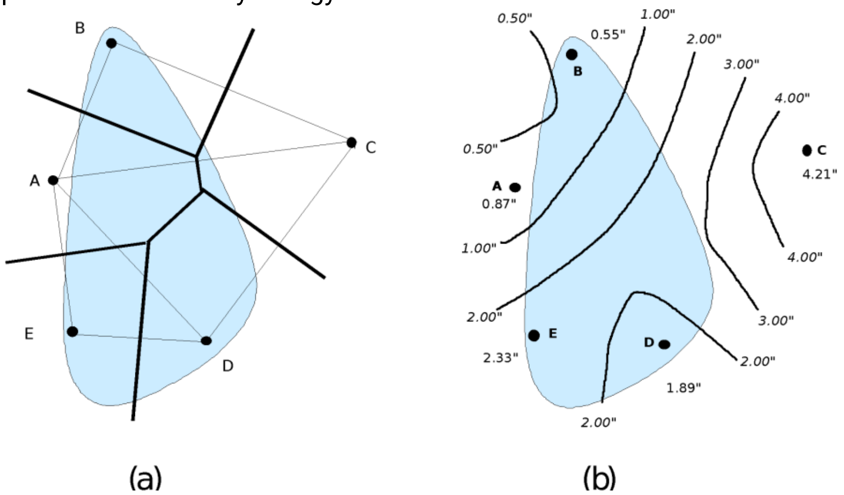


Figure 13.Illustration of MAP depth computation schemes.

Time Weighting Factors

If a single recording gage is used to establish the temporal pattern of the hyetograph, the resulting MAP hyetograph will have the same relative distribution as the one recording gage. 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 a weighted average of the pattern 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 subbasin. This is illustrated by the temporal distributions shown in Figure 14. Here, hyetographs of rainfall at two gages are shown. At gage A, rain fell at a uniform rate of 10 mm/hr from 00:00 hours until 02:00 hours. No rain was measured at gage A after 02:00. At gage B, no rain was observed until 02:00, and then rainfall at a uniform rate of 10

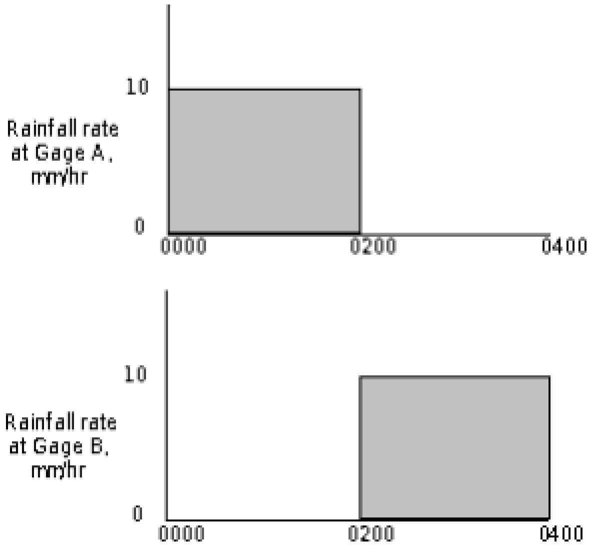


Figure 14.Illustration of the hazard of averaging rainfall temporal distributions.

mm/hr was observed until 04:00. The likely pattern is that the storm moved across the subbasin from gage A toward gage B. If these gage data are used with Equations 4 and 5 to compute an average pattern, weighting each gage equally, the result is a uniform rate of 5 mm/hr from 0:000 until 04:00. 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.

5.2.4 Inverse Distance Gage Weighting

The inverse distance gage weighting method was originally designed for real-time forecasting applications. It includes procedures for automatically using the closest available gage data, but switching to more remote gages if close gages stop reporting or contain missing data.

5.2.4.1 Basic Concepts and Equations

As an alternative to separately defining the total mean areal precipitation (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 subbasin 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 subbasin such that they provide adequate spatial resolution of precipitation in the subbasin. 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 15. 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 15, the weight for the gage C in the northeastern quadrant of the grid is computed as:

$$w_C = \frac{\frac{1}{d_C^2}}{\frac{1}{d_C^2} + \frac{1}{d_D^2} + \frac{1}{d_E^2} + \frac{1}{d_A^2}}$$

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. The distance between each gage and the node is computed using a curved earth assumption as:

$$d = rad \cdot \cos^{-1}(\cos(A) \cdot \cos(B) + \sin(A) \cdot \sin(B) \cdot \cos(C))$$

where *d* is the distance between a gage and the node; *rad* is the radius of the earth at 6370 km; *A* is one-half pi minus latitude of the gage in radians, *B* is one-half pi minus the latitude of the node in radians, and *C* is the longitude of the node in radians minus the longitude of the gage.

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

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

This computation is repeated for all times *t*. Note that gage B in Figure 15 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.

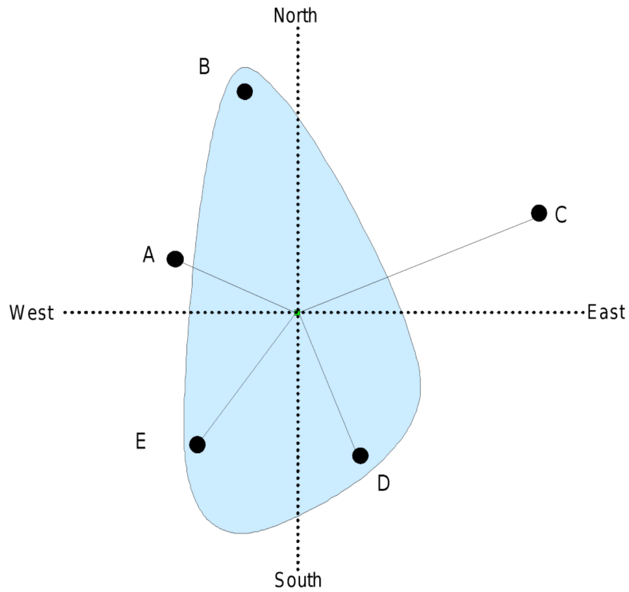


Figure 15.Illustration of inverse-distance-squared scheme.

As noted in the description of the user-specified gage weights method, regional trends in meteorology and specifically precipitation can lead to problems. The index precipitation can be used to account for these trends. Each node can have an index depth as well as each gage. The index correction can only be used if it is specified for all gages and nodes that will be used for a subbasin. When used, the index precipitation values are included in the calculations as:

$$p_{node}(t) = \frac{I_{node}\,w_A}{I_A}p_A(t) + \frac{I_{node}\,w_C}{I_c}p_c(t) + \frac{I_{node}\,w_D}{I_D}p_D(t) + \frac{I_{node}\,w_E}{I_E}p_E(t)$$

Where I_{node} is the index at the node, I_A is the index at gage A, and I_C, I_D, and I_E are the index at each of the remaining gages.

This example has used only one node in the subbasin. However, it is possible to include more than one node. In this case a weight must be specified for each node as the final precipitation hyetograph for the subbasin is computed as:

$$p(t) = \sum_i w_i\,p_i(t)$$

Where *p(t)* is the precipitation at each time *t* for the subbasin; w_i is the weight for node *i*; $p_i(t)$ is the precipitation at each time *t* for node *i* as computed by equation 10.

Most recording gages that are used with this method report data at a 1-hour or shorter interval. However, there are many gages available that only report once a day, giving the total daily precipitation. These gages can also be used for calculating the precipitation at each node. Each "daily" gage is preprocessed before beginning the calculations for a node. The processed daily gage is used exactly as if it were a recording gage when dynamically computing the precipitation for each node.

The preprocessing for a daily gage utilizes recording gages near the daily gage to compute a pattern hyetograph and then applies the precipitation recorded at the daily gage. The processing is similar to what happens at a node. Hypothetical north-south and east-west axes are constructed through the coordinates of the daily gage. The adjacent recording gages are sorted into each of the four quadrants surrounding the daily gage. The closest gage is selected in each quadrant; this process is performed once and not for each time interval as is done when processing for a node. For each time step, pattern precipitation is computed at the daily gage using equations 7, 8, and 10 with the substitution of the daily gage coordinates for the node coordinates. Subsequently, the pattern precipitation hyetograph is normalized by dividing each value by the total depth in the pattern precipitation. This step yields a pattern hyetograph for the daily gage that has exactly 1 unit of precipitation depth. The next step in preprocessing the daily gage is to determine the total depth recorded at the daily gage. Only time intervals at the daily gage that are completely within the simulation time window are included for computing the total depth. For example, consider a daily gage that is used in a simulation at begins on the first day at 12:00 hours and continues until 12:00 hours on the fifth day. Only the daily gage values recorded on the second through fourth days will be included. This is necessary because it is not possible to know when the depth recorded by the daily gage on the first day happened during the day. It is possible that it occurred during the first 12 hours of the day which are not included in the simulation time window, so they must be excluded. Each value in the normalized pattern hyetograph is multiplied by the calculated total depth at the daily gage to yield the final gage hyetograph used in node calculations.

5.2.4.2 Parameter Estimation

This method requires at least one node for each subbasin. The parameters then are the latitude and longitude of the node and any gages that will be used. The coordinates of each gage are generally known or can be found by examining a map. The use of the index precipitation is optional.

Selecting Node Locations

A common practice is to specify a single node for each subbasin, located at the centroid of the subbasin. This can be a quick way to initially estimate parameters because it is relatively easy to compute the coordinates of the subbasin centroid using a geographic information system. This is less arbitrary that it first seems. By definition, the centroid is closer to more of the subbasin than any other point. Using this placement assumes that centering in the subbasin is the best representation of the subbasin-average precipitation.

An alternate method of placing the node would be to examine the precipitation trends over the subbasin. Compute the average annual subbasin precipitation by consulting regional maps. One source for such maps is the PRISM project (Daly, Neilson, and Phillips, 1994) which includes both total annual and monthly estimates of precipitation. Once the average annual precipitation amount for the subbasin is known, a node could be located at a point in the subbasin with the same average annual precipitation depth. Ideally, the selected point would be near the centroid. This placement attempts to match individual storm events that will be simulated consistent with the average trends in the subbasin.

Regardless of the method used to locate the node, placement should consider the surrounding gages. Sometimes the location initially selected for the node will almost always ignore a gage that is relatively close, as shown in Figure 16. Because of the node placement, all four gages fall into only three quadrants. Most of the time only the three closest gages will be used and gage B will never be used unless there is missing data at gage A. By moving the node slightly East, each gage will fall into a separate quadrant and be used.

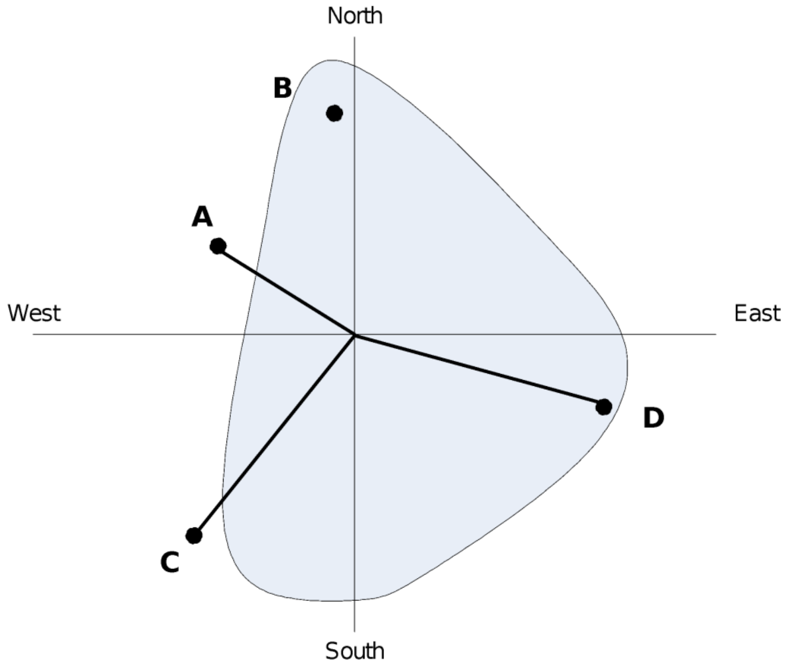


Figure 16.Placing a subbasin node where one gage is excluded. Moving the node to the West would use all four gages.

It is possible to use more than one node in a subbasin. However, if two or more nodes are used, the pattern for the subbasin will be a weighted average of that computed at those nodes. Consequently, if the temporal distribution at those nodes is significantly different, as it might be with a moving storm, the average pattern may obscure information about the precipitation intensity over the subbasin. If circumstances cause precipitation patterns to differ significantly over a subbasin so that multiple nodes may be necessary, it is often better to subdivide the subbasin rather than use multiple nodes.

Daily Gages

Strictly speaking, any gage can be considered a "recording" or "daily" gage. A recording gage will be interpolated in time in order to obtain values at the time step of the simulation. As an example, consider a recording gage with data recorded at 1-hour intervals. If the simulation is computed at a 15-minute time interval, then each value in the gage time-series will be divided into four equal amounts during each hour in order to have a value for each simulation time step. This so-called down scaling can be an acceptable approximation of the precipitation but the approximation becomes worse as the ratio of simulation time interval divided by the gage recording interval becomes smaller. It is a fact that gages cannot capture information about the temporal distribution of precipitation at time scales smaller than the recording interval of the gage.

As a second example, consider a gage that records data at a 1-day time interval that will be used in a simulation with a 1-hour time step. If the gage were treated as a typical recording gage, each daily value would be divided into 24 equal values for the simulation time step during each day. In general, this would lead to unacceptable smoothing of the daily data. Designating the gage as "daily" will allow the processing described above to convert the data to a better approximation of what happened at short time scale by using adjacent high-resolution gages.
You should always have at least one gage with a data time interval near the simulation time interval. Any gage with a data time interval more than several times the simulation time interval should be evaluated for treatment as a "daily" gage. For example, if the simulation time interval is 1-hour and some gages report at 6-hour intervals, they should be evaluated for treatment as daily gages. Even though they report at less that 24-

hour or daily intervals, their time interval is so much longer than the simulation interval that results may be significantly improved by using the "daily" processing option.

5.2.5 Gridded Precipitation

By far, the most common use of gridded precipitation is for data collected by weather radar. It is possible to create gridded precipitation from gages, for example, using an inverse distance squared scheme between all gages and each grid cell. It is also possible to extract gridded precipitation data from some atmospheric models. The remainder of this section will focus on the common application with weather radar.

5.2.5.1 Basic Concepts and Equations

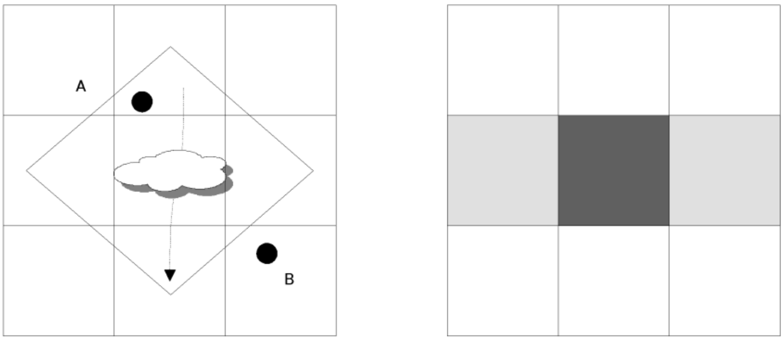
Given the radar reflectivity, the rainfall rate for each grid cell can be inferred because the power of the reflected signal is related to the size of and density of the reflecting obstacles. The simplest model to estimate rainfall from reflectivity is a Z-R relationship, and the most commonly-used of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

$$Z = aR^b$$

in which Z is the reflectivity factor; R is the rainfall intensity; and a and b are 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 National Weather Service, 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.

The following figure illustrates the advantages of acquiring weather radar data. Figure (a) shows the watershed with a 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 (b), with color codes for various intensity ranges. (This is similar to the images seen on television weather reports.)



(a) (b)

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.

5.2.5.2 Parameter Estimation

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. Precipitation gages must be used to measure the amount of water that actually reaches the ground. Furthermore, radar cannot differentiate liquid and frozen water in the atmosphere. Rain drops and snowflakes have very different reflectance properties and different Z-R relationships must be used. Temperature information must be used to determine the likely state of the precipitation so that the correct Z-R relationship can be used. As noted earlier, the process is complicated by the fact that the gridded precipitation represents an integrated average over the grid cell and not the value at a particular point in the cell, such as a precipitation gage. Because of these and other complications, the work of correcting or calibrating weather radar into a usable precipitation estimate should be done by experienced meteorologists. When such processing is done by the National Weather Service, the final estimate is referred to as a Stage III product. All processing must be done external to the program. The final integrated gridded product is loaded into the program via a HEC-DSS file.

5.2.6 Frequency 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. Nesting the various precipitation depths leads to the notion of a "balanced" storm. For example, consider a synthetic storm with 0.1 annual exceedance probability (AEP.) If the storm is 6 hours long, it will also contain the 3-hour storm, and the 1-hour storm. When actual historical gage records are examined, it is not necessarily true that the 0.1-AEP storm 6 hours long contains the 0.1-AEP storm 1 hour long. However, generating nested storms does produce consistent results that are valuable for design and regulation purposes.

5.2.6.1 Basic Concepts and Equations

The development of a frequency storm begins with precipitation depths entered by the user. Each depth is associated with a duration. The shortest duration is often called the peak intensity duration and usually should match the simulation time interval. The longest duration may be up to 10 days; the exact length depends on the purpose for developing the frequency storm. A precipitation depth must be entered for each duration, from the selected shortest to the selected longest. All depth-duration precipitation values must be for the same annual exceedance probability.

The depth-duration values entered by the user are first augmented using relationships found in HYDRO-35 (Fredrick, Myers, and Auciello, 1977.) Analysis of high-resolution precipitation gages found that depths for intermediate durations can be reliably estimated as:

$$P_{10-min} = 0.59P_{15-min} + 0.41P_{5-min}$$
$$P_{30-min} = 0.49P_{60-min} + 0.51P_{15-min}$$

The estimated 10-minute and 30-minute depths are inserted into the depth-duration values entered by the user to create an augmented depth-duration relationship.

The augmented depth-duration relationship is next adjusted for storm area. The values entered by the user are so-called point values. Point values represent the precipitation characteristics observed at a point in the watershed. Precipitation at a point (perhaps measured by a rain gage) can be very intense and change rapidly over a short time, but high intensity cannot be sustained simultaneously over a large area. As the area of consideration increases, *average* intensity decreases. For example, a small thunderstorm may release an intense burst of rainfall over a small area. However, the physical dynamics of thunderstorms do not allow for the intense rainfall to be widespread. Further, if you were to consider a large area around the thunderstorm, the same precipitation volume averaged over the large area would result in a much lower intensity. For a specified frequency and storm 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) used averages of annual series of point and areal values for several dense, recording-raingage networks to develop reduction factors. The factors indicate how much 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 18. These factors are used in HEC-HMS to automatically adjust the depth-duration values entered by the user and the augmented values based on the storm area, which is also specified by the user.

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 small time of concentration. A small time of concentration, in turn, is indicative of a relatively small watershed, which, in turn, requires no adjustment. HEC-HMS implements this recommendation.

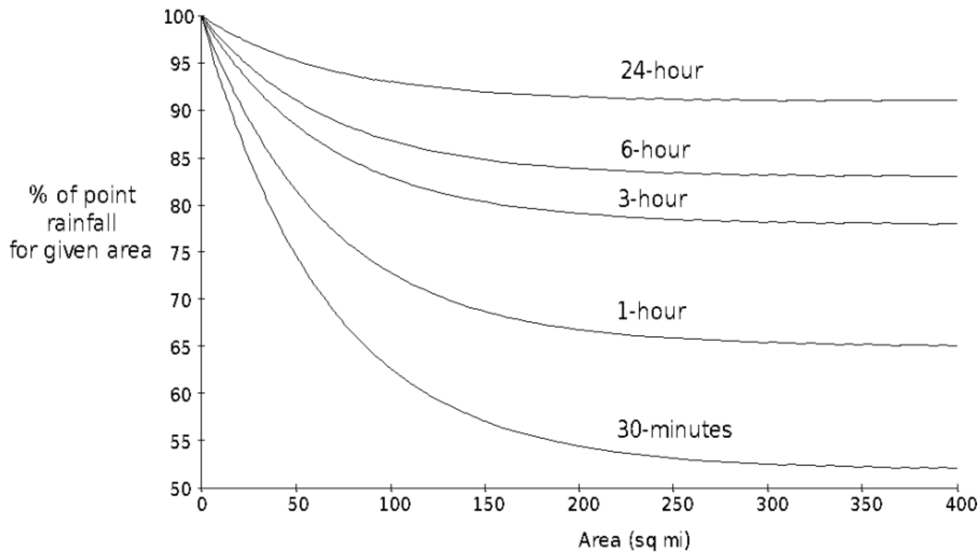


Figure 18.Reduction of point rainfall depth as storm area increases.

The last adjustment of the depth-duration relationship accounts for the type of input that is used, and the type of output that is desired: annual duration or partial duration. Virtually all precipitation analyses that give depth for a specific duration use partial duration values. This means that the entire precipitation record was scanned and all events greater than a threshold value were included in the statistical analysis. In an analysis of this type, some years may contribute multiple events while other years provide none. However, in some cases the input precipitation data may use annual duration instead. The user must select the type of precipitation data that will be entered, and the type of output that is desired. When the input and output types do not match, the reduction factors shown in Table 11 are used to convert the data as necessary. Note that the conversion only applies to relatively frequent storms. As the annual exceedance probability decreases, the difference between annual and partial duration statistics becomes negligible.

Table 11.Reduction factors for converting partial-duration input to annual-duration output.

Exceedance Probability	**Reduction Factor**
0.50	0.88
0.20	0.96
0.10	0.99

Finally, the program is able to use the processed depth-duration relationship to compute a hyetograph. It interpolates to find depths for durations that are integer multiples of the time interval selected for runoff modeling. Linear interpolation is used, after taking logarithms of both the depth and duration data. Performing the interpolation in log-log space improves the quality of intermediate estimates (Herschfield, 1961.) The interpolation yields successive differences in the cumulative depths, thus computing a set of incremental precipitation depths, each of duration equal to the selected computation interval.
The alternating block method (Chow, Maidment, Mays, 1988) is used to develop a hyetograph from the incremental precipitation values (blocks). This method positions the block of maximum incremental depth first at the specified location in the storm. The user may choose from values of 25, 33, 50, 67, or 75% of the

time measured from the beginning of the storm to the total storm duration. The remaining blocks are arranged then in descending order, alternating before and after the central block. When the maximum incremental depth is not located at 50%, the arranged blocks will stop alternating as soon as the front (25 and 33%) or back (67 and 75%) of the storm is filled; remaining blocks are placed on the free side of the storm. Figure 19 is an example of this temporal distribution; this shows the rainfall depths for a 24-hour hypothetical storm with a 1-hour computation interval.

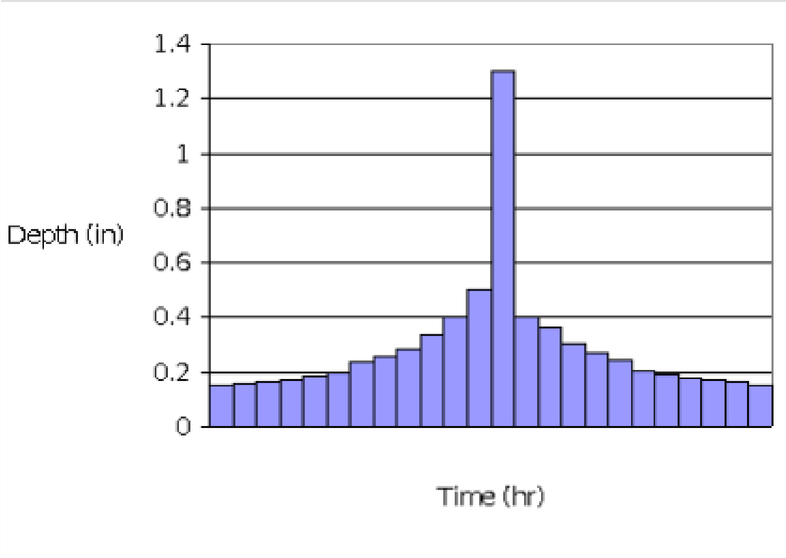


Figure 19.Example of distribution of frequency-based hypothetical storm.

5.2.6.2 Parameter Estimation

In the United States, depths for various durations can be obtained from a variety of sources. Several products are available for the entire country, including TP-40 (Herschfield, 1961) for durations from 30 minutes to 24 hours and TP-49 (Miller, 1964) for durations from 2 to 10 days. The Eastern part of the country has extra data for short durations in HYDRO-35 (Fredrick, Myers, and Auciello, 1977.) Some locations have specialized data developed locally, for example the Midwest has available Bulletin 71 (Huff and Angel, 1992.) The Pacific coast, inter-mountain, and Rocky Mountain states are covered by NOAA Atlas 2 (Miller *et al.*, 1973) with a separate volume for each state. The Ohio River Valley and the Southwest Dessert are covered by NOAA Atlas 14 (Bonnin *et al.*, 2004). These various reports are all similar in that they contain maps with isopluvial lines of constant precipitation depth. Each map is labeled with an annual exceedance probability and storm duration. Knowing the location of the watershed on the map, the depth for each required duration and exceedance probability can be interpolated between the isopluvial lines.

Each of the maps included in the reference sources is developed independently of other maps. That is, the map for the 0.01 probability of exceedance and 1-hour duration is developed separately from the 0.01 probability of exceedance and 6-hour duration. Because of this independence there can be inconsistencies in the values estimated from the maps. If this raw data is input to the program, there can result fluctuations in the computed hyetograph. These fluctuations can be reduced by smoothing the data before entering it into the program. The precipitation depth values for the range of durations, all with the same exceedance probability, should be plotted. A smooth line should be fit through the data. A best-fit line can be used without attempting to fit a line of a particular function. Precipitation values can then be determined from the smooth line, and those values input to the program.

The storm area should be set equal to the drainage area at the evaluation location. The evaluation location will be where the flow estimate is needed, for example at the inflow to a reservoir or at a particular river station where a flood damage reduction measure is being designed. When there are several evaluation locations in a watershed, separate storms must be prepared for each location. Failure to set the storm area equal to the drainage area at the evaluation location leads to incorrect depth-area adjustments and either over or underestimation of the flow for a particular exceedance probability.

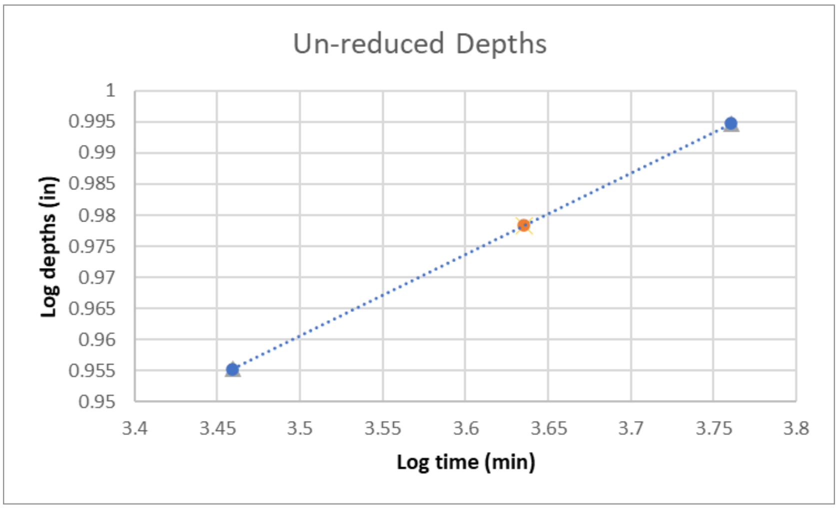
As stated earlier, virtually all precipitation maps are given in partial duration form, so input is assumed to be of partial duration type. The selection of the output type as partial or annual duration depends on the intended use of the computed results. Frequently the computed results are used for floodplain regulation or the design of flood damage reduction measures. In these cases, the type of output is determined by the type of damages that are expected. Some damages are generally assumed to happen only once in a year. For example, the time required to rebuild residential housing usually means it can only be damaged once in a year. If two large floods occurred in the same year, the housing would be flooded twice before it could be rebuilt and no additional damage would occur. Annual duration output should be selected. Partial duration output should be used if damages can happen more than once in the same year. This is often the case in agricultural crops, where fields can be plowed and replanted after a flood only to be reflooded. Partial duration output may also be appropriate when ever the recovery time is so short that multiple damaging floods may happen in the same year.

5.2.6.3 Frequency Storm 3-Day Depth Compute

The Frequency based hypothetical storm creates a balanced storm for a given time interval by performing a linear interpolation on the precipitation-frequency duration and depths in log-log space. Prior to HEC-HMS version 4.10, the 3-day depth was not a depth that users could input, while in HEC-HMS version 4.11 and onward, users are allowed to directly input 3-day depths. Users could experience differences in their balanced hyetograph after migrating their software from 4.10 to a later version of the software.

5.2.6.3.1 3-day depth migrating from version 4.10 (or older) to 4.11 (or newer)

The 3-day depth is computed by logging of the 2-day and 4-day depth (blue) before areal reduction due to the storm area and interpolating the 3-day depth (orange) as shown in the Figure below. Converting the logarithm by raising the base value gives you the 3-day depth value added to the component editor.

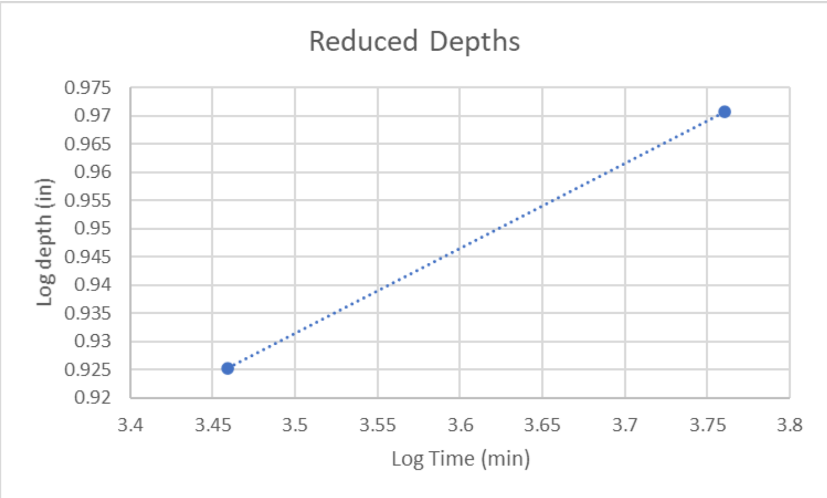


For example:

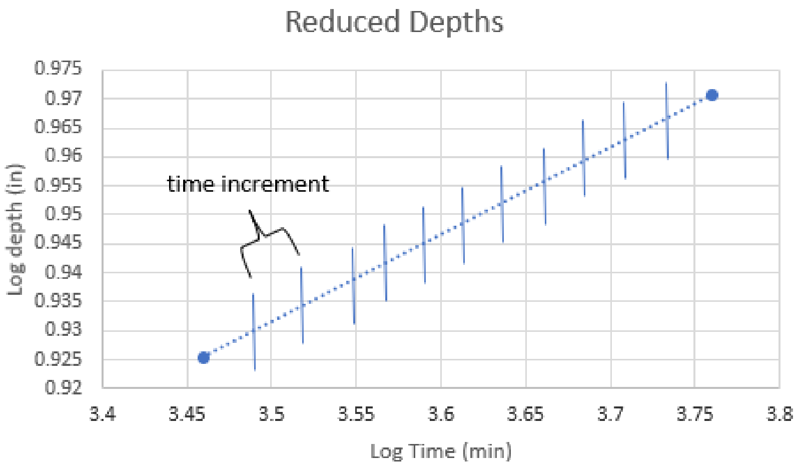
- Interpolated unreduced 3-day Log Depth = 0.9783 in
- Un-log 3-day Depth = 9.5135 in

5.2.6.3.2 3-day depth in HEC-HMS 4.10 (and older)

The 3-day depth was not directly computed in the software but could be estimated after the balanced hyetograph is created. To estimate a 3-day depth, the 2-day and 4-day depths are first areally reduced. The figure below shows just the log 2-day and 4-day reduced depths; however, this interpolation is performed from the intensity duration out to the storm duration to build the storm hyetograph.



For a given time increment (defined by the computation time interval), linear interpolation in log-log space is performed at the time interval to estimate the cumulative depths between the 2-day and 4-day durations.



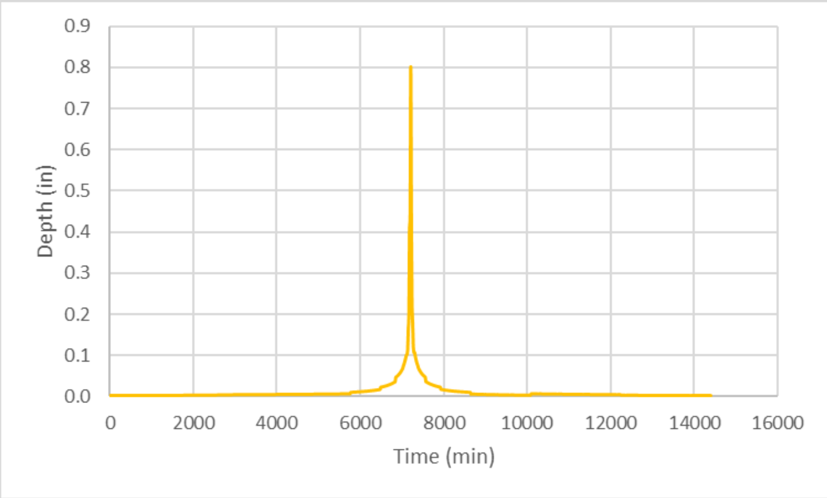
The interpolated log depths are used to compute an incremental depth value. Un-logging the interpolated depth values and sequentially subtracting the cumulative depth by the next cumulative depth gives you the incremental depth value.

For example, for a 15 minute time interval:

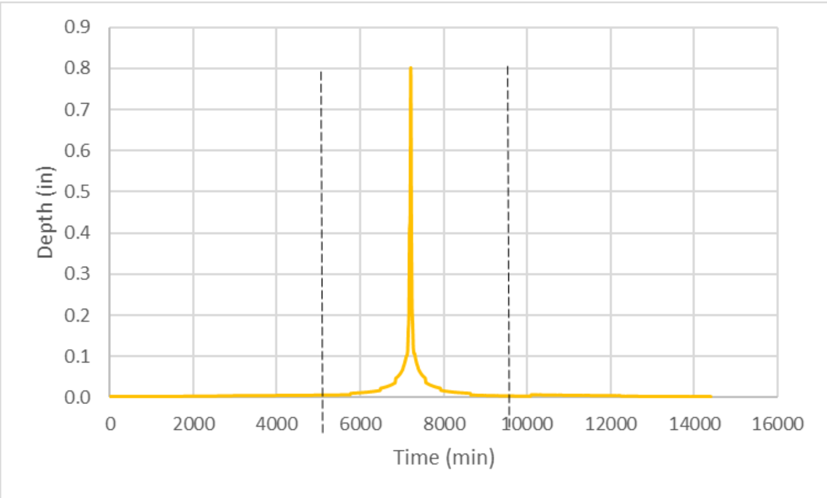
- Log depth for 2910 min = 0.9260 in
- Un-log depth for 2910 min = 8.4333 in
- Log depth for 2925 min = 0.9263 in
- Un-log depth for 2925 min = 8.4399 in

- 15 minute increment = 0.0066 in

The increment depths are sorted to by staggering the largest increment depths in the center, and interchanging the next largest depth to the right or left of the largest depth.



The 3-day "reduced" depth can be computed by finding the maximum 3-day depth after sorting the increments.



To obtain the "un-reduced" 3-day depth (backing out of areal reduction), the areal reduction factor needs to be computed. For TP-40 areal reduction, the equation is

$$Areal\,Reduction\,Factor = 1 - factor * (1 - e^{-0.015 * drainagearea}))$$

The factor is different for each duration. The factor is set to 0.12 for the 3-day reduction curve.

For example:

- Reduced 3-day Log Depth = 0.9519 in
- Reduced 3-day Depth = 8.9514 in
- Reduction Factor (3-day for 254.52 square mile watershed, TP-40): 0.9413
- Un-reduced 3-day Depth = 9.5100

5.2.7 MetSim Precipitation

5.2.7.1 Basic Concepts and Equations

The **MetSim Precipitation** method, as implemented within HEC-HMS, follows that which is described in Bohn, et al (2019)[2]. This method interpolates a sub-daily precipitation depth from an input daily precipitation accumulation through the use of an isosceles triangle shaped temporal distribution. This method uses two parameters for disaggregation: a storm duration and a storm time to peak. The base width of the triangle is equivalent to the storm duration parameter while the triangle's time of peak is equivalent to the storm time to peak parameter.

The computations begin by calculating the peak intensity of the triangle, such that the area of the triangle equals the daily total P_{daily}. This value is calculated as:

1)
$$I(d,t) = P_{daily}(d) * k_{tri}(t)$$

where *I(d, t)* [in/hr or mm/hr] is the instantaneous intensity at time *t* within day *d*, $P_{daily}(d)$ [in or mm] is the daily total *P* on day *d*, and $k_{tri}(t)$ (hours) is the kernel function (unit hyetograph) of the isosceles triangle. The kernel function is defined as:

2)
$$k_{\text{tri}}(t) = \begin{cases} \frac{2}{D} + \frac{4}{D^2}\left(t - t_{\text{pk}}\right), & t_{\text{pk}} - 0.5D < t < t_{\text{pk}} \\ \frac{2}{D} - \frac{4}{D^2}\left(t - t_{\text{pk}}\right), & t_{\text{pk}} < t < t_{\text{pk}} + 0.5D \\ 0, \text{ all other } t \end{cases}$$

where t_{pk} is the storm time to peak [hours] and *D* is the storm duration [hours], which is allowed to range from 0.5 * Δt (where Δt is the computational time step) to 24 hours. A non-monotonic linear interpolation function, implemented using the Apache Commons Mathematics Library[3], is used to construct this kernal function. Once a kernal function has been constructed, a kernal value for the current computational interval, *k(t)*,is extracted. This kernal value is then normalized as:

3)
$$k_{normalized} = k(t) * \frac{2}{D}$$

The normalized sum of all kernal values, $k_{normalized\ sum}$, within a day for the kernel function is then computed. Finally, the sub-daily precipitation depth, $P_{sub\text{-}daily}$, is computed as:

4)
$$P_{sub-daily} = P_{daily}/k_{normalizedsum} * k_{normalized}$$

2 https://doi.org/10.1175/JHM-D-18-0203.1
3 https://commons.apache.org/proper/commons-math/

5.2.7.2 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include a daily precipitation gridset, a Temporal Disaggregation method, a Storm Characteristics method, a Storm Duration [hours], and a Storm Time to Peak [hours]. An optional Time Shift method can be used to adjust the gridded precipitation data in time. Currently, only the Bohn el al. 2019 option is available for the Temporal Disaggregation method. Additional options will be added in the future. There are three options for the Storm Characteristics method: Fixed Value, Annual Pattern, and Grid.

A tutorial describing an example application of this precipitation method can be found here: Using the New MetSim Precipitation and Temperature Methods[4].

5.2.8 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)

5.2.8.1 Basic Concepts and Equations

The SPS model included in the program is applicable to watersheds within the continental United States and East of 105° longitude. 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, the regulations that originally instituted the SPS have not been rescinded and it may still be necessary to compute it as part of designing a flood protection project. A detailed description of the SPS can be found in the EM 1110-2-1411 and also in HEC Training Document No. 15 (USACE, 1982). Development of the SPS begins with the specification of the index depth. The program calculates a total storm depth distributed over a 96-hour duration using:

$$Totaldepth = \sum_{i=1}^{4} (R_{24HR}(i) \cdot \mathrm{SPFE})$$

4 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/under-development-not-visible-to-public/using-the-new-metsim-precipitation-and-temperature-methods

where SPFE is the standard-project-flood index-precipitation depth in inches; and R24HR(i) is the percent of the index precipitation occurring during 24-hour period i. R24HR(i) is given by:

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

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:

$$R_{6HR(i)} = \begin{Bmatrix} R_{6HR}(4) - 0.033 & \text{if } i = 1 \\ 0.055 * (SPFE - 6.0)^{051} & \text{if } i = 2 \\ \frac{13.42}{(\text{SPFE} + 11.0)^{0.93}} & \text{if } i = 3 \\ 0.5 * (1.0 - R_{6HR}(3) - R_{6HR}(2)) + 0.0165 & \text{if } i = 4 \end{Bmatrix}$$

where R6HR(i) is the ratio of 24-hour precipitation occurring during hour period i. The program computes the precipitation for each time interval in the jth 6-hour interval of the ith 24-hour period (except the peak 6-hour period) with:

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

where Δt is the computation time interval, in hours. The peak 6-hour precipitation of each day is distributed according to the percentages in Table 12. When using a computation time interval less than one hour, the peak 1-hour precipitation is distributed according to the percentages in Table 13. (The selected time interval must divide evenly into one hour.) When the time interval is larger than shown in Table 12 or Table 13, 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.

Following the development of the distribution, the hyetograph for each subbasin is computed as the transposition factor multiplied by the distribution depth.

Table 12. 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 13.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

5.2.8.2 Parameter Estimation

A storm area must be selected in order for the distribution to be developed. In general, the area should match the drainage area for the watershed that drains to the location where the flood protection project will be constructed. The area may be slightly larger than the drainage area at the actual proposed construction site.
The SPS index precipitation value is taken from Plate 2 in EM 1110-2-1411, as shown in Figure 20. The lowest isohyet line has a value of 9 inches and passes through central Minnesota, Northern Michigan, New York, and Maine. A high isohyet line with a value of 19 inches follows the Texas-Louisiana gulf coast and crosses to Florida. Select the best index precipitation value based on the location of the flood protection project.
Each subbasin must have a so-called transposition factor. The factors are selected by overlaying the SPS isohyetal pattern over the complete project watershed. An area-weighted average should be used to determine the factor for each subbasin. The isohyetal pattern is taken from Plate 12 in EM 1110-2-1411, as shown in Figure 21.

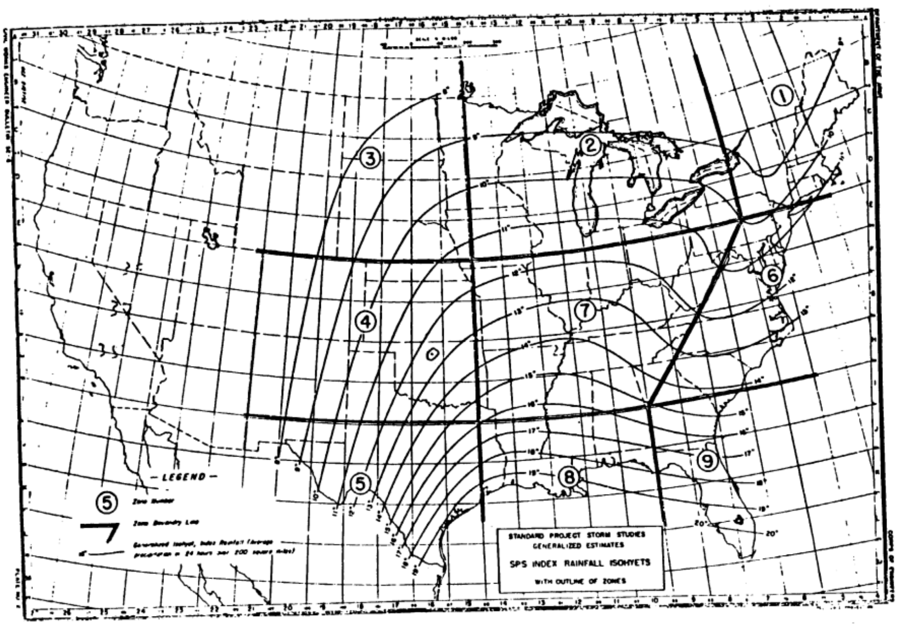


Figure 20.Reproduction of Plate 2 from EM 1110-2-1411.

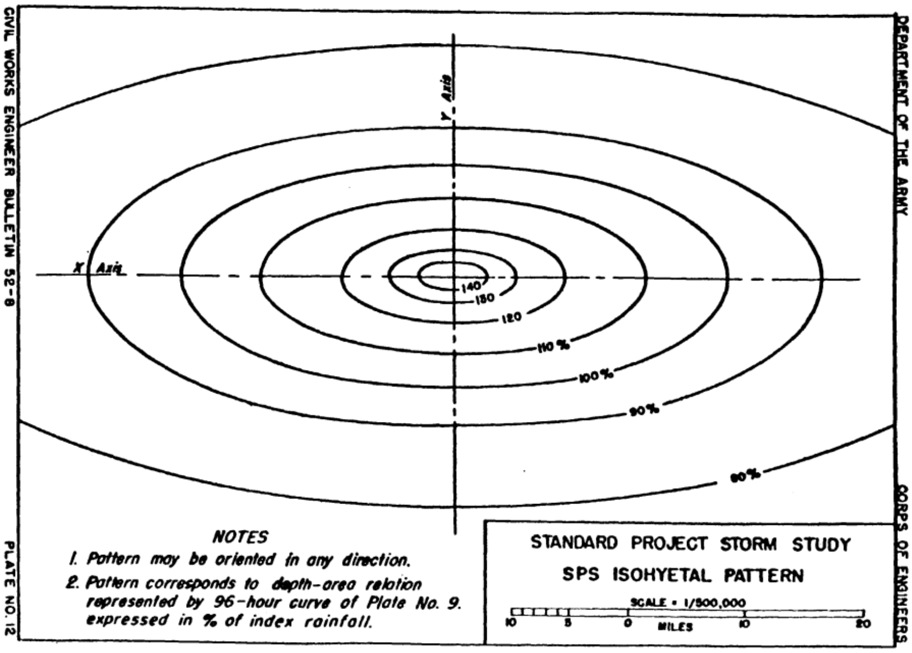


Figure 21.Reproduction of Plate 12 in EM 1110-2-1411.

5.2.9 SCS Storm

Drainage planning in the United States is often performed using hypothetical storms developed by the Soil Conservation Service (SCS), now known as the Natural Resources Conservation Service (NRCS). These storms were developed by the SCS as averages of rainfall patterns; they are represented in a dimensionless form.

5.2.9.1 Basic Concepts and Equations

The SCS designed the storm for small drainage area of the type for which they usually provide assistance. The intended use is for estimating both peak flow rate and runoff volume from precipitation of a "critical" duration. Storm producing mechanisms vary across the United States so four different storm patterns were developed. The patterns are shown in Figure 22; the actual data values can be found separately (USDA, 1992). The so-called Type I and Ia storms represent Pacific climates with generally wet winters and dry summers; these are used on the Pacific coast from Washington to California, plus Alaska and Hawaii (Figure 23). The Type III storm represents areas bordering the Gulf of Mexico and Atlantic seaboard where tropical storms and hurricanes generate heavy runoff. The Type II storm is used in the remainder of the United States. Storm types have not been defined for other locations in the world; a storm type may be selected based on similar weather patterns and comparisons of cumulative precipitation for typical storms.

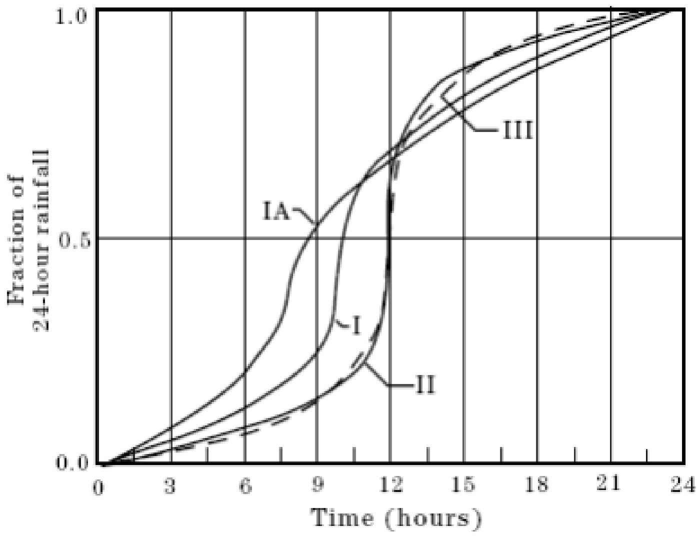


Figure 22.Reproduction of Figure B-1 in TR-55. SCS 24-hour rainfall distributions.

5.2.9.2 Parameter Estimation

The storm type should be selected based on the location of the watershed after consulting Figure 23. The boundaries are approximate so engineering judgement may be used to select a storm type on the basis of the meteorologic patterns. The precipitation depth to be applied to the pattern can be selected from any of the sources discussed in the Frequency Storm section above. Because the SCS storm method does not account for depth-area reduction or annual-partial duration conversion, the user must make these adjustments manually before entering a depth value.

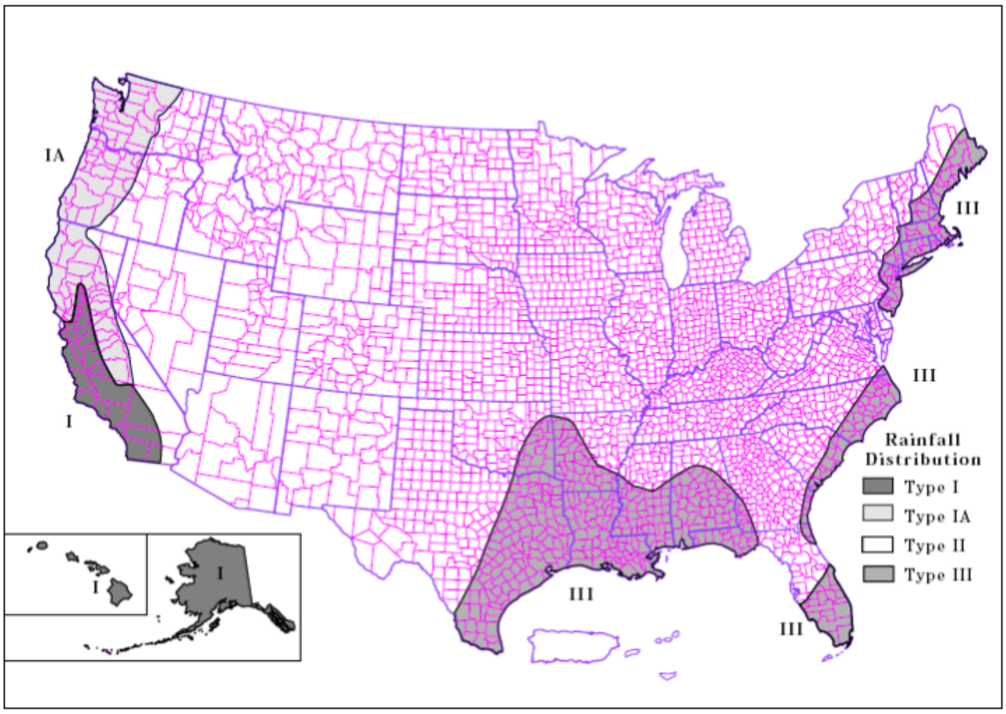


Figure 23.Reproduction of Figure B-2 in TR-55. Approximate geographic boundaries for SCS storm distributions.

5.2.10 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. A level of risk may be set by determining the acceptable risk of a project failing during its design life. For example, it may be decided that it is tolerable for there to be a 0.05 probability that the project fails during its 50-year design life. Conversely, many older projects were designed to provide protection from flooding should a selected large historical event re-occur. Such historical adequacy methods are not generally used in current design. Instead, an acceptable failure risk is determined for a project. Alternately, agencies or companies that operate multiple projects may set a risk tolerance for any project in the managed portfolio to fail in a given year.

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 timing 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 location of interest, design discharges for specified AEP can be estimated using statistical-analysis methods. In the United States, 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).

5.2.11 Selecting a Design Storm

In many cases the design storm will be required by regulation or governmental policy. When no such regulation exists, you must determine the appropriate storm for you specific application. The following guidelines can help in making the decision.

5.2.11.1 Exceedance Probability

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.

5.2.11.2 Storm Duration

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.

5.2.11.3 Storm Temporal Pattern

Should a frequency-based hypothetical storm temporal distribution be used, 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.

5.2.12 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.

5.2.13 Precipitation References

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5.3 Shortwave Radiation

Shortwave Radiation is a radiant energy produced by the sun with wavelengths ranging from infrared through visible to ultraviolet. Shortwave radiation is therefore exclusively associated with daylight hours for a particular location on the Earth's surface. The energy arrives at the top of the Earth's atmosphere with a flux (Watts per square meter) that varies very little during the year and between years. Consequently, the flux is usually taken as a constant for hydrologic simulation purposes. Some of the incoming radiation is reflected by the top of the atmosphere and some is reflected by clouds. A portion of the incoming radiation is absorbed by the atmosphere and some is absorbed by clouds. The Albedo is the fraction of the shortwave radiation arriving at the land surface that is reflected back into the atmosphere. The shortwave radiation that is not reflected or absorbed above the land surface, and is not reflected by the land surface, is available to drive hydrologic processes such as evapotranspiration and snowpack melting.

5.4 Longwave Radiation

5.5 Pressure

5.6 Humidity

6 Snow Accumulation and Melt

Snowmelt is the primary focus area of snowmelt hydrology. Snowmelt is the phase change of solid ice crystals that form the snowpack into liquid water. The goals of understanding snowmelt are estimation of the timing, rate, and ultimate volume of water produced by snowmelt. Snowmelt is determined through the snowpack energy balance.

6.1 Snowmelt Basic Concepts

6.1.1 Snowpack/Snow Cover

The terms *snowpack* and *snow cover* are often used interchangeably but they do have slightly different meanings. The term *snowpack* is used when referring to the physical and mechanical properties of the snow on the ground. The term *snow cover* is used when referring to the snow accumulation on ground, and in particular, the areal extent of the snow-covered ground. This distinction will be respected for the most part. Both snowpack and snow cover refer to the total snow and ice on the ground, including both new snow and any existing un-melted snow and ice.

From the time of its deposition until melting, snow on the ground is a fascinating and unique material. Snow is a highly porous, sintered material made up of a continuous ice structure and a continuously connected pore space, forming together the snow microstructure. As the temperature of snow is almost always near its melting temperature, snow on the ground is in a continuous state of transformation, known as *metamorphism*.

At the melting temperature, liquid water may partially fill the pore space. In general, therefore, all three phases of water - ice, vapor, and liquid - can coexist in snow on the ground.

Due to the intermittent nature of precipitation, the action of wind and the continuously ongoing metamorphism of snow, distinct layers of snow build up the snowpack. However, in hydrologic applications, the snowpack is treated as a single layer and the properties of the snowpack must represent the conditions of the entire layer from the snow surface to the ground. This single layer representation is required by the practical limits of current hydrologic practice, and the currently available data on snow observed in the field. Generally, in practice, the distinct properties of individual layers of the snowpack are not known.

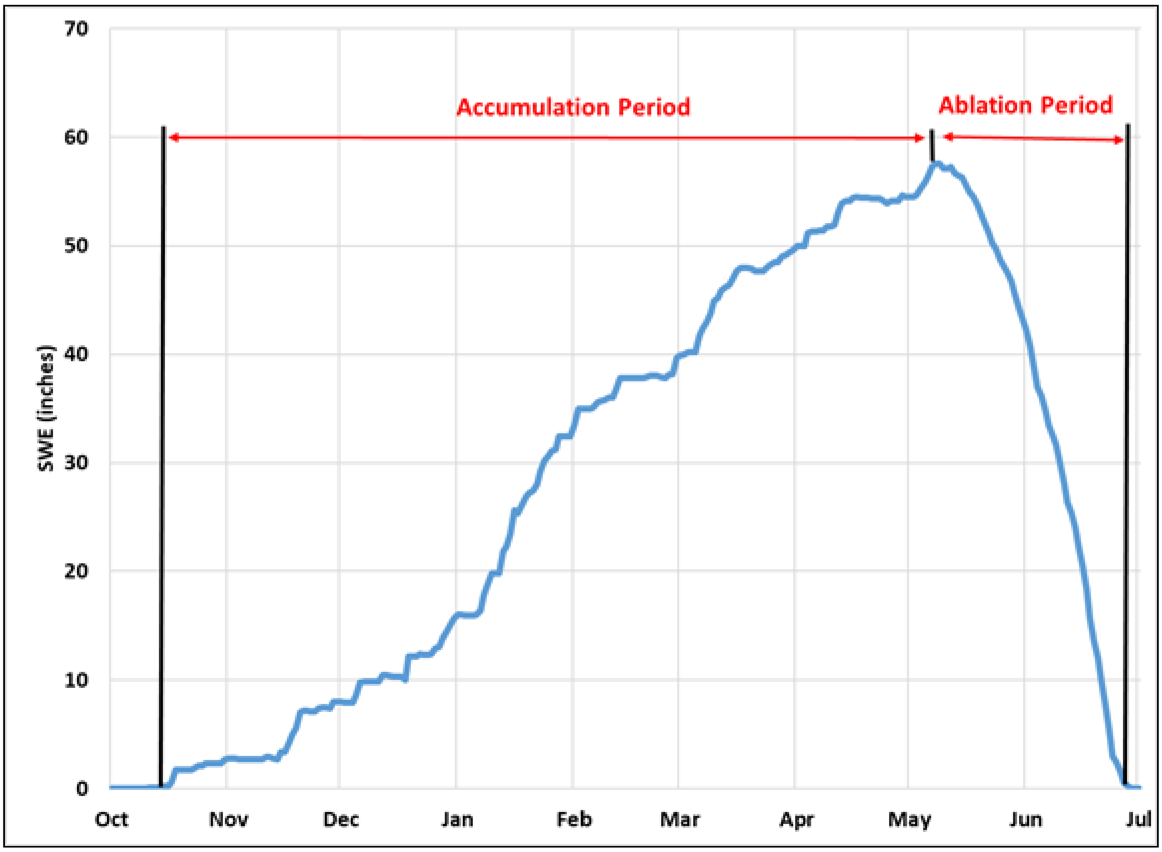
Snowpack
A laterally extensive accumulation of snow on the ground that persists through winter and melts in the spring and summer. *AMS glossary*
Snowpack
The total snow and ice on the ground, including both new snow and the previous snow and ice which have not melted. *NSIDC glossary*
Snowpack
The accumulation of snow at a given site and time; term to be preferably used in conjunction with the physical and mechanical properties of the snow on the ground. *UNESCO glossary*

Snow cover
In general, the *accumulation* of snow on the ground surface, and in particular, the areal extent of *snow-covered* ground (NSIDC, 2008); term to be preferably used in conjunction with the climatologic relevance of snow on the ground. *UNESCO glossary*

6.1.2 Seasonal Snow

In almost all cases, the HEC-HMS snowmelt model is applied to seasonal snow. *Seasonal snow* is snow that accumulates during one season and does not last for more than one year. An example of seasonal snow is shown in Figure 1. The chart on this slide displays the Snow Water Equivalent (SWE) measured on Mount Alyeska in Alaska over the winter of 2000-01. Notice that the chart starts with a near monotonic increase in SWE up to the *Annual Maximum SWE*. This is the accumulation period. After the Annual Maximum SWE the *ablation* or melting period occurs. In many cases, the accumulation period is longer than the ablation period. In this case, the accumulation lasts for 7 months and the melt for 2 months. Other sites will vary in the timing and length of the accumulation and ablation periods. Some sites may have two or more accumulation periods and the SWE may drop to zero between the periods.

Temperature gradients and sub-freezing temperatures are more likely to exist in the snowpack during the accumulation period. Uniform temperatures throughout the snowpack at 32°F (0°C) are most likely to exist during the ablation period, especially during periods of continuous snowmelt.



2 Snow Accumulation at Mt. Alyeska, Alaska for the winter season of 2000-01

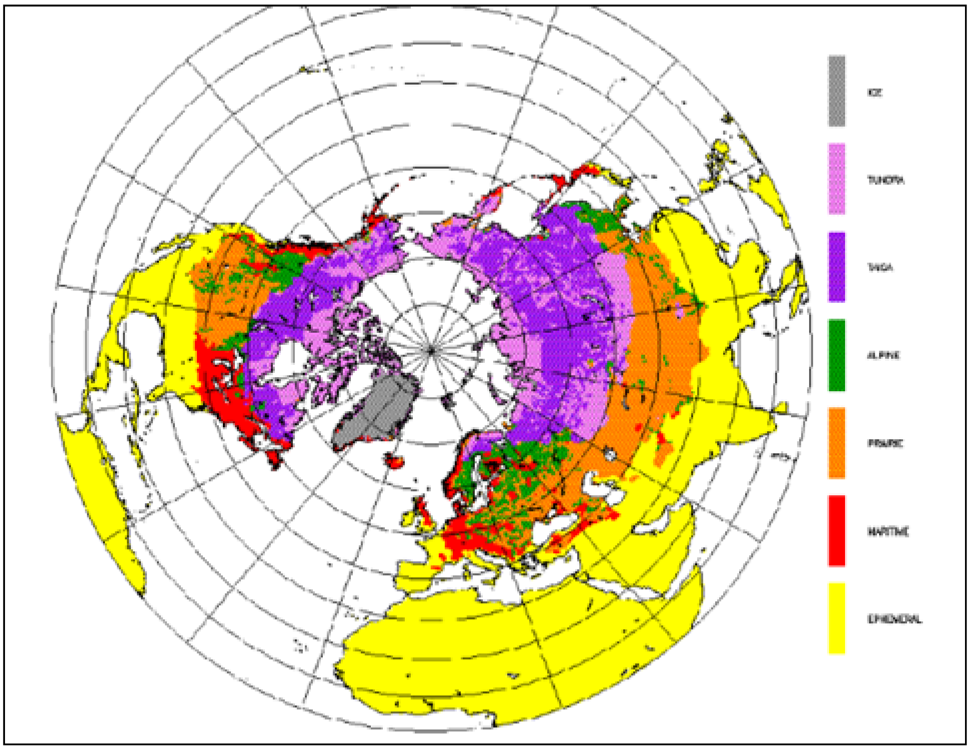
6.1.2.1 Seasonal Snow Classification

There is a tremendous amount of variability in seasonal snow. The range of seasonal snow characteristics has been catalogued and classified by Sturm et al (1995). They divided seasonal snow covers in to six classes: tundra, taiga, alpine, prairie, maritime, and ephemeral. They found it difficult to classify the snow cover of mountainous regions. Mountain snow covers have as their chief attribute a high degree of lateral or spatial variability. Wind turbulence over steep, complicated terrain and highly variable solar radiation distribution are the chief causes of this variability. So, the mountain snow class is used to flag regions of high snow variability. They found that these snow classes could be discriminated by a group of variables that they felt were relatively easy to measure. These variables are the winter average values of snow depth, air temperature, snow-ground interface temperature, and bulk density. The Table and Figure below describes the seasonal snow classifications and locations.

Class	Description	Depth (cm)	No. Layers
Tundra	A thin, cold, wind-blown snow. Max. depth approx. 75 cm. Usually found above or north of tree line. Consists of a basal layer of depth hoar overlain by multiple wind slabs.	10-75	0-6
Taiga	A thin to moderately deep low-density cold snow cover. Max. depth: 120 cm. Found in cold climates in forests where wind, initial snow density, and average winter air temperatures are all low. By late winter consists of 50% to 80% depth hoar covered by low-density new snow.	30-120	>15
Alpine	An intermediate to cold deep snow cover. Max depth approx. 250 cm. Often alternate thick and thin layers, some wind affected. Basal depth hoar common, as well as occasional wind crusts. Most new snowfalls are low density. Melt features occur but are generally insignificant.	75-250	>15
Maritime	A warm deep snow cover. Max depth can be in excess of 300 cm. Melt features (ice layers, percolation columns) very common. Coarse grained snow due to wetting ubiquitous. Basal Melting common.	75-500	>15

Class	Description	Depth (cm)	No. Layers
Ephemeral	A thin extremely warm snow cover. Ranges from 0 to 50 cm. Shortly after it is deposited, it begins melting, with basal melting common. Melt features common. Often consists of a single snowfall, which melts away, then a new snow cover reforms at the next snowfall.	0-50	1-3
Prairie	A thin, except in drifts, moderately cold snow cover with substantial wind drifting. Max. depth approx. 1 m. Wind slabs and drifts common.	0-50	<5
Mountain	A highly variable snow cover, depending on the solar radiation effects and local wind patterns. Usually deeper than associated type of snow cover from adjacent low-lands.		Variable

Basal: base (bottom) of snowpack
Hoar: Crystals that form in the snowpack after deposition. Common feature of Arctic snow
Melt Features: Evidence that melting has occurred; Could include an ice layer, percolation column, etc.
Wind slab: Both wind crusts and wind slabs are layers of small, broken or abraded, closely packed and well-sintered particles.



3 Snow Classification

6.1.2.2 Snow Metamorphism

A snowpack is not static. The ice crystals and grains that comprise the snowpack are constantly evolving and changing throughout the winter season. This process is called *snow metamorphism*. There are two major processes of snow metamorphism that can occur. The first results from the general tendency of ice crystals to change their form to a more spherical shape. The rate of this metamorphism depends on the temperature of the snowpack. The closer the pack temperature is to 0°C (32°F) the faster metamorphism will occur. It proceeds relatively rapidly when the snowpack is melting. The second process of snowpack metamorphism is driven by vertical temperature gradients in the snowpack. The temperature gradient creates a vapor gradient which effectively moves water molecules between ice crystals by vapor transport. A strong temperature gradient through the snowpack (>10°C m^{-1}) may result in the formation of new, relatively large, ice crystals within the snowpack termed *depth hoar*. This second form of metamorphism is common in the arctic but it can occur anywhere the difference between cold air temperatures and relatively warm ground temperatures are large and long lasting.

The first process of metamorphism occurs in all snowpacks. Its beginnings lay in the incredible variety of ice crystal shapes deposited on a snowpack. Something most crystal shapes share in common on reaching the snowpack is large surface-area-to-volume ratios. These large ratios are created during the rapid crystal growth that occurs in the atmosphere when snow crystals form. When the ice crystals are incorporated into the snowpack they no longer grow and tend towards their *spherical equilibrium form*. Snow metamorphism describes the change in the snow crystals and grains to less angular, more rounded forms with time. This type of metamorphism causes a gradual increase in the snowpack density, a reduction in the surface reflection of sunlight (described by the surface albedo) and changes other snowpack properties with time. Metamorphism occurs quickly when the snowpack is melting. This rapid metamorphism causes the surface albedo to decline relatively rapidly. The decline in albedo increases the shortwave radiation (sunlight) that can be absorbed by the snowpack and can increase the rate of snowmelt.

Hydrologic snowmelt models generally do not model snow metamorphism directly. Some approaches do model the changes in the snowpack density and albedo that result from metamorphism, as will be shown later.

6.1.2.3 Snow Phase Change

Phase change describes the transition between ice and liquid water, between ice and water vapor, and between liquid water and water vapor. Adding heat to snow causes the snow to become warmer until it reaches the ice/water equilibrium temperature, 0°C (32°F). (This is the value for fresh water under the normal range of atmospheric pressure.) The relationship between the amount of heat added (or removed) per unit volume of snow and rate of temperature increase (or decrease) is determined by the snow density and the *specific heat* of ice. Once the temperature of snow reaches the ice/water equilibrium temperature further addition of heat will not change the snow temperature but will cause the snow to *melt*, to change phase to liquid water. The relationship between the amount of heat added and the amount of liquid water created is determined by the *latent heat of fusion* of water.

The transfer of water molecules directly from the ice crystals to water vapor occurs by *sublimation*. Sublimation can occur over the entire temperature range at which snow exists. Sublimation removes heat from the snowpack to provide the ice molecules enough energy to escape from the snow ice crystal surface. Sublimation cools the snowpack. The relationship between the amount of ice removed by sublimation and the amount of heat removed defines the *latent heat of sublimation* of ice. The transfer of water from the vapor state to the snowpack occurs by *condensation*. Condensation adds heat to the snowpack to remove the energy of the water vapor molecules so they can join the liquid water or ice crystals that form the snowpack. Condensation warms the snowpack.

The transfer of water molecules between the snow surface and the atmosphere is one of the several modes of heat transfer between snow and the atmosphere. (The four major heat transfer modes being sensible heat transfer, latent heat transfer (evaporation and condensation), long wave radiation, and short-wave radiation.) Overall, the transfer of water is driven by the difference between the amount of water vapor per unit volume contained in a *saturated vapor layer* immediately above the snow surface and the amount of water vapor per unit volume contained in the atmosphere above. The actual rate of sublimation and condensation is controlled by the mass transfer ability of the meteorological boundary layer above the snow surface (as will be discussed below).

6.2 Snow Properties

The primary physical properties of a snowpack important to hydrology are three properties that can vary from point to point: *Snow Water Equivalent* (SWE), snowpack temperature, as represented by its *Cold Content*, and *Liquid Water Content;* and one spatial property: the *Snow Covered Area* (SCA). The point properties are applicable at a specific location at a specific time, and represent the entire single layer of the snowpack at that location. These properties will vary from location to location throughout a watershed and with time. The primary spatial property of the snow cover for hydrology is the SCA. SCA describes the area of a watershed that is snow covered. SCA often changes with time, especially during periods of snowmelt.

6.2.1 Snow Covered Area (SCA)

The Snow Covered Area (SCA, pronounced "Ska") is the portion of a given area covered by snow. In theory, the given area can be an entire watershed, a subbasin, an elevation band, a grid cell, or any other arbitrarily designated area. Generally SCA is given in terms of area, or as a fraction of the designated area.

In mountainous watersheds SCA is usually closely associated with elevation. In watersheds with low topographic relief, SCA reflects the paths of snow storms that have crossed the watershed. Determination of the SCA of a watershed may be easy or very difficult, depending on the size, accessibility, remoteness, topographic relief, and the extent of installed instrumentation. In recent decades, the development of optical and passive microwave satellite sensors has provided an extensive suite of instruments for accurately mapping snow.

6.2.2 Snow Water Equivalent (SWE)

The Snow Water Equivalent, (SWE, pronounced "sweeee") is the height of water if a snow cover is completely melted, on a corresponding horizontal surface area. The continuity equation equating the mass of liquid water in SWE with the mass snow in the snowpack is

$$\rho_w SWE = \overline{\rho}_S D$$

where ρ_w = the density of water; *SWE* = the snow water equivalent; $\overline{\rho}_S$ = the depth averaged snow density; and *D* = the snow depth. Snow depth denotes the total height of the snowpack, i.e., the vertical distance from the ground to the snow surface. Unless otherwise specified, *SWE* and snow depth are related to a single location at a given time. Snow density, i.e., mass per unit volume, is normally determined by weighing snow of a known volume. Theoretically, total snow density encompasses all constituents of the snowpack - ice, liquid water, vapor, and air. In practice, only the ice and liquid water are included in estimates of SWE as the air and water vapor make only a negligible contribution to the density. Rearranging equation

$$SWE = \frac{\overline{\rho}_s}{\rho_w} D$$

As shown in equation , if *D* and are both known, then *SWE* can be estimated (as the density of water is always known). It is also clear that *D* alone is not sufficient to estimate *SWE* unless some estimate of can be made. In fact, snow depth is a parameter that is of little use in snowmelt hydrology, except when used to estimate *SWE*.

Snow Type	Density (kg/m3)	Snow Depth for One Inch Water
Wild Snow	10 to 30	33" to 98"
Ordinary new snow immediately after falling in still air	50 to 65	20" to 15"

Snow Type	Density (kg/m3)	Snow Depth for One Inch Water
Settling Snow	70 to 90	11” to 14”
Average wind-toughened snow	280	3.5”
Hard wind slab	350	2.8”

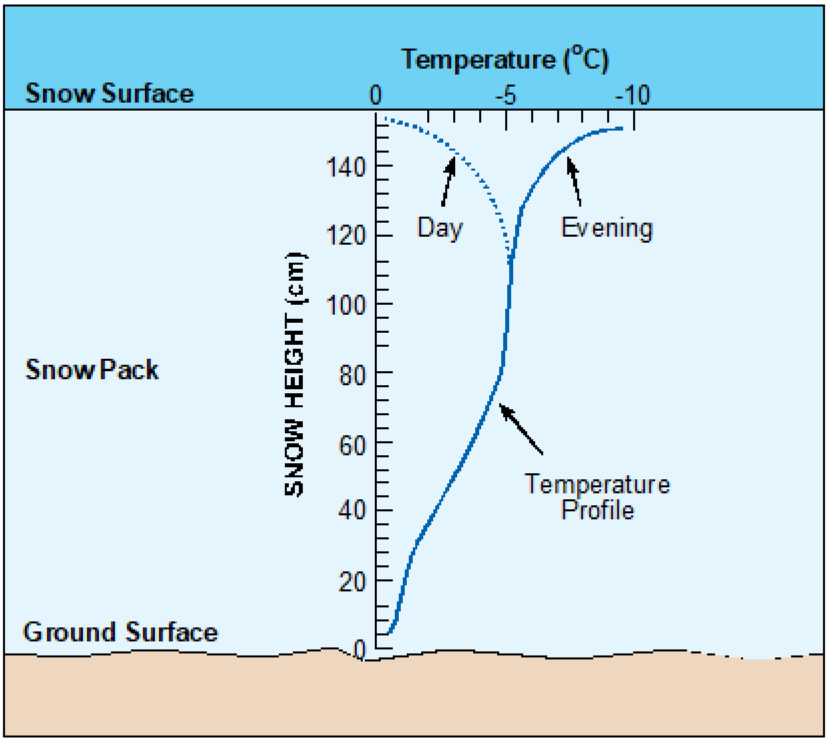
6.2.3 Snowpack Temperature/Cold Content

The snowpack temperature is controlled by heat transfer through the snow surface to the atmosphere, heat transfer through the snowpack base to the ground below, and the temperature of the falling snow deposited in the snowpack. The trend of the snowpack temperature is to follow the air temperature as long as the air temperature is below 0°C (32°F). The actual vertical temperature profile through the snowpack will depend on the snowpack surface temperature, the base temperature, the snowpack depth, and the thermal conductivity of the snow in the pack.

Phase Change of Snow

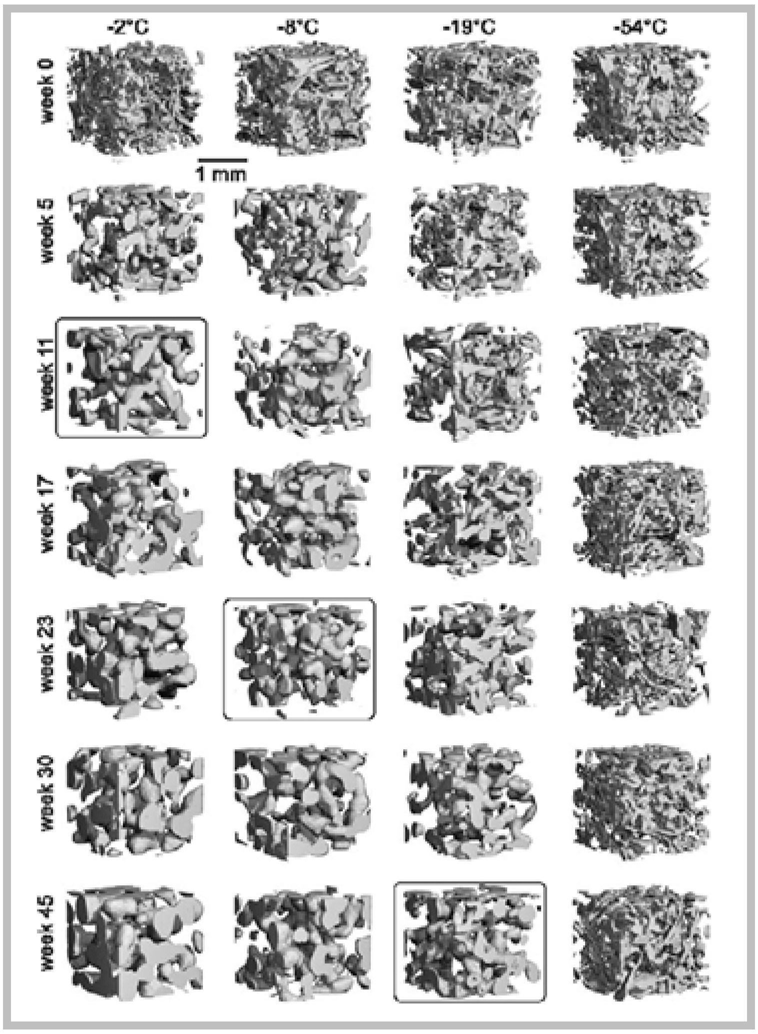
While 0°C (32°F) is often called the melting temperature of ice, ice happily survives at 0°C (32°F). It is only when heat is transferred into ice which is at 0°C (32°F) that melting, the phase change from solid ice to liquid water, occurs. Similarly, liquid water also happily survives at 0°C (32°F). It is only when heat is transferred away from liquid water which is at 0°C (32°F) that freezing, the phase change from liquid water to solid ice, occurs. If ice and liquid water are coexisting in a given volume, their masses are not changing with time, and there is no heat transfer into or out of the volume, then their temperature must be 0°C (32°F).

In many cases, during the accumulation phase of the winter season, a difference in temperature exists between the top surface and the base of the snowpack. This *temperature gradient* may be large or small depending on the conditions. Small temperature gradients occur with deep snow and moderately cold temperatures. Large gradients tend to occur in shallow snowpacks with very cold air temperatures.



4 Temperature gradient across the snowpack

Large gradients can drive heat and water vapor from the warmer portions of the snowpack (generally the base) to the colder portions (generally the surface). This heat and mass flux can lead to rapid *metamorphism*, causing changes in the ice crystal size and shape.



5 Changes in snow morphology over time

The snowpack temperature cannot be greater than 0°C (32°F), the temperature at which ice and water coexist. If the snowpack temperature is less than 0°C (32°F) then liquid water cannot exist for any length of time in the snowpack. If the snowpack temperature is at 0°C (32°F), then liquid water can exist in the snowpack. The snowpack is at 0°C (32°F) and *isothermal*, with uniform temperature from surface to base (zero temperature gradient), during the active melt period.

The concept of “Cold Content” grew directly out of attempts to predict snowmelt using the Temperature Index approach. Snowmelt cannot occur unless the snowpack temperature is at 0°C (32°F). Cold content is the heat necessary to warm the snowpack up to 0°C (32°F) in terms of the amount of ice this heat would melt.

The heat required per unit area to raise temperature of the snowpack to 32°F (0°C), H_{c_heat}, is

$$H_{c\_\text{heat}} = \bar{\rho}_s C_{p\_\text{ice}} D\Delta T$$

where $\overline{\rho_s}$ = the depth averaged snow density; C_{p_ice} = the heat capacity of ice; D = the snow depth; and ΔT = the temperature below 0°C (32°F).

Parameter	Definition	SI	Customary
$C_{p\ ice}$	Heat Capacity ice	2114 J/kg °C	0.505 Btu/lb °F
λ	latent heat of fusion of water	333.4 k J/Kg	143.3 Btu/lb
ρ_w	Density of water (0°C- 32°F)	999.87 kg/m^3	62.42 lb/ft^3

In practice, the flow of heat is not estimated during snowmelt modeling when using the Temperature Index approach. Rather (as will be shown in the section on modeling below), the snowpack temperature is more conveniently described by expressing the Cold Content as the negative of the depth of frozen water that H_{c_heat} would melt.

$$C_c = -\frac{H_{c\_\text{heat}}}{\rho_w \lambda} = -\frac{\bar{\rho}_s}{\rho_w} D \frac{C_{p\ \_\text{ice}} \Delta T}{\lambda}$$

where λ = the latent heat of fusion of water; and C_c = the Cold Content in units of negative inches (or cm).

6.2.4 Liquid Water Content

Liquid Water Content (*LWC*) describes the amount of liquid water contained in the snowpack. Liquid water in the snowpack results from ice melting or by rain falling on the snow surface and can only permanently exist in the snowpack when the snowpack temperature is at 0°C (32°F). Another, and equivalent, way of stating this is: liquid water can exist in the snowpack only if the Cold Content is zero.

There is a limit to the amount of liquid water that a snowpack can hold, described by the percentage, $LWC_{max\%}$, of the *SWE*. Further liquid water added to the snowpack once the percentage of *LWC* in the snowpack has reached $LWC_{max\%}$ leads to the downward vertical movement of the liquid water through the snowpack. Once the liquid water reaches the base of the snowpack it becomes *snowmelt runoff*. Another term for snowmelt runoff is Liquid Water at the Soil Surface (LWASS).

A "ripe" snowpack has C_c = 0; and the percentage of *LWC* in the snowpack is $LWC_{max\%}$. Any heat added to a ripe snowpack will result in snowmelt runoff.

The downward vertical movement of liquid water through the snowpack is an interesting physical process. In most cases, the snowpack is not saturated with water. Air can move freely throughout the snowpack and the air pressure is uniform throughout. The effects of capillarity are generally regarded as negligibly small compared to the effects of gravity, and the inertial forces are small compared to viscous forces. An analysis of flow under these conditions balances the downward pull of gravity on the water and the fluid viscous forces. The movement of liquid water occurs at very low Reynolds number and can be described as *creeping* or *Stokes flow*. The result is a simplified form of Darcy's equation. The effective permeability of the snowpack - a key parameter of Darcy's equation - has only been roughly measured. Observations suggest that the permeability is proportional to the third power of the effective saturation. This strong nonlinearity greatly complicates the solution of the flow equation.

A number of factors can impact the movement of liquid water through the snowpack, including non-uniform temperatures with depth, non-homogeneity of the crystal and void sizes, formation of *ice lenses*, etc.

Nonetheless, once the liquid water has reached the base of the snowpack and snowmelt runoff has commenced, further additions of liquid water move through the snowpack relatively rapidly. So rapidly, in fact, that the any delay in the appearance of snowmelt runoff caused by the vertical movement of water through the snowpack is not considered hydrologically significant. As a result, once the models determine that the conditions are appropriate for snowmelt runoff (C_c = 0 and LWC = LWC_{max}), snowmelt runoff will occur without delay.

Term	Description	Range %	Mean %
Dry	Usually *T*s is below 0°C, but dry snow can occur at any temperature up to 0°C. Disaggregated snow grains have little tendency to adhere to each other when pressed together, as in making a snowball.	0	0
moist	*T*s = 0°C. The water is not visible even at 10x magnification. When lightly crushed, the snow has a distinct tendency to stick together.	0–3	1.5
wet	*T*s = 0°C. The water can be recognized at 10x magnification by its meniscus between adjacent snow grains, but water cannot be pressed out by moderately squeezing the snow in the hands (pendular regime).	3–8	5.5
very wet	*T*s = 0°C. The water can be pressed out by moderately squeezing the snow in the hands, but an appreciable amount of air is confined within the pores (funicular regime).	8–15	11.5
Soaked or slush	*T*s = 0°C. The snow is soaked with water and contains a volume fraction of air from 20 to 40% (funicular regime).	>15	>15

Pendular regime (of water) The condition of low liquid water content where a continuous air space as well as discontinuous volumes of water coexist in a snowpack, i.e., air-ice, water-ice, and air-liquid interfaces are all found. Grain-to-grain bonds give strength. The volume fraction of free water does not exceed 8 %.
Funicular regime (of water) The condition of high liquid water content where liquid exists in continuous paths covering the ice structure; grain-to-grain bonds are weak. The volume fraction of free water exceeds 8 %.

6.3 Snowpack Mass and Energy Accounting

The two primary requirements of snowmelt hydrology are accounting for the time varying and spatially varying snowpack SWE and energy. The snowpack SWE is accounted for by *balancing* changes in SWE with the SWE deposited on or removed from the snow surface and the SWE lost from the snowpack as liquid water *runoff.* The snowpack energy is accounted for by *balancing* changes in the snowpack temperature and phase change with the heat flux at the snowpack top and bottom surfaces.

6.3.1 Snowpack Mass Balance

The snowpack mass balance describes changes in the snowpack SWE as a result of precipitation, runoff, sublimation, and blowing snow:

$$\frac{d\,(SWE_t)}{dt} = P_t - R_t \pm V_t \pm B_t$$

where SWE_t = the SWE of the snowpack (depth); t = time; P_t = the precipitation rate (depth/time); R_t = the runoff rate (depth/time); V_t = mass gained from or lost to water vapor (depth/time); and B_t = the snow gained from or lost to blowing snow (depth/time). (Variables with the subscript *t* are *time varying*.) The precipitation can be in the form of snow or rain – both will increase the SWE. The runoff rate (R_t) is determined by phase change in the snowpack which is determined through the energy balance calculations.

In general, the principal mass input into the snowpack is precipitation in the form of snow, which increases the snowpack SWE during the accumulation period; and the principal mass loss is liquid water runoff, which decreases the SWE during the ablation period. These are the processes modeled by the HEC-HMS Temperature Index Snow Model. However, mass lost to water vapor and blowing snow erosion and deposition can also be important at some locations.

Sublimation and condensation (V_t) describe the phase change of ice crystals of the snowpack directly into water vapor or the phase change of water vapor into ice or liquid water in the snowpack. Sublimation and condensation occur when heat is transferred between the snowpack and the atmosphere through the latent heat flux. (More about this in the heat transfer section below.) Blowing snow and sublimation can play significant roles in tundra, along alpine ridges, and other areas where strong winds and low humidity are common. In any case, sublimation, condensation, and blowing snow are not by the HEC-HMS Temperature Index Snow Model. As a result, the snowpack mass balance can be simplified to

$$\frac{d\,(SWE_t)}{dt} = P_t - R_t$$

6.3.2 Snowpack Energy Balance

The snowpack energy balance describes changes in the snowpack temperature and phase change in the snowpack as a result of heat transfer at the top and bottom surfaces of the snowpack. The snowpack energy balance is done per unit area of the snowpack surface. The snowpack is treated as a single layer with a uniform temperature. All heat transfer is vertical and there is no horizontal heat transfer within the snowpack.

If <0°C (32°F) the energy balance is

$$C_{p\_ie} \frac{d\left(\bar{\rho}_s D\bar{T}\right)}{dt} = Q_{\text{bta}}$$

where Q_{total} = the net heat flux at the snow surface (units of joules per second per unit area). (Q_{total} > 0, implies a net heat flow into the snowpack). Note that equation includes the snow depth and density inside the differentiation as the depth and density can also change with time. In general, the snow depth will only change with time under these conditions (<0°C (32°F)) if there is snowfall occurring. This will be discussed in more detail below. Assume that the snow depth and density are constant with time as is usual under these conditions. Equation is then written as

$$C_{p\_ice} \bar{\rho}_s D \frac{d\bar{T}}{dt} = C_{p\_ice} \rho_w SWE \frac{d\bar{T}}{dt} = Q_{\text{total}}$$

Equation can be re-written in terms of the cold constant as

$$\frac{dC_c}{dt} = \frac{Q_{\text{total}}}{\rho_w \lambda}$$

where C_c = the Cold Content of the snowpack, as defined in equation . Note that equation and equation are *entirely equivalent*.

If =0°C (32°F), which is equivalent to C_c =0, and the net heat transfer is into the snowpack, that is, Q_{total} > 0, then the conditions are set for phase change, melting, to occur. The rate liquid water is formed from melting ice, M_t (depth/time), is

$$M_t = \frac{Q_{total}}{\rho_w \lambda}$$

If the ratio $LWC/SWE \leq LWC_{max\%}$ then

$$\frac{dLWC}{dt} = M_t$$
$$R_t = 0$$

If the percentage of the Liquid Water Content of the snowpack is less than $LWC_{max\%}$ then the liquid water created by melting snow increases the Liquid Water Content of the snowpack and no runoff occurs.

If LWC/SWE = $LWC_{max\%}$ then

$$R_t = M_t$$
$$\frac{d(SWE)}{dt} = -R_t$$
$$\frac{dLWC}{dt} = \frac{LWC_{\max\%}}{100.} \frac{dSWE}{dt}$$

Once the percentage of the Liquid Water Content of the snowpack is equal to $LWC_{max\%}$ then all liquid water formed goes into runoff, which reduces the *SWE* and the *LWC*. This process can continue until the SWE is zero.

There are a number of different modes of heat transfer that are included in Q_{total}. These will be discussed next.

6.3.3 Snow Surface Heat Transfer

6.3.3.1 Overview

Snowmelt is ultimately driven by the transfer of heat energy into the snowpack from the atmosphere and the surrounding environment. There are four primary modes of heat transfer between the snowpack and its environment: sensible heat transfer, latent heat transfer, long wave radiation heat transfer, and short-wave radiation heat transfer. There is also heat transfer from precipitation falling on the snow surface as rain or snow, and heat transfer from the soil layer beneath the snowpack. This can be stated as

$$Q_{\text{total}} = Q_{\text{latent}} + Q_{\text{sensible}} + (1-\alpha)Q_{\text{SW}\downarrow} + Q_{\text{LWnet}} + Q_{\text{Precipitation}} + Q_{\text{Ground}}$$

where Q_{Latent} = the rate of latent heat transfer; $Q_{sensible}$ = the rate of sensible heat transfer; $Q_{SWâ}$ = the rate of downwelling shortwave radiation at the snow surface; α = the snowpack albedo (0< α <1); Q_{LWnet} = the rate of net longwave heat transfer; $Q_{Precipitation}$ =the rate of heat transfer due to rain and snow falling on the snow surface; and Q_{Ground} = the rate of heat transfer from the ground beneath the snowpack.

It is important to note that HEC-HMS Temperature Index Snow Model does not model the individual modes of heat transfer included in equation explicitly. It uses a temperature index approach to simplify and model the net heat transfer instead. However, it is important to have insight into the physics of each mode of heat transfer. This is provided next.

6.3.3.2 Sensible and Latent Heat Transfer

Sensible and latent heat transfer are discussed together because both are driven by turbulent transport in the air between the snow surface and the lower part of the atmosphere. In fact, sensible and latent heat transfer are often referred to as *turbulent fluxes* due to the fact that they occur largely through turbulent transport in the air.

Sensible heat is the heat energy that can be measured with a thermometer. The direction of sensible heat transfer is always from the warmer object to the colder. So, for heat to be transferred *into the snowpack from the atmosphere* the air temperature must be warmer than the snowpack surface temperature. The heat transferred into the snowpack will cause the pack temperature to rise until the pack temperature reaches the equilibrium temperature of 0°C (32°F). Additional heat transfer into the snowpack after this point causes snowmelt. In similar fashion, for heat to be transferred *from the snowpack into the atmosphere*, the air temperature must be colder than the snowpack surface temperature. In this case, the snowpack temperature will drop. If the snowpack surface temperature and the air temperature are the same there will be no heat transfer between them as sensible heat.

Latent heat transfer is due to the transfer of water vapor between the snowpack surface and the atmosphere. Latent heat transfer always involves phases change between the water in the form of ice in the snowpack and the water in the form of vapor in the atmosphere. The transfer of water molecules directly from snow to vapor occurs by *sublimation.* Sublimation absorbs heat energy from the snowpack and cools it off. The transfer of water molecules directly from vapor to ice occurs by *condensation* (sometimes also referred to as deposition.) Condensation releases heat energy into the snowpack. The direction of latent heat transfer is determined by the difference between the *water vapor pressure* of the air immediately above the snowpack and the water vapor pressure in the atmosphere above the snowpack. The direction is from the higher vapor pressure layer to the lower vapor pressure layer. The water vapor pressure of the air immediately above the snowpack is the *saturated vapor pressure* at the temperature of the snow surface. Saturated vapor pressure is largely determined by air temperature and to a much less degree the atmospheric air pressure. Saturated vapor pressure means that the water contained in the air is in equilibrium with the ice of the snowpack and is the maximum amount of water that can be contained in the air at that temperature and atmospheric pressure immediately above ice. The vapor pressure of the atmosphere above the snowpack is determined by the relative humidity of the air and the air temperature.

The latent heat flux is from the air into the snowpack (condensation occurs) when

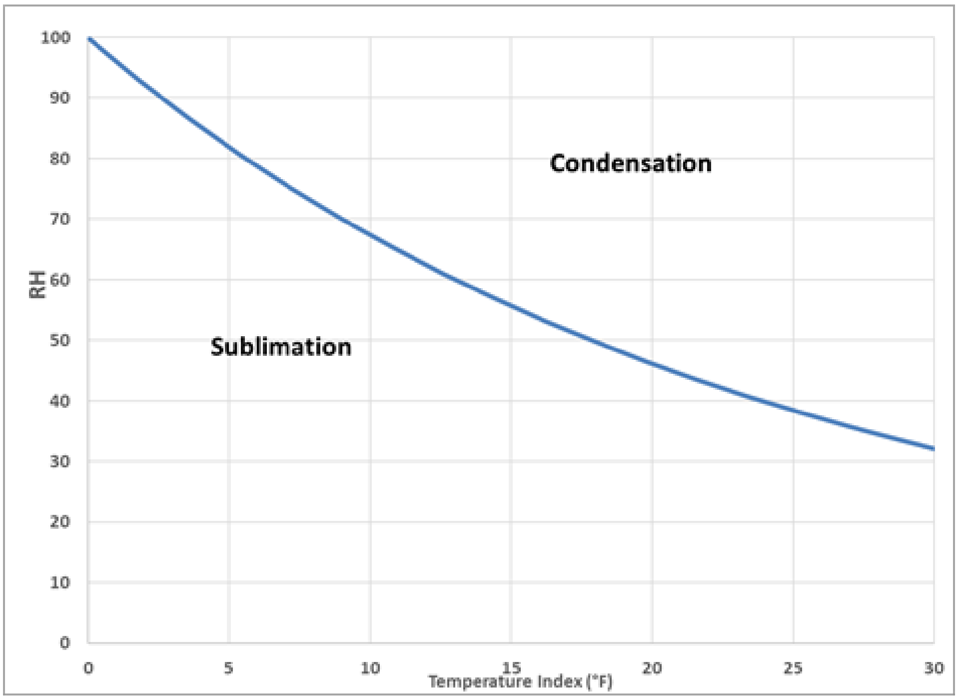
$$e_a > e_{sat(T_{ss})}$$

where e_a = the vapor pressure of the air; and $e_{sat(Tss)}$ = the saturated vapor pressure immediately above the snow surface at the snow surface temperature, T_{ss}. The equation can be cast in terms of the relative humidity of the air (which is often known) by noting that the definition of relative humidity, *RH*,

$$RH = 100\frac{e_a}{e_{sat(T_a)}}$$

where $e_{sat(Ta)}$ = the saturation vapor pressure of the air at the air temperature T_a. Substituting this into the previous equation, it can be seen that the latent heat transfer is the from air into the snowpack when

where RH_{nuet} = the neutral relative humidity at which no latent heat flux occurs. During the ablation period, the snow surface temperature is 0°C (32°F) and the saturated vapor pressure above the snowpack, $e_{sat(Tss)}$ can be calculated directly. The neutral relative humidity under these conditions can be estimated as a function of the air temperature index (the degrees that the air temperature is above 32°F) and is shown in Figure below.



It can be seen in Figure that sublimation of the snowpack will not be an uncommon occurrence during the ablation period, even when the air temperature is 10°F above freezing or more. During the ablation period, the sensible and latent heat fluxes will often be in different directions: the sensible heat flux into the snowpack and the latent heat flux out of the snowpack. The latent heat flux will be into the snowpack generally only during periods of high relative humidity, for example rain on snow events, fog, and other periods when very moist air is present.

The rates of sensible and latent heat transfer are both determined by the degree of turbulence in the atmosphere above the snow and the stability of the atmosphere. The primary way by which turbulence is generated is by wind drag over the snow and ground surface. The rate of the turbulent energy generation is very sensitive to the velocity of the wind. The ability of wind to increase the heat transfer rate is called *convection*. However, wind is not the only creator of turbulence in the atmosphere – *natural convection* of sensible and latent heat from the snow and ground surface can also create turbulence. Natural convection occurs when the density of the air immediately at the snow surface is less than the density of the air above. This difference in density causes the air to rise vertically upwards though buoyancy. The atmosphere is said to *unstable* when natural convection occurs. *Mixed convection* occurs when wind convection is augmented by natural convection. A contrasting case occurs when the atmosphere is *stable* – that is the air near the ground is denser than the air above. Under stable conditions, natural convection does not occur and the wind convection is damped. Under very stable conditions convection may not occur at all if the wind velocity is low, and sensible and latent heat transfer can drop to very low levels.

The rates of sensible and latent heat transfer can be calculated using formulas such as

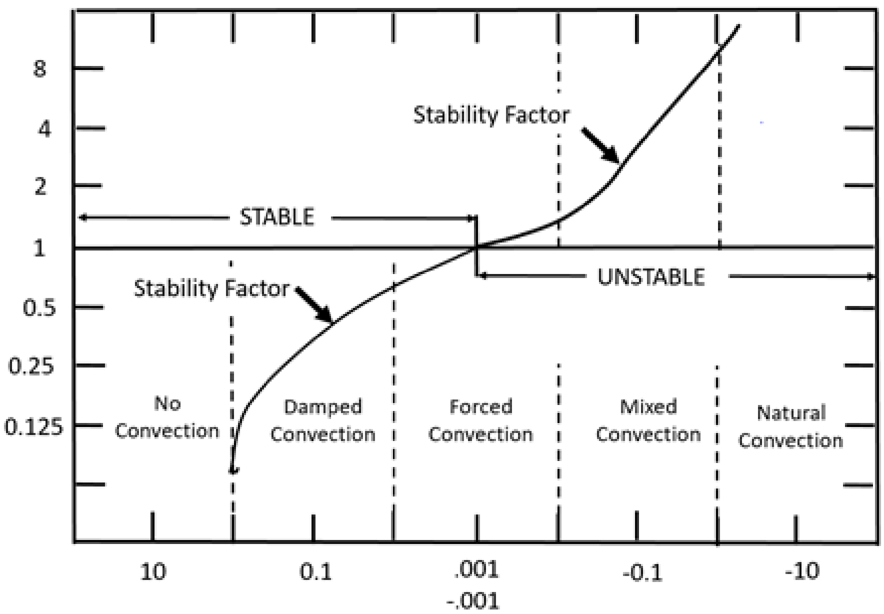
$$Q_{\text{Latent}} = \rho_a L_v C_L U_Z \left(e_a - e_{\text{sat}(T_{\text{snow}})}\right)$$
$$Q_{\text{sensible}} = \rho_a c_{pa} C_s U_Z \left(T_a - T_{\text{snow}}\right)$$

Where Q_{Latent} = the rate of latent heat transfer; $Q_{sensible}$ = the rate of sensible heat transfer; ρ_a = the density of air; L_v = the latent heat of sublimation; C_{pa} = the heat capacity of air; U_Z = the wind speed measured at an elevation Z; e_a = the water vapor pressure in the atmosphere; $e_{sat(Tsnow)}$ = the saturated vapor pressure immediately above the snow surface; T_a = the air temperature; and T_{snow} = the snow surface temperature. C_L

and C_S are *stability factors* that account for the influence of the stability of the atmosphere on the rates of sensible and latent heat transfer. There are a variety of ways of estimating the stability factors, but often they are presented as functions of the dimensionless bulk Richardson Number (*Ri*). *Ri* is calculated as

$$Ri = \frac{gZ\,(T_a - T_{\text{snow}}\ )}{0.5\,(T_a + T_{\text{snow}}\ )\,U_Z^2}$$

where *g* = the acceleration of gravity. When *Ri* >0, (that is, $T_a > T_{snow}$) the conditions are stable, and the stability factor tends to be small, indicating that sensible and latent heat transfer rates are small. If *Ri* >>0 then any convection is effectively damped and the transfer rates drop to near zero. When *Ri* = 0 ($T_a = T_{snow}$), the conditions are neutral, and the heat transfer is controlled by forced convection. When *Ri* < 0 ($T_a < T_{snow}$), the conditions are unstable, and the stability factor tends to be large, indicating that the sensible and latent heat transfer rates are greater due to augmentation of forced convection by natural convection. Representative values of the stability factors are shown in Figure below.



During the ablation period, the snow surface temperature is 0°C (32°F) and the air temperature is generally greater than 0°C (32°F) which means that $T_a > T_{snow}$ and *Ri* > 0 and the atmosphere is stable. The atmosphere becomes more and more stable as the air temperature increases relative to the snow surface temperature. This means that the turbulent fluxes of latent and sensible heat are effectively suppressed by the increasing stability of the air during the ablation period unless there is a significant wind velocity.

6.3.3.3 Longwave Radiation Heat Transfer

All bodies possessing energy emit electromagnetic radiation (and this is always true if their temperature is above absolute zero). The amount of radiation emitted by a body at each wavelength of the electromagnetic spectrum is determined by the temperature of the surface of the body according to Planck’s Law. (Longwave radiation of bodies with surface temperatures in the range of temperatures found near the earth’s surface typically emit radiation at wavelengths between 4 and 50 μm.) The total longwave radiation emitted by a body per unit area can be calculated by integrating over all frequencies to arrive at the Stefan-Boltzmann Law

$$Q_{LW} = \varepsilon T_S^4$$

where Q_{LW} = the longwave radiation emitted per unit time per unit area; σ = the Stefan-Boltzmann constant (5.67 x 10^{-8} W m^{-2} °K^{-1}); T_s = the surface temperature in degrees Kelvin; and ε = the emissivity of the surface (emissivity is between 0 and 1. If a body is a "perfect" emitter of radiation, ε = 1. In fact, many bodies, such as snow and vegetation are close to being perfect emitters).

Longwave radiation is emitted by the snow surface according to the Stefan-Boltzmann Law. This is known as *upwelling* (or outgoing) radiation. The upwelling longwave radiation emitted by the snow surface is energy lost from the snow that cools the snow. The longwave radiation emitted by the snow surface is found as

$$Q_{LW} \uparrow = \varepsilon T_S^4$$

where ε_s = the emissivity of the snow surface (accepted values range from .97 to 1.0); and T_{snow} = the temperature of the snow surface (°K).

There is also *downwelling* (or incoming radiation) emitted from the atmosphere itself and by vegetation and structures in the vicinity. The downwelling radiation absorbed by the snow surface is energy gained by the snow that warms the snow. The overall impact of longwave radiation is found by summing the downwelling and upwelling longwave radiation at the surface:

$$Q_{LWnet} = Q_{LW\uparrow} + Q_{LW\downarrow}$$

where Q_{LWnet} = the net longwave at the snow surface; and $Q_{LWâ}$ = the downwelling longwave radiation. The longwave radiation emitted by the atmosphere that reaches the snow surface is

$$Q_{LWa\downarrow} = \varepsilon_{a(e_a,T_a,clf)} \sigma T_a^4$$

where ε_a = the emissivity of the atmosphere. ε_a is affected by the vapor pressure (e_a), air temperature (T_a), and cloud cover (*clf*). The cloud cover is parameterized by the sky *cloud fraction*, with *clf* = 1 for a complete cloud cover, *clf* = 0 clear skies. ε_a can be estimated by a variety of formulas. A representative formula is

$$\varepsilon_a (clf + (1 - clf)\varepsilon_{cl})$$

where ε_{cl} = the clear-sky emissivity. Note that as *clf* increases from 0 to 1, ε_a proportionally increases between the clear sky value, ε_{cl} and the limiting value of 1 for a completely cloud covered sky. The clear-sky emissivity is often estimated as

$$\varepsilon_{cl} = 0.68 + 0.036\sqrt{e_a}$$

The equation for the downwelling longwave radiation from the atmosphere can be found by combining the above equations as

$$Q_{LWa\downarrow} = \left(clf + (1 - clf)\left(0.68 + 0.036\sqrt{e_a}\right)\right)\sigma T_a^4$$

The downwelling longwave radiation from the atmosphere is relatively small during cold, clear periods with low humidity (*clf* = 0, e_a ~0, ε_{cl} ~0.68). It is relatively large during warm cloud covered periods (*clf* = 1, ε_{cl} =1).

Downwelling longwave radiation can also be emitted by the vegetative canopies above the snow surface. The total downwelling radiation can be described as the sum of the radiation from the sky and the vegetative canopies as

$$Q_{LW\downarrow} = S_{vf} Q_{LWa\downarrow} + \left(1 - S_{vf}\right) Q_{LW\downarrow}$$

where $Q_{LWvâ}$ = the longwave radiation emitted by the canopy. The vegetative canopy is parameterized by the *sky view factor*, S_{vf}, with S_{vf}, = 1 if there is no vegetative canopy above the snow, and S_{vf} = 0 if the view of the sky is completely blocked by the canopy. The longwave radiation emitted by the canopy is estimated as

$$Q_{LW\downarrow} = \sigma T_a^4$$

The branches, stems, leaves, and other components of the vegetative canopy are generally assumed to have an emissivity of 1 and their temperature assumed to be equal to the air temperature.

The net longwave radiation at the snow surface can now be written as

$$Q_{LWnet} = S_{vf} Q_{LWa\downarrow} + \left(1 - S_{vf}\right) Q_{LW\downarrow} - Q_{LW\uparrow}$$

Note the upwelling longwave radiation emitted by the snow surface has been given the opposite sign of the downwelling radiation. Expanding this equation

$$Q_{LW_{net}} = S_{vf} \varepsilon_a \sigma T_a^4 + \left(1 - S_{vf}\right) \sigma T_a^4 - \varepsilon_s \sigma T_{snow}^4$$

6.3.3.4 Shortwave Radiation Heat Transfer

Shortwave radiation is radiation produced by the sun that reaches the surface of the earth. The sun, like all bodies, emits radiation determined by the temperature of the surface of the body according to Planck's Law. The intense temperature of the sun (about 10,000°F or 5,500°C) produces the majority of its radiation in the wavelengths from 0.295 to 2.85 µm. The human eye is sensitive to wavelengths between about 0.390 to 0.700 µm. It is the shortwave radiation from the sun that the human eye uses to see.

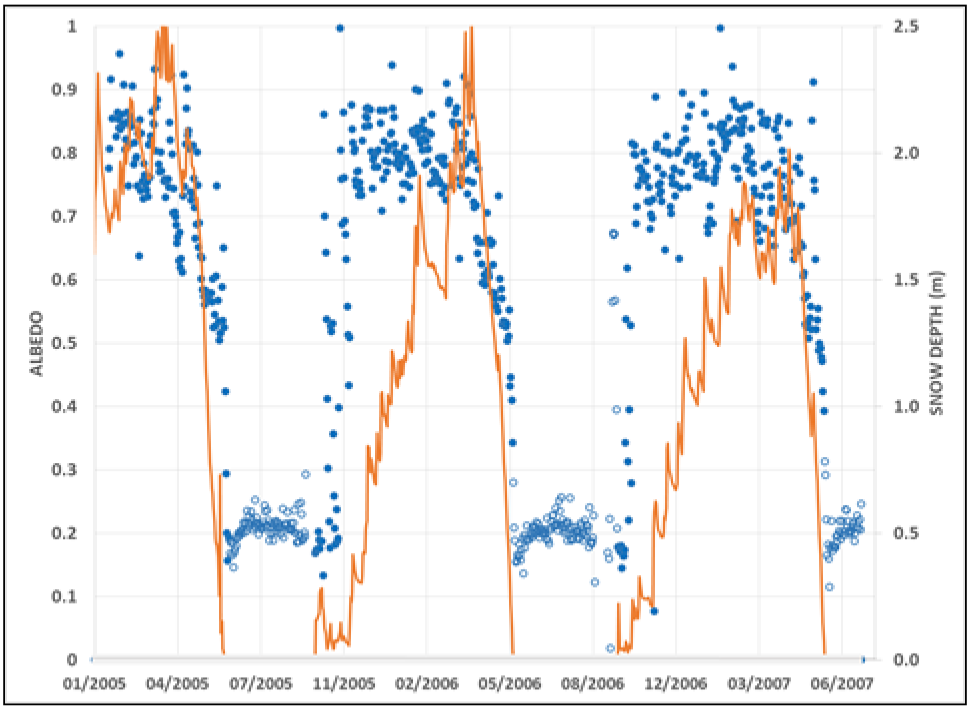
In the context of snow hydrology, shortwave radiation is not approached wavelength by wavelength but rather in terms of the *broadband* value of downwelling shortwave radiation. The broadband value of shortwave radiation is found through a weighted integration of the downwelling shortwave radiation at each wavelength that comprises shortwave radiation. When shortwave radiation reaches the snow surface a portion is reflected and a portion absorbed. It is only the absorbed portion that can change the snowpack temperature and/or cause snowmelt. The ability of snow to reflect shortwave radiation is determined by the *albedo* of the snow, α. The energy absorbed by the snow is

$$Q_{SW} = (1 - \alpha) Q_{SW\downarrow}$$

where Q_{SW} = the shortwave radiation absorbed at the snow surface; $Q_{SW\text{â}}$ = the downwelling shortwave radiation that reaches the snow surface; and α = the albedo of the snow (0< α <1)

6.3.3.4.1 Albedo.

Albedo is the ratio of the reflected shortwave radiation to the downwelling shortwave radiation reaching the surface. It is well known that the albedo of snow varies wavelength by wavelength. However, in snow hydrology, it is generally the *broadband albedo* that is of interest. The broadband albedo is found through a weighted integration of the albedo at each wavelength that comprises shortwave radiation. The albedo is determined by the crystalline structure of the snowpack surface. Shortwave radiation tends to be reflected by the surface of ice crystals and absorbed in the interior of crystals. As mentioned above, newly fallen snow typically has large surface areas to volume ratios. As a result, the albedo of newly fallen snow is large, generally in the range of 0.85-0.95. Snow metamorphism is the modification of the snow crystals and grains to less angular, more rounded forms with time. Metamorphism increases the size of the crystals which decreases the surface area and increases their volume. This causes the albedo to decline as the metamorphism progresses. As long as the air temperature is less than 0°C (32°F), metamorphism proceeds slowly and the rate of decline of the albedo is relatively slow. Each new snowfall 'resets' the albedo back to the newly fallen value and the metamorphism and albedo decline start over again. However, when the air temperature is greater than 0°C (32°F) and active snowmelt is occurring, metamorphism occurs quickly and the rate of decline of the albedo is relatively rapid. The albedo can decline to values of about 0.40 for well-aged snow. The albedo may drop to even lower values when the snowpack is shallow (snow depths of 0.5 m or less) allowing the ground surface beneath the snow to have an influence. Dust, soot, forest debris such as bark and twigs, and other deposited matter can also influence the snow surface albedo, and generally cause it to decline.



6 Three winters of albedo measurements and snow depth observations in the Senator Beck basin in Colorado. The sold blue circles are the daily average snow albedo, the open blue circles are the daily average albedo when the snow depth was reported as zero, and the orange line is the daily average snow depth.

6.3.3.4.2 Modeling Downwelling shortwave radiation.

Many factors can influence the amount of shortwave radiation reaching the ground at any location. The journey of shortwave radiation begins at the surface of the sun where it is emitted. It then travels through space for a short span of 8 minutes and 20 seconds to reach the top of the atmosphere of the earth. This top-of-the-atmosphere value can be directly calculated as

$$I_{o\downarrow} = S_0 \left(\frac{r_0}{r} \right)^2 \cos \theta_0$$

where $I_{0\hat{a}}$ = the top of the atmosphere shortwave radiation (Wm^{-2}); r_0 = the mean distance between the earth and sun; r = the actual Earth-Sun distance; S_0 = the solar constant at the mean Earth-Sun distance r_0 (1369.3 w/m^2); and q_0 = the solar zenith angle, the angle measured at the earth's surface between the location of the sun in the sky and the local zenith (The local zenith is the point in the sky directly above a particular location.) The Earth-Sun distance r varies throughout the year because the earth follows a slightly elliptical orbit around the sun. Each of these geometrical parameters in this equation, r_0, r, and q_0, can be calculated with precision because the clockwork nature of the earth's orbit around sun and the obliquity (tilt) of the earth itself are both well understood. (Whether or not the solar constant, S_0, is, in fact, a constant is a question beyond the scope of this write up. Certainly, ongoing observations suggest that any variations are relatively small.) The formulas for the geometrical parameters are straightforward but computationally intensive if done by hand. In short, the top of the atmosphere shortwave

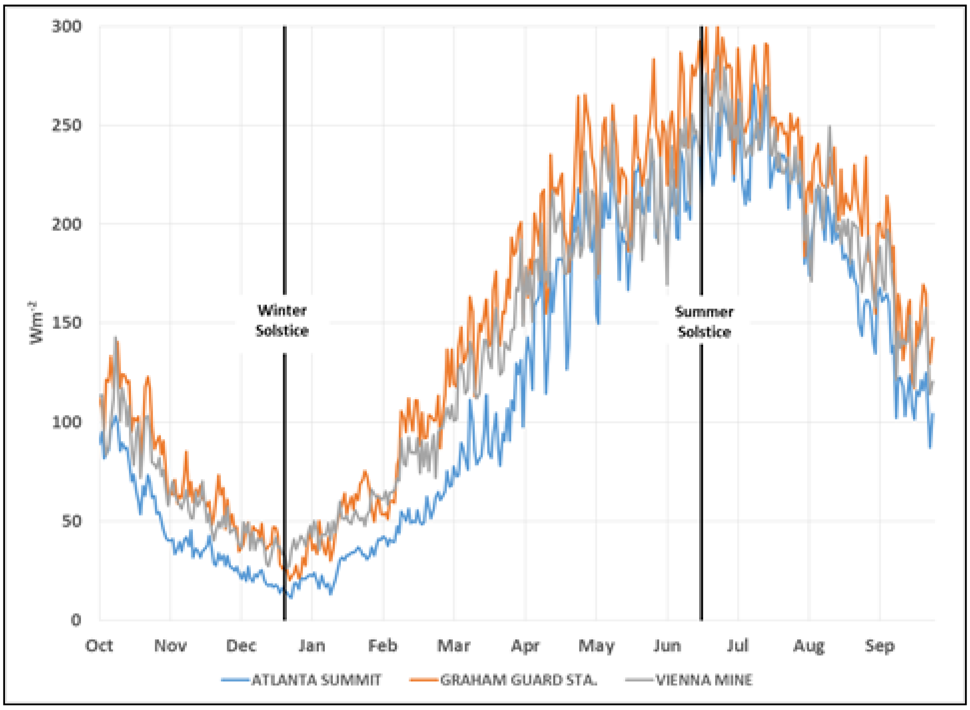
radiation can be calculated precisely for any location on earth for any time if the following are known: day of year, latitude, longitude, time of day, and offset from Greenwich Mean Time. The portion of the top of the atmosphere shortwave radiation that actually reaches the surface of the earth depends on the conditions of the atmosphere, - primarily the presence of clouds. The earth's atmosphere is not perfectly transparent to all the shortwave radiation even on a cloud free day when some radiation will be scattered, absorbed by gases and water vapor, and scattered by aerosol particles. These reductions in solar radiation on cloud-free days will tend to be relatively small unless the atmosphere is particularly turbid.

Clouds have a major impact on the sunlight reaching the earth. The impact will vary depending on the location of clouds relative to the position of the sun, the type of clouds, and the percentage of the sky covered by clouds. An example relationship describing the shortwave radiation absorbed by a horizontal snow surface given the top of atmosphere value can be written

$$Q_{sw} = (1-\alpha)a_t\left(1-0.65clf^2\right)I_{0\downarrow} = (1-\alpha)Q_{sw\downarrow}$$

where a_t = an attenuation factor due to dust, scattering and absorption by the atmosphere (a_t < 1); *clf* = the sky cloud factor (0 < *clf* < 1); α = the snow albedo; $I_{\text{oâ}}$ = the top of the atmosphere shortwave radiation; and $Q_{\text{SWâ}}$ = the shortwave radiation reaching the snow surface. If the snow surface is not horizontal corrections can be made based on the slope and aspect of the immediate topography. Shadows from surrounding terrain can also impact the downwelling shortwave radiation.

The top of the atmosphere shortwave radiation arriving at any location follows a seasonal cycle. In the Northern Hemisphere, the minimum top of the atmosphere radiation occurs at the winter solstice (December 21st). The value of the daily average shortwave radiation at the winter solstice decreases from south to north. North of the Arctic Circle (66° 33' 47.3"), the daily average shortwave radiation is zero at the winter solstice because north of the Arctic Circle is continually dark at that time of year. As the season progresses in time the solar radiation increases at every latitude in the northern hemisphere reaching a maximum on the summer solstice (June 21st). The relative change from winter minimum to summer maximum is greatest in the northern latitudes and less in the southern. The further south a position is located the earlier in the year it will reach a given level of solar radiation above its minimum. At the summer solstice the *daily average* shortwave radiation is remarkably uniform from the North Pole to the equator. However, the length of the sunlit portion of the day also varies from the North where there are 24 hours of continuous daylight to a minimum at the equator where there are 12 daylight hours. This means that the instantaneous or hourly radiation is less in the north because the daily average is applied over more hours of daylight.



7 Daily average broadband downwelling shortwave radiation measured at three SNOTEL sites located in the same region of Idaho. Each day has been averaged all years in the POR.

6.3.3.5 Precipitation heat transfer

Heat can be transferred into the snowpack by precipitation. The heat transfer process is very different depending on the *form* of the precipitation, rain or snow. In the HEC-HMS Temperature Index Snow Model, the form of the precipitation is determined by comparing the air temperature, T_a, to the rain/snow determinate temperature, also known as the PX temperature, or T_{PX}. If the air temperature is warmer than the PX temperature, $T_a > T_{PX}$, than the precipitation is falling as rain. If the air temperature is less than or equal to the PX temperature, T_a £ T_{PX}, than the precipitation is falling as snow. T_{PX}.is considered a constant, and is set by the model user. In general, T_{PX} is usually set warmer than the ice/water equilibrium temperature (32°F (0°C)), by a small margin.

6.3.3.5.1 Snowfall.

The precipitation is falling as snow when T_a £ T_{PX}. The sensible heat that arrives at the surface of the snowpack due to snowfall is

$$Q_{snow} = \rho_w C_{p\_ice} S_t T_a$$

where S_t = the snowfall rate in terms of the snow water equivalent (depth/time); and T_a = the air temperature. Note that it is assumed that the temperature of the snowfall is the same as the air temperature, T_a. The snowfall sensible heat may or may not have an impact on the average snowpack temperature. This can be determined by restating equation as

$$\rho_w C_{p\_ice} \frac{dSWE_t \bar{T}_t}{dt} = Q_{\text{snow}} \ = \rho_w C_{p\_ice} S_t T_a$$

Which can be expanded into

$$\rho_w C_{p\_ice} \left(\bar{T}_t \frac{dSWE_t}{dt} + SWE \frac{d\bar{T}_t}{dt} \right) = \rho_w C_{p\_ice} S_t T_a$$

Note that the rate of change in SWE is equal to the snowfall rate, which is stated as

$$\frac{dSWE}{dt} = S_t$$

Substituting equation into equation , the change in the average snowpack temperature, , due to snowfall is

$$\frac{d\bar{T}_t}{dt} = \frac{S_t \left(T_a - \bar{T}_t \right)}{SWE_t}$$

The average snowpack temperature, , will be changed by snowfall only if the air temperature and are different.

The above equation can also be stated in terms of the Cold Content, Cc. First, the definition of Cold Content, as written in equation , is restated in terms of SWE

$$C_c = -\frac{\bar{\rho}_s}{\rho_w} D \frac{C_{pic} \Delta T}{\lambda} = -SWE \frac{C_{pice} \left(T_M - \bar{T} \right)}{\lambda}$$

The change of cold content with time can be found by taking the derivative of equation with respect to time and substituting in the expression for the rate of change of the average snowpack temperature, , from equation

$$\frac{dC_c}{dt} = -S_t \frac{C_{pice} \left(T_M - T_a \right)}{\lambda}$$

The rate of change of the Cold Content is described by , however, this can be stated in a more compact form if the rate that Cold Content arrives as snowfall,   is defined as

$$C_{c-precip} = -S_t \frac{C_{pice} \left(T_M - T_a \right)}{\lambda}$$

Then

$$\frac{dC_C}{dt} = C_{c\_precip}$$

6.3.3.5.2 Rainfall.

The precipitation is falling as rain when $T_a > T_{PX}$. Rainfall impacts the energy balance of the snowpack through the sensible heat that it brings to the snowpack and the through the possibility of phase change of the liquid water. The sensible heat is a determined by the temperature of the rain when it reaches the snow surface. Once the liquid water has cooled to the ice/water equilibrium temperature further heat extraction must result in phase change of the liquid water to ice. Generally, freezing of rainfall in the snowpack can only happen if the snowpack temperature is less than the ice/water equilibrium temperature.

The sensible heat that arrives at the surface of the snowpack due to rainfall is

$$Q_{\text{rain\_sensible}} = \rho_w C_{p\_\text{water}} P_t (T_a - T_m)$$

where P_t = the rainfall rate (depth/time); and T_a = the air temperature. Note that it is assumed that the temperature of the rainfall is the same as the air temperature, T_a. Also note that the water can only be cooled to T_m, the ice/water equilibrium temperature (32°F (0°C)).

Once the liquid water has reached the ice/water equilibrium temperature further cooling must result in phase change of the liquid water to ice. Generally, freezing of rainfall in the snowpack can only happen if the snowpack temperature is less than the ice/water equilibrium temperature. Freezing of rainfall in the snowpack is a very effective means of raising the snowpack temperature due to the latent heat released by the liquid water when it freezes.

The potential latent heat that arrives at the surface of the snowpack due to rainfall is

$$Q_{rain\_latent} = \rho_w \lambda P_t$$

Note that latent heat will be extracted from the liquid rainfall only as long as the snowpack temperature, , is less than the ice/water equilibrium temperature (32°F (0°C)), T_m.

$$Q_{\text{total}} = Q_{\text{latent}} + Q_{\text{sensible}} + (1 - \alpha)Q_{SW\downarrow} + Q_{LW\downarrow} + Q_{LW\uparrow} + Q_{\text{Precipitation}} + Q_{\text{Ground}}$$

6.4 Temperature Index Method

The HEC-HMS Temperature Index Snow Model is described in this section. This temperature index approach is derived, almost directly, from the Temperature Index Snow Model of SSARR model (USACE 1987). The SSARR Temperature Index Snow Model was based, in large part, on the original snow hydrology studies conducted by the Corps of Engineers in the 1950s (USACE 1956).

6.4.1 Overview

6.4.1.1 Mass Balance

A temperature index snow model simplifies the heat transfer calculations into the snowpack by estimating the heat transfer into or out of the snowpack as a function of the difference between the surface temperature and the air temperature.

During dry melt conditions, the program uses the following equation to compute snowmelt (assuming no cold content in the snow pack):

$$Melt = DryMeltRate * (AirTemperature - BaseTemperature)$$

where DryMeltRate is in inches/(Degree Fahrenheit-Day) or mm/(Degree Celsius-Day).

During rain on snow conditions, the program uses the following equation to compute snowmelt (assuming no cold content in the snow pack):

$$Melt = (WetMeltRate + 0.168 * PrecipitationIntensity) * (AirTemperature - BaseTemperature)$$

where WetMeltRate is in inches/(Degree Fahrenheit-Day) or mm/(Degree Celsius-Day) and the constant term, 0.168, has units of hour/(Degree Fahrenheit-Day) or hour/(Degree Celsius-Day). The rain on snow equation is based on equation 5-18 in Engineering Manual 1110-2-1406.

The snowmelt capability in HEC-HMS estimates the following snowpack properties at each time step: The Snow Water Equivalent (SWE) accumulated in the snowpack; the snowpack temperature (actually, the snowpack *cold content* but this is equivalent to the snowpack temperature); snowmelt (when appropriate); the liquid water content of the snowpack; and finally, the runoff at the base of the snowpack.

6.4.2 Energy Balance

6.4.2.1 Cold Content

The rate of change of cold content with time can be approximated starting with the definition from equation as

$$\frac{dC_c}{dt} = \frac{Q_t}{\rho_w \lambda_w} \approx \frac{h\left(T_{a_t} - T_s\right)}{\rho_w \lambda_w} = c_r \left(T_{a_t} - T_s\right)$$

where Q_t = the rate of heat transfer per unit area (energy per unit area per time); *h*= a heat transfer coefficient (energy per unit area per time per degree air temperature); T_{at} = the air temperature; T_s = a representative temperature of the snow pack; and c_r = the “cold rate” that will be discussed below. Note that, following the example of Anderson (1973) and others, the engineering approximation of heat transfer has been used. There is a question of what the representative temperature of the snowpack should represent. To be entirely consistent with the concept of engineering heat transfer coefficient, T_s should equal the surface temperature of the snowpack. However, this is not very satisfactory because the surface temperature of the snow pack is not known *a priori*. This is because the heat transfer from the snow pack is controlled both by the heat transfer from the surface to the atmosphere *and* by the heat conduction through the snowpack itself; with the slower of the two processes controlling the rate. To overcome this problem, the representative temperature of the snow pack will be considered to represent some interior temperature of the snowpack. If the snowpack is shallow, the temperature will be representative of the entire snowpack; if the snowpack is deep, the temperature will be representative of the upper layer. This representative temperature, termed the “Antecedent Temperature Index for Cold Content” (ATICC) will be estimated using quasi-engineering approach to heat transfer in a somewhat similar manner as the cold content, as described below.

6.4.2.1.1 Index Temperature

The cold content is found by first estimating an “Antecedent Temperature Index for Cold Content” (ATICC) “near” the snow surface, *ATICC,* defined and estimated as (Anderson 1973, Corps of Engineers 1987, p 18)

$$ATICC_2 = ATICC_1 + TIPM \cdot (T_a - ATICC_1)$$

where $ATICC_2$ = the index temperature at the current time step; $ATICC_1$ = the index temperature at the previous time step; and *TIPM* is a non-dimensional parameter. The problem is that limited documentation exists to describe how the parameter *TIPM* is related to the time step, snow material properties, or heat transfer conditions.

In this section a consistent approach for estimating cold content is developed that is based on the approach of estimating changes in cold content based on the temperature difference between the air temperature and ATICC. First, an approach for estimating ATICC is developed. To do this, we turn to a simple heat budget type analysis of the snow pack in order to gain some insight. A straightforward heat budget of the snow pack can be written

$$\rho_s C_p d \frac{\partial T_{ATI}}{\partial t} = h^* \left(T_a - T_{ATI} \right)$$

where d = the "depth" of the snow pack associated with the depth of the index temperature; T_{ATI} = the snow temperature measured by the antecedent temperature index, h^* = the "effective" heat transfer coefficient from the snow surface to the atmosphere ($wm^{-2}{}^\circ C^{-1}$); and T_a = the air temperature. Note that we are assuming that a region of the snow pack temperature has a uniform temperature T_{ATI}. This assumption is a bit dubious BUT it makes T_{ATI} entirely analogous to *ATICC*. Note also that h^* can be defined as

$$h^* = \frac{1}{\frac{1}{h} + \frac{l_s}{k_s}}$$

where h = the heat transfer coefficient from the snow surface to the atmosphere; k_s = the snow thermal conductivity; and l_s = the effective snow depth through which thermal conduction occurs. h^* will be dominated by whichever process is slower: heat transfer from the snow to the atmosphere or thermal conduction through the snow depth l_s. If it is assumed that the snowpack temperature is T_o at time t = 0, the solution for equation is

$$T_{ATI} = T_a + \left(T_o - T_a \right) e^{-\frac{h^*}{\rho_s C_p d} t}$$

where T_o = the initial snow pack temperature; and t = time from start. Setting T_{ATI} $ATICC_2$ and T_o $ATICC_1$, equation (6) can be restated as

$$ATICC_2 = ATICC_1 + \left(1 - e^{-\frac{h^*}{\rho_s C_p d} t} \right) \cdot \left(T_a - ATICC_1 \right)$$

By comparing equation and equation , the expression for *TIPM* results:

$$TIPM = \left(1 - e^{-\frac{h^*}{\rho_s C_p d} t} \right)$$

We can see that *TIPM* is a function of the material properties of the snow pack, ρ_s, c_p, and d; the heat transfer regime as indicated by h^*; ***and*** the time step, t. If we assume that the material properties and heat transfer regime of the snow pack are set by the value of *TIPM* corresponding to a given time step of one day (for example, $TIPM_1$ is the value of *TIPM* corresponding to a time step, t_1, of one day) then the value of *TIPM* at another time step with same material and heat transfer properties can be found as

$$TIPM_2 = 1 - \left(1 - TIPM_1 \right)^{\frac{t_2}{t_1}}$$

where t_2 = the time step corresponding to $TIPM_2$. Equation was also presented without explanation in Anderson (2006). Equation can now be restated as

$$ATICC_2 = ATICC_1 + \left(1 - \left(1 - TIPM_1 \right)^{\frac{t_2}{2}} \right) \cdot \left(T_a - ATICC_1 \right)$$

where $TIPM_1$ is the value of *TIPM* corresponding to a time step of one day; and t_2/t_1 is the ratio of the model time step (t_2) to one day (t_1). Equation employs the value of $TIPM_1$ calibrated from a time step of one day, and allows it to be used in model runs of 1 hour or even 1 minute and arrive at the same results for ATICC if the air temperature is the same.

If a simple differential equation for cold content is used

$$\frac{\partial cc}{\partial t} = c_r \left(T_a - T_{ATI}\right)$$

where cc = cold content (inches day^{-1}); and c_r = cold rate (in. day^{-1} °F^{-1}). Equation can be integrated by again setting T_{ATI} $ATICC_2$; noting the solution for $ATICC_2$ as given in equation to arrive at

$$cc_2 = cc_1 - \frac{c_r t_1 \left(1 - (1 - TIPM_1)^{\frac{t_2}{t_1}}\right)}{\log(1 - TIPM_1)}(T_a - ATICC_1)$$

Where log = the natural logarithm; and, as before, $TIPM_1$ is the value of $TIPM$ calibrated for a time step of one day; and t_2/t_1 is the ratio of the model time step (t_2) to one day (t_1). where t_2/t_1 is the ratio of the model time step (t_2) to one day (t_1) (or, more exactly t_1 should correspond to the units of c_r.).

6.5 Gridded Temperature Index Method

The Gridded Temperature Index Method is the same as the Temperature Index method but applied on a grid cell by grid cell basis rather than an area-average over the watershed. The Gridded Temperature Index is used in conjunction with the ModClark Unit Hydrograph Transform method.

6.6 Snowmelt References

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6.7 Applicability and Limitations of Snow Accumulation and Melt Methods

The following table contains a list of various advantages and disadvantages regarding the aforementioned snowmelt methods available for use within HEC-HMS. However, these are only guidelines and should be supplemented by knowledge of, and experience with, the methods and the watershed in question.

Method	Advantages	Disadvantages
Temperature Index	• "Mature" method that has been used successfully in thousands of studies throughout the U.S. • Easy to set up and use • Only requires precipitation and air temperature boundary conditions • More parsimonious than other methods	• May be too simple for some situations • Limited snowpack outputs compared to other methods
Hybrid/ Radiation-derived Temperature Index	• Incorporates factors such as short and longwave radiation into snowmelt equations • Can incorporate terrain slope, aspect, and shading • More snowpack outputs than Temperature Index method	• Requires more meteorologic boundary conditions than Temperature Index • Less mature than other methods
Energy Budget	• Incorporates factors such as short and longwave radiation, sensible heat flux, sublimation, condensation, and wind into snowmelt equations • Can incorporate terrain slope, aspect, and shading • Lots of snowpack outputs available including SWE, snow density, snow depth, snowpack temperature, snowpack energy, albedo, etc.	• Requires many meteorologic boundary conditions • Computationally intensive • Solution isn’t guaranteed to converge • Much less parsimonious than other methods

7 Evaporation and Transpiration

Precipitation provides nearly all of the influx of water to a watershed. It is obvious to engineers and hydrologists that precipitation must be accurately estimated before any estimates of channel flow can be made. It is much less common for equal consideration to be given to ways water leaves a watershed other than as outflow. In many areas of the United States, more than 50% of precipitation returns to the atmosphere through evaporation and transpiration. Transpiration makes up most of the return. These two processes are key to accurate long-term hydrologic simulations.

7.1 Evaporation and Transpiration Basic Concepts

Evaporation and transpiration are responsible for returning massive quantities of precipitation back to the atmosphere. While evaporation is present in a watershed, the dominant pathway for water to return to the atmosphere is via transpiration. Transpiration describes the process of plants extracting water from the soil through their roots and releasing it to the air through their leaves.

According to EM 1110-2-1417:

> *The fundamental water balance relationship that a continuous simulation model must satisfy to accurately represent the hydrologic cycle is:*
>
> *runoff = precipitation - evapotranspiration*
>
> *Consequently, estimating evapotranspiration is of major importance.*

7.1.1 Evaporation

Evaporation is the process of converting water from the liquid to the gaseous state. The process happens throughout a watershed. Water evaporates from the surface of lakes, reservoirs, and streams. Water also evaporates from small depressions on the ground surface that fill with precipitation during a storm. Water held in pore spaces near the surface of the soil may evaporate. Finally, precipitation that has landed on vegetation may evaporate before it can fall from the vegetation to the ground.

Thermodynamics of Evaporation

Evaporation is the conversion of water from the liquid to the gaseous state and involves large quantities of energy. The latent heat of vaporization is the amount of energy required to affect the state transition. The latent heat of vaporization is 539 calories per gram of water at 1 atmosphere of pressure. Additional energy is required to bring the water from its current temperature to the boiling point; water must be at the boiling point before it can vaporize. The energy required to raise the temperature of water is 1 calorie per Celsius degree. As energy is adsorbed in the water, some of the water molecules will begin moving faster. This increased movement is increased kinetic energy and is the result of applying energy to the water. The most common source of energy to drive evaporation is shortwave radiation from the sun. Eventually the fastest water molecules break free from the water and move into the air as water vapor. This process at the molecular level is repeated many times per second over the water surface. When taken in bulk, this is the process of evaporation. Because the fastest water molecules are the ones to break free to the air, the molecules left behind in the water are moving slower. A slower speed means a lower kinetic energy. Energy is equivalent to temperature so the temperature of the water will decrease during evaporation, even as some of the water is converted to vapor in the air.

Factors Affecting Evaporation

Many factors affect the amount of evaporation occurring in a watershed at any given time, and the factors are interrelated. However, some generalities exist. Evaporation occurs as long as the vapor pressure of the water is greater than the vapor pressure of the air. Therefore, evaporation will decrease as the relative humidity increases and will stop when the relative humidity reaches 100%. A number of environmental factors affect the vapor pressure of water and air through difference mechanisms. Evaporation will increase as the ambient temperature of the water increases. Several sequential days of high temperature could warm water and increase evaporation, but also shallow water is usually warmer than deep water. Evaporation decreases as the atmospheric pressure increases, or example when a high pressure system is present. Conversely, evaporation increases as atmospheric pressure decreases, such as with increasing elevation. Note however that increasing elevation typically is associated with decreases in temperature and the net result can be unpredictable. Wind also increases evaporation by removing the thin layer of vapor saturated air that forms over the water under calm conditions.

Measuring Evaporation

The most common way to measure evaporation is with special pans specifically designed for the purpose. Water is added to the pan and the depth of water is measured using either calibrated graduations marked on the side of the pan or measurement tools. The measurements are usually taken daily. Regardless of the method, the measurement relates directly to the equivalent depth of water that is evaporating at the location where the pan is sited. The pan must be periodically refilled with water.

7.1.2 Transpiration

Transpiration is the process of plants removing water from the soil and expelling it to the atmosphere. The water is extracted by the roots, travels through the plant vascular system, and exits through structures called stomata on the underside of the leaves. Some of the soil water is retained for the biological processes of the plant, while the process of evaporation that happens in the stomata cools the plant.

Root-Water Uptake

Water uptake does not begin with the roots; it begins within the stomata which are usually found on the underside of leaves. The stomata are tiny chambers with an opening to the air that can be regulated by the plant. The stomata are opened or closed in response to many different environmental and physiological factors. When the stomata are open, water vapor leaves through the opening as long as the relative humidity is less than 100%. The source of the vapor is water that evaporates inside the stomata, where it is found in the space between the cells that form the walls of the stomata. The evaporated water causes a meniscus to form in the space between the cells and a consequent capillary force is transferred to the vascular system of the plant. The capillary force is transmitted through the water in the vascular system from the leaves down to the roots. Microscopic hairs on the roots keep them in contact with the moist soil. Water is thus drawn into the roots due to the transmitted capillary force. The water moves throughout the vascular system of the plant performing functions such as transporting suspended nutrients. Water evaporating in the stomata performs the critical function of cooling the plant. If the stomata close for any reason, almost all water uptake by the plant will stop. Most plants transpire during daylight hours and cease at night.

Factors Affecting Transpiration

The factors affecting the rate of transpiration are related both to the amount of water in the soil and the plant. The factors are interrelated in very complex ways, but there are some generalities. One of the functions of transpiration is the cooling of the plant. The plant will attempt to increase transpiration during periods of high temperature to avoid heat damage, and may reduce transpiration in cool temperatures. The plant may open the stomata to initiate transpiration, but it can only occur if there is sufficient soil water. Plants cannot extract water in the soil below the permanent wilting point. This is the water content equivalent to the maximum capillary force which the plant can exert to extract water from the soil matrix. Different species of plants have differing abilities to generate capillary force in the vascular system. Further, the grain size distribution of the soil can also affect the permanent wilting point and changing the amount of water in pore spaces susceptible to the capillary force exerted by the plant. It is also the case that plants have a greater need for water during a rapid growth phase, and use less water after reaching maturity. Transpiration is affected by the density of plants in the landscape and by overall health. Finally, because transpiration is driven by evaporation of water in the stomata, all of the factors that effect evaporation also effect transpiration in a secondary manner.

Measuring Transpiration

The most accurate and complete method to measure transpiration is with a lysimeter. The lysimeter is essentially a large steel container on the order of 5 meters in diameter and 1 meter deep that sits on a scale. The container is installed in an agricultural field so that the top of the lysimeter is even with the ground surface. The container is filled with soil and crops are planted in the lysimeter and the surrounding field. Irrigation may be applied. The scale installed under the lysimeter measures the weight of the steel container, soil, water in the soil, and the crops. The reduction in weight on two subsequent days is equal to the amount of water evaporated plus the amount transpired. Given the density of water and the weight of water, the volume of water can be calculated. The combined evaporation and transpiration can then be calculated from the volume of water and the area of the lysimeter.
Transpiration can also be measured using the eddy covariance technique. Eddies are turbulent vorticies of air carrying varying concentrations of water vapor. If the amount of moving air and the concentration of water vapor in the air can be measured, then the transpiration can be calculated. This requires the installation of a tower at the site where transpiration will be measured. It is important that areas upwind of the measurement site be characteristic of vegetation at the site. Measurements may not be possible if the ground in the vicinity of the site is not level. Instruments for measuring vertical windspeed and air moisture content are installed on the tower. Separate instruments are used to monitor upward and downward eddies. Complex mathematics is used to take the instrument measurements and determine the net amount of water moving up from the ground surface. The upward moving water vapor is the result of transpiration.

7.1.3 Combined Evapotranspiration

It can be relatively straightforward to measure evaporation from an open water body such as a lake. However, measuring transpiration separately from evaporation over vegetation is very difficult. Consider that the trees, grass, or crops will be transpiring during daylight hours. Also, any water on the ground surface between the plants will be evaporating. Any measurement techniques will record the sum of evaporation and transpiration. In most cases it is not important in the context of hydrologic simulation to be able to separate the two distinct processes. The important component of the hydrologic cycle is the water that is removed from the soil and returned to the atmosphere. Therefore, an inability to measure the evaporation and transpiration separately is not a limitation in hydrologic simulation. It is almost always the case that evaporation and transpiration are combined and termed evapotranspiration.

7.2 Available Evaporation and Transpiration Methods

A total of seven different evapotranspiration methods are available for use within HEC-HMS. These methods include:

- Hamon Method (see page 116)
- Hargreaves Method (see page 118)
- Priestley Taylor Method (see page 119)
- Penman Monteith Method (see page 120)
- Monthly Average Method (see page 107)
- Specified Evapotranspiration

- Annual Evapotranspiration

The following sections detail their unique concepts and uses.

7.3 Monthly Average Method

The monthly average method is designed to work with data collected using evaporation pans. Pans are a simple but effective technique for estimating evaporation. There is a long history of using them and data is widely available throughout the United States and other regions.

7.3.1 Basic Concepts and Equations

Evaporation pans are measurement instruments. A standard pan is 121 centimeters in diameter and 24 centimeters deep, though variations exist. It is set on a wooden platform close to the ground as shown in Figure 24. It is common to measure the windspeed adjacent to the evaporation pan, as well as the minimum and maximum water temperature each day. Water is added to the pan. Care must be taken to keep the water surface at least 5 centimeters below the top of the pan so that wind does not blow waves over the edge. Usually the pan will be refilled daily to keep the water level from falling more than 7.5 centimeters below the top. If the pan is allowed to empty too much, the temperature of the water may increase and cause overestimation of the evaporation rate.

Figure 24.Example installation of a Class A evaporation pan.

Measurements are often performed in two separate ways. First, the pan may be marked with calibrated graduations. The graduations represent depth and measure the volume in the evaporation pan. The water surface is noted against the graduations to determine the water level each day. The difference on two sequential days is the amount of evaporation that occurred, measured as an equivalent depth. Second, the volume of water added to the pan each day can be measured; with the pan always being filled to the same level. Given the volume of water added and the area of the pan, the equivalent depth can be calculated. An average of the two depths may be taken as the estimate of the evaporation for each day.
Data collected with evaporation pans is usually reported in monthly averages. Averages must be determined

for a long period of time in order to eliminate yearly variations. The United Nations World Meteorological Organization (ref needed) recommends a minimum record length of 30 years. It has been found that for long-term simulations greater than 15 years, using pan evaporation data compares favorably with more sophisticated models of evaporation (ref needed).

7.3.2 Estimating Parameters

There are many sources of data for pan evaporation rates. The source could be the monthly evaporation normals for a site where data is collected with an evaporation pan. More often the data are presented within the context of a regional analysis utilizing multiple measure sites, for example Roderick and Farquhar (2004). Within the United States the National Weather Service has estimated average evaporation as shown in Figure 25.

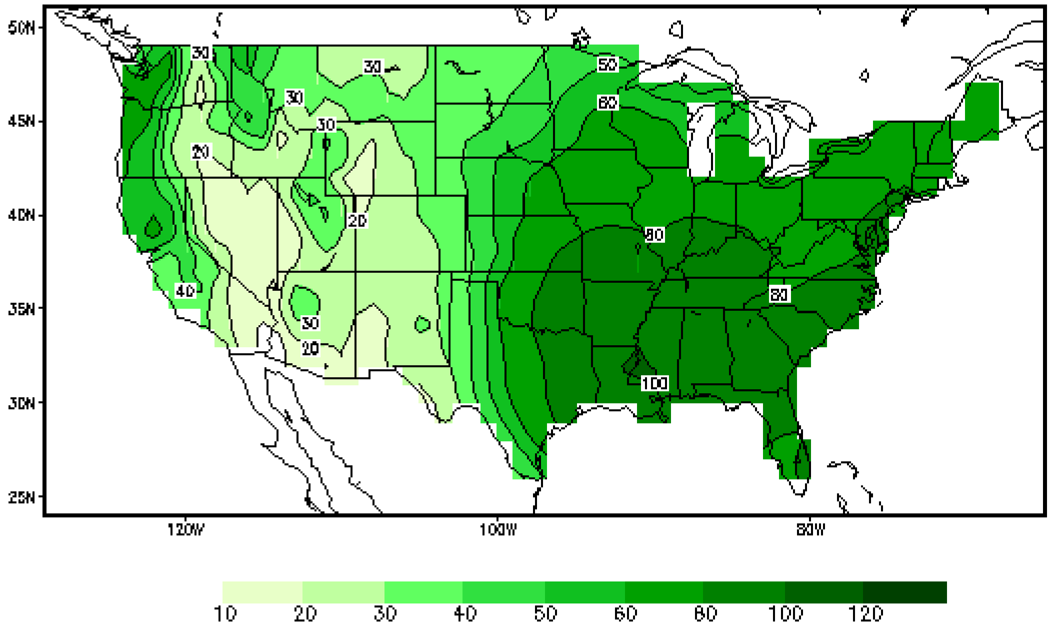

Figure 25.Calculated evaporation climatology in millimeters for the month of January using data from 1971 to 2000.

The measured pan evaporation rates overestimate evapotranspiration. The usual practice is to multiple the pan evaporation rate by a reduction ratio in order to approximate the evapotranspiration. The ratio typically ranges from 0.5 to 0.85 with the specific value depending on how the evaporation pan is sited and the atmospheric conditions. The ratio is larger when the relative humidity is higher. The ratio decreases as the windspeed increases. Typical values taken from the United Nations Food and Agriculture Organization (FAO, 1998) are shown in Table 14 and Table 15. The values are for a Class A evaporation pan and depend on the how the pan is located relative to vegetation, as shown in Figure 26.

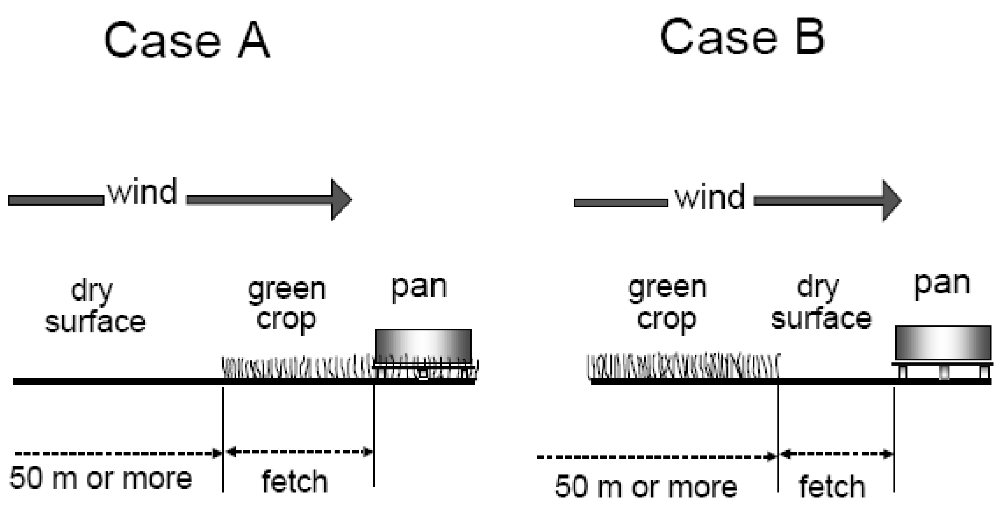

Figure 26.Two cases of evaporation pan siting and their environment.

Table 14.Pan coefficients for Class A pan sited on grass with a grass fetch but measuring transpiration over bare ground, and different levels of mean relative humidity (RH) and windspeed.

RH (%) -->		Low (< 40)	Medium (40-70)	High (> 70)
Windspeed (m/s)	Fetch Length (m)			
Light	1	0.55	0.65	0.75
< 2	10	0.65	0.75	0.85
	100	0.70	0.80	0.85
	1,000	0.75	0.85	0.85
Moderate	1	0.50	0.60	0.65
2-5	10	.06	0.70	0.75
	100	0.65	0.75	0.80
	1,000	0.70	0.80	0.80
Strong	1	0.45	0.50	0.60

5-8	10	0.55	0.60	0.65
	100	0.60	0.65	0.70
	1,000	0.65	0.70	0.75
Very strong	1	0.40	0.45	0.50
> 8	10	0.45	0.55	0.60
	100	0.50	0.60	0.65
	1,000	0.55	0.60	0.65

Table 15.Pan coefficients for Class A pan sited on bare ground with a bare ground fetch but measuring transpiration over grass, and different levels of mean relative humidity (RH) and windspeed.

RH (%) -->		Low (< 40)	Medium (40-70)	High (> 70)
Windspeed (m/s)	Fetch Length (m)			
Light	1	0.70	0.80	0.85
< 2	10	0.60	0.70	0.80
	100	0.55	0.65	0.75
	1,000	0.50	0.60	0.70
Moderate	1	0.65	0.75	0.80
2-5	10	0.55	0.65	0.70
	100	0.50	0.60	0.65
	1,000	0.45	0.55	0.60

Strong	1	0.60	0.65	0.70
5-8	10	0.50	0.55	0.65
	100	0.45	0.50	0.60
	1,000	0.40	0.45	0.55
Very strong	1	0.50	0.60	0.65
> 8	10	0.45	0.50	0.55
	100	0.40	0.45	0.50
	1,000	0.35	0.40	0.45

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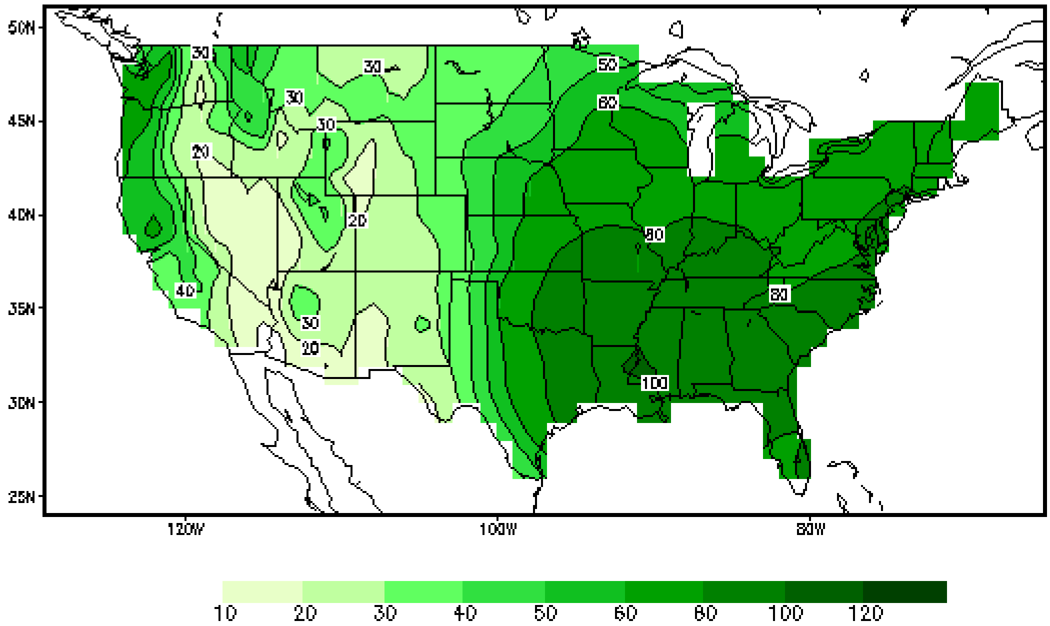

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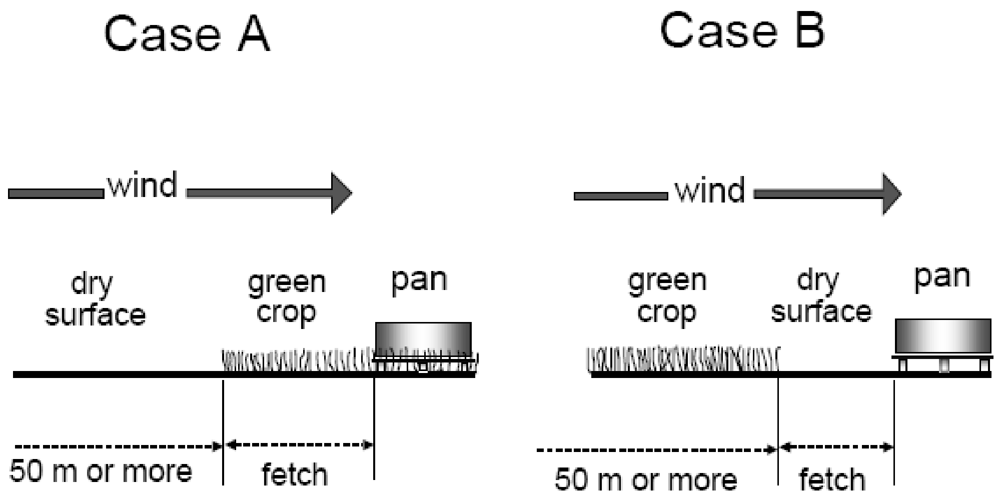

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Strong	1	0.45	0.50	0.60
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Very strong	1	0.40	0.45	0.50
> 8	10	0.45	0.55	0.60
	100	0.50	0.60	0.65
	1,000	0.55	0.60	0.65

Table 15.Pan coefficients for Class A pan sited on bare ground with a bare ground fetch but measuring transpiration over grass, and different levels of mean relative humidity (RH) and windspeed.

RH (%) -->		Low (< 40)	Medium (40-70)	High (> 70)
Windspeed (m/s)	Fetch Length (m)			
Light	1	0.70	0.80	0.85
< 2	10	0.60	0.70	0.80
	100	0.55	0.65	0.75
	1,000	0.50	0.60	0.70
Moderate	1	0.65	0.75	0.80
2-5	10	0.55	0.65	0.70
	100	0.50	0.60	0.65
	1,000	0.45	0.55	0.60
Strong	1	0.60	0.65	0.70
5-8	10	0.50	0.55	0.65
	100	0.45	0.50	0.60
	1,000	0.40	0.45	0.55
Very strong	1	0.50	0.60	0.65
> 8	10	0.45	0.50	0.55
	100	0.40	0.45	0.50

	1,000	0.35	0.40	0.45

7.4 Hamon Method

7.4.1 Basic Concepts and Equations

The Hamon method (Hamon, 1963) [6]is one of two temperature-based evapotranspiration methods included in HEC-HMS. These method use an empirical relationship between temperature and net radiation. In this method, potential evapotranspiration is proportional to saturated water vapor concentration, at the mean daily air temperature, adjusted for daytime hours. Since transpiration occurs during the day, the daytime hour adjustment accounts for plant response, duration of turbulence, and net radiation. For simulation time steps less than one day, potential evapotranspiration is redistributed for each time step based on a sinusoidal distribution between sunrise and sunset.

Average potential evapotranspiration ET_o is computed as (Hamon, 1963):

5)

$$ET_o = c\frac{N}{12}P_t$$

where c is a coefficient, N is the number of daylight hours, and P_t is the saturated water vapor density at the daily mean temperature.

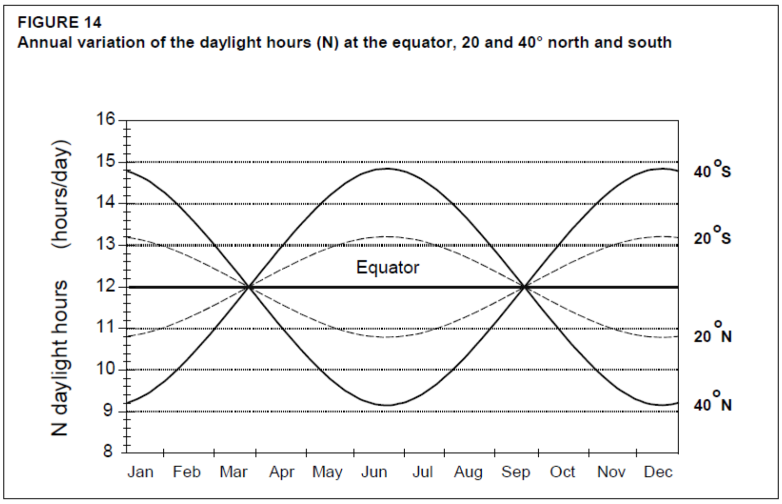
The number of daylight hours N is computed as (Allen et al., 1998):

6)

$$N = \frac{24}{\pi}\omega_s$$

where ω_s is the sunset hour angle.

6 https://ascelibrary.org/doi/epdf/10.1061/TACEAT.0008673



8 Annual variation in daylight hours (Allen et al., 1998)

The sunset hour angle is computed as (Allen et al., 1998):

7) $$\omega_s = arccos[-tan(\phi)tan(\delta)]$$

where ϕ is the latitude and δ is the solar declination.

The saturation vapor pressure e_s at the mean daily temperature T is computed as (Allen et al., 1998):

8) $$e_s(T) = 0.6108exp[\frac{17.27T}{T + 237.3}]$$

The saturation vapor density P_t is computed as (Wiederhold, 1997):

9) $$P_t = \frac{216.7e_s(T)}{T + 273.16}$$

7.4.2 Required Parameters

The only parameter required to utilize this method within HEC-HMS is the coefficient [in/g/m^3 or mm/g/m^3]. In addition, air temperature must be specified as a meteorologic boundary condition.

A tutorial using the Gridded Hamon method in an event simulation can be found here: Gridded Precipitation Method[7].

7 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/meteorologic-models-for-historical-precipitation/gridded-precipitation-method

A tutorial using the Gridded Hamon method in an continuous simulation can be found here: Advanced Applications of HEC-HMS Final Project[8].

7.4.2.1 A Note on Parameter Estimation

While HEC-HMS provides a default coefficient value of 0.0065 in/g/m^3 (0.1651 mm/g/m^3), this value **must be calibrated and validated**. In addition, air temperature must be provided as a meteorologic boundary condition.

7.5 Hargreaves Method

7.5.1 Basic Concepts and Equations

The Hargreaves method (Hargreaves and Samani, 1985) is one of two temperature-based evapotranspiration methods included in HEC-HMS. The method is based on an empirical relationship where reference evapotranspiration was regressed with solar radiation and air temperature data. The regression was based on eight years of precision lysimeter observations for a grass reference crop in Davis, CA. The method has been validated for sites around the world (Hargreaves and Allen, 2003). The method is capable of capturing diurnal variation in potential evapotranspiration for simulation time steps less than 24 hours.

Potential evapotranspiration ET_o is computed as (Hargreaves and Samani, 1985):

10)
$$ET_o = K * RS * (T + 17.8)$$

where K is a coefficient, RS is solar radiation, and T is mean daily temperature.

Hargreaves and Samani (1982) developed an equation for determining solar radiation from extraterrestrial radiation and the measured temperature range. Extraterrestrial radiation is the amount of solar energy that would be on a horizontal plane on the earth's surface if the earth was not surrounded by an atmosphere:

11)
$$RS = K_{RS} * R_a * \sqrt{T_{max} - T_{min}}$$

where K_{RS} is a coefficient, R_a is extraterrestrial radiation, and T_{max} and T_{min} are the daily maximum and minimum air temperature, respectively.

When the Hargreaves Evapotranspiration Method is used in combination with the **Hargreaves Shortwave Radiation Method**, the computed Hargreaves evapotranspiration form is equivalent to Hargreaves and Allen (2003) Eq. 8:

12)
$$RS = K_{RS} K R_a (T + 17.8) \sqrt{T_{max} - T_{min}}$$

8 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/hec-hms-example-applications/advanced-applications-of-hec-hms-final-project

7.5.2 Required Parameters

The only parameter required to utilize this method within HEC-HMS is the coefficient [deg C^{-1}]. In addition, air temperature must be specified as a meteorologic boundary condition.

7.5.2.1 A Note on Parameter Estimation

While HEC-HMS provides a default coefficient value of 0.0135 deg C^{-1}, this value **must be calibrated and validated**.

7.6 Priestley Taylor Method

7.6.1 Basic Concepts and Equations

Priestley and Taylor (1972)[9] developed a simplified energy balance approach to modeling evapotranspiration. Data from multiple sites, each with saturated surface or open water conditions and without advection, were used to develop a widely applicable formula for the relationship between sensible and latent heat fluxes. For partially saturated land surfaces, the reference evapotranspiration is related to the saturated evaportranspiration rate by a coefficient:

$$ET_o = \alpha(R - G)\frac{\Delta}{\Delta+\gamma}$$

where α is the dryness coefficient, R is the net incoming radiation, G is the heat flux into the ground (R - G = LE + H) where H is sensible heat and LE is latent heat, Δ is the slope of the saturation vapour pressure curve, and γ is the psychrometric constant.

7.6.2 Required Parameters

The only parameter required to utilize this method within HEC-HMS is the dryness coefficient. In addition, air temperature and net radiation must be specified as a meteorologic boundary condition. Net radiation should be computed, entered in the program as a radiation time-series gage, and selected as the shortwave radiation method.

7.6.2.1 A Note on Parameter Estimation

While HEC-HMS provides a default coefficient value of 1.26, this value **must be calibrated and validated**. The default value corresponds to a saturated surface, or wet conditions. Specifically, lower values of the dryness coefficient should be used in humid regions while higher values should be used in arid regions.

9 https://journals.ametsoc.org/view/journals/mwre/100/2/1520-0493_1972_100_0081_otaosh_2_3_co_2.xml

7.7 Penman Monteith Method

7.7.1 Basic Concepts and Equations

HEC-HMS implements the Penman-Monteith method as derived by the United Nations Food and Agriculture Organization (FAO) (Allen et al., 1998 (see page 120)). The Penman Monteith was adopted as the standard for reference evapotranspiration by the FAO. The reference evapotranspiration provides a standard to which evapotranspiration in different seasons or regions and of other crops can be compared.

Evapotranspiration can be derived using an energy balance or mass transfer method. Evaporation of water requires energy, either in the form of sensible heat or radiant energy. The rate of evapotranspiration is governed by the energy exchange at the vegetation surface and is limited by the amount of available energy. Therefore, the rate of evapotranspiration can be derived from a surface energy balance. Evapotranspiration can also be derived by balancing the incoming and outgoing water fluxes to the soil, or root zone. The mass transfer method is better suited for estimating ET over long time periods (on the order of weeks or more).

The Penman Monteith method combines energy balance and mass transfer methods (Penman, 1948[10]; Monteith, 1965[11]). The evapotranspiration rate is represented by the latent heat flux:

13)

$$\lambda ET = \frac{\Delta(R_n - G) + \rho_a c_p \frac{(e_s - e_a)}{r_a}}{\Delta + \gamma(1 + \frac{r_s}{r_a})}$$

where R_n is the net radiation at the crop surface, G is the soil heat flux , ρ_a is the mean air density at constant pressure, c_p is the specific heat of air, e_s is the saturation vapour pressure, e_a is the actual vapour pressure, e_s - e_a is the vapour pressure deficit, Δ is the slope of the saturation vapour pressure temperature relationship, and γ is the psychrometric constant, and r_s and r_a are the (bulk) surface and aerodynamic resistances, respectively.

The bulk surface resistance accounts for the resistance of vapour flow through the transpiring crop (stomata, leaves) and evaporating soil surface. The aerodynamic resistance describes the upward resistance from vegetation resulting from the friction from air flowing over vegetated surfaces.

While a large number of empirical evapotranspiration methods have been developed worldwide, some have been calibrated locally leading to limited global validity. The FAO Penman Monteith method uses the concept of a reference surface, removing the need to define parameters for each crop and stage of growth. Evapotranspiration rates of different crops are related to the evapotranspiration rate from the reference surface through the use of crop coefficients. A hypothetical grass reference was selected to avoid the need for local calibration. According to FAO (Allen et al., 1998):

> *The reference surface closely resembles an extensive surface of green grass of uniform height, actively growing, completely shading the ground and with adequate water. The requirements that the grass surface should be extensive and uniform result from the assumption that all fluxes are one-dimensional upwards.*

The reference crop is defined as a hypothetical crop with a height of 0.12 m, a surface resistance of 70 s/m, and an albedo of 0.23. The FAO's simplified equation for reference evapotranspiration is (Allen et al., 1998):

10 https://royalsocietypublishing.org/doi/10.1098/rspa.1948.0037
11 https://repository.rothamsted.ac.uk/item/8v5v7/evaporation-and-environment

14) $$ET_o = \frac{0.408\Delta(R_n - G) + \gamma\frac{900}{T+273}u_2(e_s - e_a)}{\Delta + \gamma(1 + 0.34u_2)}$$

where ET_o is the reference evapotranspiration, R_n is the net radiation at the crop surface, G is the soil heat flux density, T is the mean daily air temperature at 2 m height, u_2 is the wind speed at 2 m height, e_s is the saturation vapour pressure, e_a is the actual vapour pressure, e_s - e_a is the vapour pressure deficit, Δ is the slope of the saturation vapour pressure curve, and γ is the psychrometric constant.

7.7.2 Required Parameters

The parameterization is entirely dependent on the atmospheric conditions: solar radiation, air temperature, humidity, and wind speed measurements. Weather measurements should be made at 2 m above the ground surface (or converted to that height).

7.8 Applicability and Limitations of Evapotranspiration Models

The following table contains a list of various advantages and disadvantages regarding the aforementioned evapotranspiration methods available for use within HEC-HMS. However, these are only guidelines and should be supplemented by knowledge of, and experience with, the methods and the watershed in question.

Method	**Advantages**	**Disadvantages**
Hamon	• Simple, parsimonious method. • Mean air temperature is the only required meteorologic input.	• Based on an empirical relationship between air temperature and net radiation.
Hargreaves	• Simple, parsimonious method. • Mean air temperature is the only required meteorologic input.	• Based on an empirical relationship between air temperature and net radiation.
Priestley Taylor		
Penman Monteith	• Energy balance with mass transfer method. • Widely used and documented through United Nations Food and Agricultural Organization (FAO).	• Method is <u>less</u> parsimonious than simpler ET methods; it requires many more meteorologic boundary conditions.

Method	Advantages	Disadvantages
Monthly Average	• Simple, parsimonious method. • Pan evaporation data is widely available.	• Differences in the water and cropped surface can produce significant differences in the water loss from an open water surface and the crop. • Empirical coefficients used to relate evapotranspiration to pan evaporation.

7.9 Evaporation and Transpiration References

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8 Infiltration and Runoff Volume

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.

8.1 Infiltration and Runoff Volume Basic Concepts

Determining the portion of precipitation that becomes runoff volume is a complicated matter. Precipitation may first fall on a vegetation canopy that intercepts a portion of the precipitation. Surface depressions capture some of the precipitation reaching the ground and allow it to infiltrate. Water that does not infiltrate generally moves over the ground surface to become runoff volume. Once water is in the soil it can move vertically and a portion that infiltrated may return to the atmosphere through evapotranspiration. The weight of consideration given to each of these components depends on the purposes of the hydrologic study. Studies using event simulation methods tend to focus on the initial condition at the beginning of the storm and the portion of the storm volume that becomes runoff. Studies using continuous simulation methods usually focus on infiltration and evapotranspiration in order to estimate monthly or annual runoff volumes.

8.1.1 Interception

Many watersheds have some type of vegetation growing on the land surface. The vegetation in a natural watershed could be grass, shrubs, or forest. Agricultural watersheds could have field crops such as wheat or row crops such as tomatoes. Even urban watersheds often have vegetation with some cities maintaining extensive urban forests. Falling precipitation first impacts the leaves and other surfaces of the vegetation. Some of the precipitation will remain on the plant while the remainder will eventually reach the ground. The portion of precipitation that remains on the plant is called interception.

Precipitation that is intercepted can return to the atmosphere through evaporation. Evaporation is significantly reduced during a precipitation event because the vapor pressure gradient is reduced by the high humidity associated with precipitation. However, after the precipitation event is over, the humidity will usually drop and restore the vapor pressure gradient. This allows evaporation to increase and intercepted precipitation will return to the atmosphere.

The amount of interception is a function of the species of plant and the life stage of the plant. In general, forests have the highest potential for interception with evergreen species collecting more precipitation than deciduous types. Shrubs often have an intermediate about of interception capability with grasses and crops showing the least ability to capture precipitation. Life stage is also important. Young plants are usually smaller and consequently capture less precipitation. Deciduous trees can capture a significant amount of precipitation during summer months when the canopy is full, but collect almost no precipitation in the winter when the leaves have fallen off.

Water that impacts on vegetation and does not remain as intercepted precipitation can reach the ground through two primary routes: throughfall or stemflow.

Throughfall refers to precipitation that initially lands on the vegetation surface, and then falls off the vegetation to reach the ground. The leaves of a particular plant species have a limited capacity for holding water in tension. Water beyond this capacity cannot remain on the vegetation for very long and will eventually fall off the leaf. It is possible for the amount of water that can be held on a leaf to be affected by atmospheric conditions such as windspeed.

Stemflow refers to precipitation that initially lands on the vegetation surface, and then moves along the leaf stems, branches, and trunk to reach the ground. The amount of stemflow observed in a plant species in dependent on the shape of the leaves and branches. Stemflow will be high in plants that have leaves and branches shaped in a way that collects intercepted water and directs it to the trunk. Evergreen trees with needles or plants with irregular branches often have higher throughfall and reduced stemflow.

8.1.2 Surface Depressions

Precipitation can arrive on the ground surface through a variety of pathways. The precipitation lands directly on the ground when there is no vegetation in the watershed, or the precipitation can pass through gaps in the vegetation cover. Precipitation may also arrive on the surface as throughfall or stemflow. The water on the ground will collect in depressions. The capacity of depressions to hold water varies according to the land use. For example, a typical asphalt parking lot has a very small capacity for storing surface water. Conversely, conservation agriculture practices use tillage techniques designed to increase the capture of water in surface depressions.
Water captured in surface depressions can infiltrate into to the soil after precipitation has stopped. The amount of depression storage can control the partitioning of precipitation between infiltration and surface runoff. Watersheds with a small depression storage capacity will capture very little precipitation and infiltration will occur only during storm events. Watersheds with substantial depression storage will capture precipitation and infiltration it during the storm event, and water in depressions at the end of the event will infiltration after the storm has stopped. Water that is not captured in surface depressions will usually flow over the surface as direct runoff.

8.1.3 Infiltration During a Storm

Infiltration is the process of water on the surface of the soil moving down into the soil. Soil is a porous media with a structure composed of a variety of grain sizes with air between the individual grains. In some rare cases, the pores of the soil are completely saturated with water and no air remains. In most cases the soil is unsaturated and only some of the pores contain water. Therefore, simulating the behavior of water in soil is complicated because the exact nature of the pore spaces (their volumetric ratio and connectedness) is not known. Further, while the total amount of water in the soil may be estimated with reasonable accuracy, it is generally unknown exactly where the water is located in the pore spaces. This entire descriptive task is complicated many times over by the spatial variation of soil and pore properties horizontally and vertically throughout the watershed.

Soil as a porous media can be visualized similar to a sponge that cannot deform. The soil grains are analogous to the structure of the sponge and the pore spaces are analogous to the empty space in the sponge. In order to be a porous media, the pore spaces must be small enough for capillary forces to be significant. The magnitude of capillary force is proportional to the radius of the pore space. Most soils have a wide variety of pore space sizes, and the individual spaces are irregularly shaped. Therefore, a measurement of the capillary force can change dramatically from point to point throughout the soil. The only way to development a meaningful measurement is to use an instrument that determines the average value over a volume of several cubic centimeters. Such a measurement is termed the soil water potential and by convention is negative. Units could be kilopascals, millimeters of water, or Bar.

The flow of water in either saturated or unsaturated soil can almost always be described by Darcy's Law. It is a basic relationship that states the vertical flow of water is proportional to the potential gradient. When the spatial coordinate is taken as zero at the soil surface and measured downward, Darcy's Law can be stated in the following form:

15)
$$v = K\frac{d\psi}{dz}$$

where v is the flow per unit area, ψ is the matric potential (a negative value), z is the spatial coordinate (measured positive downward), and K is the hydraulic conductivity. If the soil is saturated, then K is the saturated hydraulic conductivity and is a function of the soil properties and the water properties. For unsaturated conditions the conductivity is still a function of soil and water properties, but is additionally a function of the matric potential. As the water content decreases from saturated toward the residual content, the matric potential becomes increasingly negative. The relationship between conductivity and matric potential is nonlinear resulting in the magnitude of the conductivity varying by several orders of magnitude over the possible range of water content. Darcy's Law is at the heart of all physically-based models of infiltration and is also used on many conceptual models.

The water moves in soil through a combination of two basic forces: absorption and gravity. The term absorption is used to describe the process of water on the surface of the soil being drawn into the soil because of a gradient in the matric potential. Recall that the magnitude of the matric potential increases as the soil becomes dryer. The greater the magnitude of the matric potential, the greater the matric potential gradient will become. It is usually the case that infiltration during the first portion of a precipitation event is dominated by absorption, since the soil is often dry when precipitation begins. The matric potential will decrease as the water content of the soil increases, eventually becoming zero at saturation. Therefore, the contribution of absorption to the total infiltration will also decrease as the precipitation event progresses.

The degree to which absorption plays a role in overall infiltration into a soil is determined by the properties of the soil. Soils with small pore spaces (such as clay soils) have a greater matric potential for a given water content than soils with large pore spaces (such as sandy soils). One measure of this property of a soil is the bubbling pressure or air entry potential. Soils with a large bubbling pressure show a great deal of absorption as part of infiltration. Soils with a small bubbling pressure will very quickly transition from absorption effects to gravity effects during a precipitation event.

Water begins moving through soil under the effect of gravity after the soil is saturated. The matric potential becomes zero when the soil is saturated and Darcy's Law predicts water will move at the saturated hydraulic conductivity. However, there is a degree to which gravity effects water flow in soil just as there is a degree to which absorption effects flow. Some portions of the soil will be saturated long before the bulk soil is saturated. A certain collection of soil pores that form a vertical path from the ground surface to the bottom of the soil layer may become saturated. Once those pores become saturated, gravity will drive infiltration through those pores even if surrounding pores are unsaturated and still dominated by the absorption process.

Absorption and gravity effects work together to produce the total infiltration rate. Water is pulled into dry soil by absorption. Absorption is the dominate force driving infiltration as a front of water enters a dry soil at the ground surface and begins moving down toward the bottom of the soil layer. Soil will be saturated some short distance behind the wetting front. Water in the saturated pore spaces, closer to the soil surface, will be dominated by gravity effects. Therefore, the gravity effects supply the water through the saturated pores to the areas along the wetting front where absorption is expanding the saturated soil region.

The combination of absorption and gravity gives a high total infiltration rate at the beginning of a precipitation storm. The total infiltration rate will then decrease as the soil layer becomes saturated. Infiltration in the saturated portions of the soil can only be driven by gravity and lacks the additional component of absorption. Eventually the soil layer becomes completely saturated and the absorption component becomes zero throughout the soil. At this point a steady-state is reached and infiltration is only a function of gravity effects. The overall result is that the high initial rate of infiltration decreases as the soil becomes saturated and eventually becomes constant at the saturated hydraulic conductivity.

8.1.4 Processes Occurring Between Storms

Physical forces act on water in the soil between storm events. The water will move in response to capillary suction forces in a process that redistributes the wetting front. Some of the water will be removed through evapotranspiration. Representation of these processes is critical in continuous simulations. A storm event simulation simply specifies the wetting front and soil moisture states as initial conditions at the beginning of a simulation. A continuous simulation must model these two key processes accurately from the end of one storm event to the beginning of the next storm event. If the simulated state of the soil is incorrect when the second storm begins, then the infiltration and runoff volume for the second storm will not be accurate.

Water does not stop moving in the soil after the precipitation stops. The process of absorption, driven by the matric potential in the unsaturated pore spaces, will redistribute water. During a precipitation event, the wetting front can advance down into the soil relatively quickly due to the combined effects of absorption and gravity. When the precipitation ends, the wetting front no longer has a supply of water coming from the surface. The wetting front will not advance without a supply of water so the water that is in the soil must move in response to matric forces. The matric potential is greater below the wetting front than it is the soil above the wetting front. Therefore, the water will move from the area above the wetting front toward the bottom of the soil layer. Reducing the water content above the wetting front will increase the matric potential in this region of the soil. The matric potential below the wetting front will decrease as water is pulled down from the formerly saturated region above the wetting front. Eventually the matric potential above the wetting front will be equal to the matric potential below the wetting front. At this point it will no longer be possible to identify the wetting front.

Evapotranspiration influences water movement in the soil between storm events by changing the matric potential. Plants extract water from the soil during the day. The water is drawn into the roots, which reduces the water content in the soil surrounding the roots. The decrease in the water content around the roots causes the matric potential to increase. The increase in the matric potential will cause water to move toward the roots from other regions of the soil layer that have a higher water content. Evapotranspiration can make it impossible for the soil matric potential to come to a true steady-state condition. The result is that soil water is dynamic and is often in a constant state of flux.

8.1.5 Data Requirements

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. The infiltration loss methods included in the program include the ability to specify the percentage of the watershed which is impervious. Impervious surface is usually associated with urbanized areas including roads, parking lots, and building roofs. Precipitation on the pervious surfaces is subject to losses.

8.2 Available Loss Methods

The following loss methods are included within HEC-HMS:

- Initial and Constant[12]
- Deficit and Constant[13]
- Green and Ampt[14]
- Layered Green and Ampt[15]
- Linear Deficit and Constant[16]
- SCS Curve Number[17]
- Exponential[18]
- Smith Parlange[19]
- Soil Moisture Accounting[20]

With each method, precipitation loss is found for each computation time interval, and is subtracted from the precipitation 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.

Some of the loss methods included in the program are gridded. These methods presume a subbasin is composed of regularly spaced cells with uniform length and width. These methods permit the user to specify initial conditions and parameters for each grid cell separate from the neighbor cells. All other loss methods simulate the entire subbasin with one set of initial conditions and parameters.

When using a unit hydrograph transform method, 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 ModClark method, the excess from the pervious and impervious portion of each cell is combined and routed to the outlet. With the kinematic wave transform method, directly connected impervious areas may be modeled separately from pervious areas if two overland flow planes are defined.

8.3 Initial and Constant Loss Model

8.3.1 Basic Concepts and Equations

The Initial and Constant loss method uses a hypothetical single soil layer to account for changes in moisture content. While this method is very simple, it is widely used to model watersheds that lack detailed subsurface information.

> ⚠ Since no means for extracting infiltrated water is included, this method should only be used for event simulation.

12 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/initial-and-constant-loss-model
13 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/deficit-and-constant-loss-model
14 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/green-and-ampt-loss-model
15 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/layered-green-and-ampt-model
16 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/linear-deficit-and-constant-model
17 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/scs-curve-number-loss-model
18 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/exponential-loss-model
19 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/smith-parlange
20 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/soil-moisture-accounting-loss-model

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+ Δt , the excess, pe_t , during the interval is given by:

16)
$$pe_t = \begin{Bmatrix} p_t - f_c & \text{if } p_t > f_c \\ 0 & \text{otherwise} \end{Bmatrix}$$

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:

17)
$$pe_t = \begin{Bmatrix} 0 & \text{if } \sum p_i < I_a \\ p_t - f_c & \text{if } \sum p_i > I_a \text{ and } p_t > f_c \\ 0 & \text{if } \sum p_i > I_a \text{ and } p_t < f_c \end{Bmatrix}$$

8.3.2 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the initial loss [inches or millimeters], constant rate [in/hr or mm/hr], and directly connected impervious area [percent].

A tutorial describing an example application of this loss method, including parameter estimation and calibration, can be found here: Applying the Initial and Constant Loss Method[21].
A tutorial describing how gSSURGO data can be formatted for use within HEC-HMS can be found here: Formatting gSSURGO Data for Use within HEC-HMS[22].

The initial loss defines the volume of water that is required to fill the soil layer at the start of the simulation. This parameter is typically defined using the product of the soil moisture state at the start of the simulation and an assumed active layer depth, but it should be calibrated using observed data. 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 rate defines the rate at which precipitation will be infiltrated into the soil layer after the initial loss volume has been satisfied. Typically, this parameter is equated with the saturated hydraulic conductivity of the soil. 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 the following table. These may be used in the absence of better information. Because the model parameter is not a

21 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/applying-the-initial-and-constant-loss-method

22 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/formatting-gssurgo-data-for-use-within-hec-hms

measured parameter, it and the initial condition are best determined by calibration. Chapter 9 of this manual describes the program's calibration capability.

Finally, the percentage of the subbasin which is directly connected impervious area can be specified. Directly connected impervious areas are surfaces where runoff is conveyed directly to a waterway or stormwater collection system. These surfaces differ from disconnected impervious areas where runoff encounters permeable areas which may infiltrate some (or all) of the runoff prior to reaching a waterway or stormwater collection system. No loss calculations are carried out on the specified percentage of the subbasin; all precipitation that falls on that portion of the subbasin becomes excess precipitation and subject to direct runoff.

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

8.3.2.1 A Note on Parameter Estimation

The values presented here are meant as **initial estimates**. This is the same for all sources of similar data including Engineer Manual 1110-2-1417 Flood-Runoff Analysis[23] and the Introduction to Loss Rate Tutorials[24]. Regardless of the source, these initial estimates **must be calibrated and validated**.

8.4 Deficit and Constant Loss Model

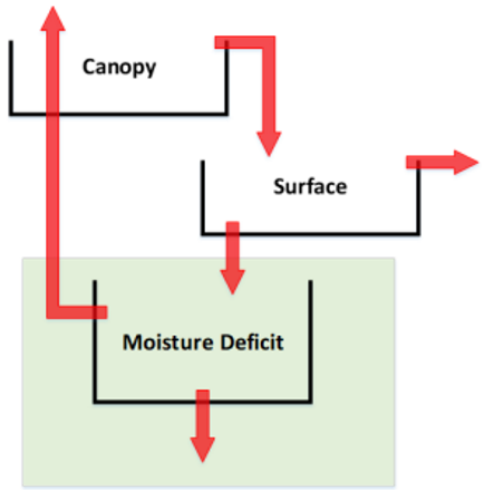
8.4.1 Basic Concepts and Equations

The Deficit and Constant loss method is very similar to the initial and constant loss method in that a hypothetical single soil layer is used to account for changes in moisture content. However, the deficit and constant method allows for continuous simulation when used in combination with a canopy method that will extract water from the soil in response to potential ET computed in the meteorologic model. Between precipitation events, the soil layer will lose moisture as the canopy extracts infiltrated water. Unless a

23 https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1417.pdf?ver=VFC-A5m2Q18fxZsnv19U8g%3d%3d

24 https://www.hec.usace.army.mil/confluence/display/HMSGUIDES/Introduction+to+the+Loss+Rate+Tutorials

canopy method is selected, no soil water extraction will occur. This method may also be used in combination with a surface method that will hold water on the land surface. The water in surface storage can infiltrate into the soil layer and/or be removed through ET. The infiltration rate is determined by the capacity of the soil layer to accept water. When both a canopy and surface method are used in combination with the deficit constant loss method, the system can be conceptualized as shown in the following figure.



9 Conceptual Representation of the Deficit and Constant Loss Method

If the moisture deficit is greater than zero, water will infiltrate into the soil layer. Until the moisture deficit has been satisfied, no percolation out of the bottom of the soil layer will occur. After the moisture deficit has been satisfied, the rate of infiltration into the soil layer is defined by the constant rate. The percolation rate out of the bottom of the soil layer is also defined by the constant rate while the soil layer remains saturated. Percolation stops as soon as the soil layer drops below saturation (moisture deficit greater than zero). Moisture deficit increases in response to the canopy extracting soil water to meet the potential ET demand.

Since this method allows for the extraction of infiltrated water, this method can be used for both event and continuous simulations.

8.4.2 Infiltration

The soil layer used in the deficit and constant loss model has a maximum capacity to hold water. The soil is saturated when the soil layer is at the maximum storage capacity, and it is not saturated when the layer contains less than the maximum storage capacity. The deficit is the amount of water required at any point in time to bring the soil layer to saturation. If the deficit is zero then the layer is saturated. When the layer is not saturated, the deficit is the amount of water that must be added to bring it to saturation. The deficit is measured in millimeters or inches. The maximum capacity minus the deficit gives the amount of water currently in storage. The current deficit (or current storage) is assumed to be uniformly distributed throughout the soil layer. The soil within the layer is assumed to have homogeneous properties.

The soil layer will have a certain moisture deficit at the beginning of a storm event. This amount could be zero, indicating the soil is completely saturated. However, it is much more common for the layer to have a deficit greater than zero, indicating it is not saturated. The moisture deficit could equal the maximum capacity if there has been an extended period without rain, and evapotranspiration has extracted all water from the soil layer.

The soil layer has an infinite capacity for infiltration when the deficit is greater than zero. This means that when the layer is below the saturation level, that all precipitation will infiltrate until the soil is saturated. This is one of the simplifying assumptions in the model since in reality it is possible for the rainfall rate to exceed the infiltration rate of the soil and result in direct runoff when the soil is not saturated. Nevertheless, within this loss model, all precipitation will infiltrate until the soil layer is saturated. All infiltrated water remains in the soil layer and does not percolate out of the layer.

8.4.3 Percolation and Excess Precipitation

Water will percolate out of the bottom of the soil layer if there is precipitation and the deficit is equal to zero. This represents precipitation infiltrating from the soil surface into the soil layer, and then percolating through the soil layer. Percolation water passes out of the bottom of the layer. It is lost from the system, unless the linear reservoir baseflow method is used. In this case only, the percolation water becomes baseflow.

Percolation will continue as long as the soil layer is at maximum storage capacity, and precipitation continues. If the precipitation rate exceeds the percolation rate, then precipitation up to the percolation rate will infiltrate into the soil layer and percolate out of the bottom of the layer. Any precipitation above the percolation rate will become excess precipitation and subject to direct runoff. If the precipitation rate is less than the percolation rate, then all of the precipitation will infiltrate into the soil layer and percolate out of the bottom of the layer. When the precipitation rate is less than the percolation rate there will be no excess precipitation. Percolation can only happen as long as the soil layer remains at saturation.

8.4.4 Evapotranspiration

Evapotranspiration removes water from the soil layer between storm events. The potential evapotranspiration rate is taken from the meteorologic model, where a variety of methods are available for representing that process. The evapotranspiration rate is used as specified by the meteorologic model without any modification. Water is removed from the soil layer at the potential rate for every time interval when there is no precipitation. There is no further evapotranspiration after the water in the soil layer is reduced to zero. Evapotranspiration will start again as soon as water is present in the soil layer and there is no precipitation.

8.4.5 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the initial deficit [inches or millimeters], maximum deficit [inches or millimeters], constant rate [in/hr or mm/hr], and directly connected impervious area [percent].

A tutorial describing an example application of this loss method, including parameter estimation and calibration, can be found here: Applying the Deficit and Constant Loss Method[25].

25 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/applying-the-deficit-and-constant-loss-method

A tutorial describing how gSSURGO data can be formatted for use within HEC-HMS can be found here: Formatting gSSURGO Data for Use within HEC-HMS[26].

The initial deficit defines the volume of water that is be required to fill the soil layer at the start of the simulation while the maximum deficit specifies the total amount of water the soil layer can hold.

The maximum deficit is typically defined using the product of the effective soil porosity and an assumed active layer depth, but it should be calibrated using observed data.

The initial deficit must be less than or equal to the maximum deficit. Both parameters are specified as effective depths (e.g., inches or millimeters).

The constant rate defines the rate at which precipitation will be infiltrated into the soil layer after the initial deficit has been satisfied in addition to the rate at which percolation occurs once the soil layer is saturated. Typically, this parameter is equated with the saturated hydraulic conductivity of the soil.

Finally, the percentage of the subbasin which is directly connected impervious area can be specified. Directly connected impervious areas are surfaces where runoff is conveyed directly to a waterway or stormwater collection system. These surfaces differ from disconnected impervious areas where runoff encounters permeable areas which may infiltrate some (or all) of the runoff prior to reaching a waterway or stormwater collection system. No loss calculations are carried out on the specified percentage of the subbasin; all precipitation that falls on that portion of the subbasin becomes excess precipitation and subject to direct runoff.

8.4.5.1 A Note on Parameter Estimation

The values presented here are meant as **initial estimates**. This is the same for all sources of similar data including Engineer Manual 1110-2-1417 Flood-Runoff Analysis[27] and the Introduction to Loss Rate Tutorials[28]. Regardless of the source, these initial estimates **must be calibrated and validated**.

8.5 Green and Ampt Loss Model

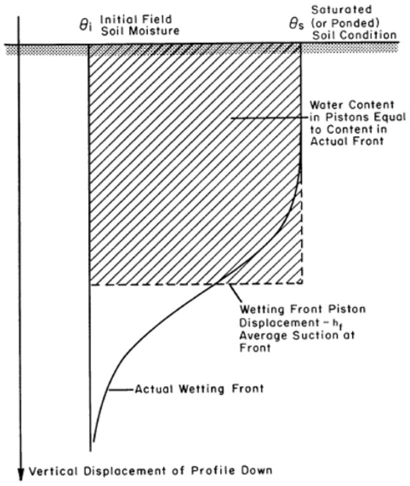
8.5.1 Basic Concepts and Equations

The Green and Ampt infiltration model included in the program is a conceptual model of infiltration of precipitation in a watershed. The Green and Ampt loss method was originally derived using a simplification of the comprehensive Richard's equation (1931) for unsteady water flow in soil. Within this method, infiltration proceeds with so-called piston displacement, as shown in the figure below.

26 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/formatting-gssurgo-data-for-use-within-hec-hms

27 https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1417.pdf?ver=VFC-A5m2Q18fxZsnv19U8g%3d%3d

28 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/introduction-to-the-loss-rate-tutorials



10 Conceptual Representation of the Green and Ampt Method Reproduced from (U.S. Army Corps of Engineers, 1994)

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:

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

in which *ft* = loss during period *t*, *K* = saturated hydraulic conductivity, (ϕ - θ_i) = volume moisture deficit, *Sf* = wetting front suction, and *Ft* = 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 the equation shown above. As implemented, the Green and Ampt model also includes an initial abstraction. This initial condition represents interception in the canopy or surface depressions not otherwise included in the model. This interception is separate from the time to ponding that is an integral part of the model. The solution method used follows that of Li et al. (1976)[29].

Since no means for extracting infiltrated water is included, this method should only be used for event simulation.

Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the initial moisture content or deficit [in/in or mm/mm], wetting front suction head [in or mm], saturated hydraulic conductivity [in/hr or mm/hr], and directly connected impervious area [percent].

29 https://ascelibrary.org/doi/pdf/10.1061/JRCEA4.0001092

A tutorial describing an example application of this loss method, including parameter estimation and calibration, can be found here: Applying the Green and Ampt Loss Method[30].
A tutorial describing how gSSURGO data can be formatted for use within HEC-HMS can be found here: Formatting gSSURGO Data for Use within HEC-HMS[31].

The initial moisture content or deficit defines the starting saturation of the soil layer at the start of the simulation. This parameter 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.

The wetting front suction head describes the movement of water downwards through the soil column.

The saturated hydraulic conductivity defines the minimum rate at which precipitation will be infiltrated into the soil layer after the soil column is fully saturated.

Finally, the percentage of the subbasin which is directly connected impervious area can be specified. Directly connected impervious areas are surfaces where runoff is conveyed directly to a waterway or stormwater collection system. These surfaces differ from disconnected impervious areas where runoff encounters permeable areas which may infiltrate some (or all) of the runoff prior to reaching a waterway or stormwater collection system. No loss calculations are carried out on the specified percentage of the subbasin; all precipitation that falls on that portion of the subbasin becomes excess precipitation and subject to direct runoff.

8.5.1.1 A Note on Parameter Estimation

The values presented here are meant as **initial estimates**. This is the same for all sources of similar data including Engineer Manual 1110-2-1417 Flood-Runoff Analysis[32] and the Introduction to Loss Rate Tutorials[33]. Regardless of the source, these initial estimates **must be calibrated and validated**.

8.6 Layered Green and Ampt Model

8.6.1 Basic Concepts and Equations

The layered Green and Ampt loss method expands upon the previously mentioned Green and Ampt method through the use of two soil layers to account for continuous changes in moisture content. The method is based on algorithms originally developed for the Guelph All-Weather Sequential-Events Runoff (GAWSER) model (Schroeter and Associates, 1996).

Using the layered Green and Ampt method allows for continuous simulation when used in combination with a canopy method that will extract water from the soil in response to potential ET computed in the meteorologic model. Between precipitation events, the soil layer will lose moisture as the canopy extracts infiltrated water. Unless a canopy method is selected, no soil water extraction will occur. This method may

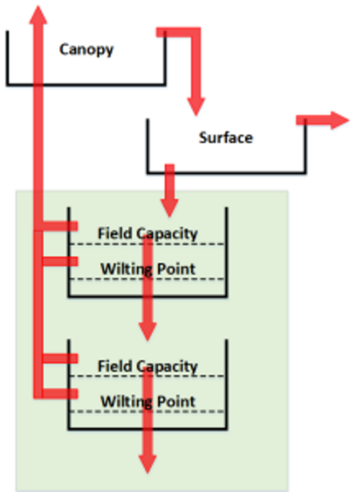
30 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/applying-the-green-and-ampt-loss-method

31 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/formatting-gssurgo-data-for-use-within-hec-hms

32 https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1417.pdf?ver=VFC-A5m2Q18fxZsnv19U8g%3d%3d

33 https://www.hec.usace.army.mil/confluence/display/HMSGUIDES/Introduction+to+the+Loss+Rate+Tutorials

also be used in combination with a surface method that will hold water on the land surface. The water in surface storage can infiltrate into the soil layer and/or be removed through ET. The infiltration rate is determined by the capacity of the soil layer to accept water. When both a canopy and surface method are used in combination with the layered Green and Ampt method, the system can be conceptualized as shown in the following figure.



11 Conceptual Representation of the Layered Green and Ampt Method

The layered Green Ampt loss method uses two layers to represent the dynamics of water movement in the soil. Surface water infiltrates into the upper layer, called layer 1. Layer 1 produces seepage to the lower layer, called layer 2. Both layers are functionally identical but may have separate and distinct parameters. Separate parameters can be used to represent layered soil profiles and also allows for better representation of stratified soil drying between storms. Each layer is described using a bulk depth and water content values for saturation, field capacity, and wilting point. Soil water in layer 2 can percolate out of the soil profile. The layered Green and Ampt method is intended to be used in combination with the linear reservoir baseflow method. When used in this manner, the percolated water can be split between baseflow and deep aquifer recharge.

First, precipitation fills the canopy storage. Precipitation that exceeds the canopy storage will overflow onto the land surface. The new precipitation is added to any water already in surface storage. The infiltration rate from the surface into layer 1 is calculated with the Green and Ampt equation so long as layer 1 is below saturation. The infiltration rate changes to the current seepage rate when layer 1 reaches saturation. Infiltration water is added to the storage in layer 1. Seepage out of layer 1 only occurs when the storage exceeds field capacity. Maximum seepage occurs when layer 1 is at saturation and declines to zero at field capacity. The seepage rate changes to the percolation rate when layer 2 is saturated. Seepage is added to the storage in layer 2. Percolation out of layer 2 only occurs when the storage exceeds field capacity. Maximum percolation occurs when layer 2 is at saturation and declines to zero at field capacity. Most soils observe decreasing hydraulic conductivity rates at greater depths below the surface. This means that typically the seepage rate is reduced to the percolation rate when layer 2 saturates, and the infiltration rate is reduced to the seepage rate when layer 1 saturates. The infiltration rate will change to the percolation rate if both layers 1 and 2 are saturated. Both convergence control and adaptive time stepping are used to accurately resolve the saturation of each layer.

The canopy extracts water from soil storage to meet the potential ET demand. First, soil water is extracted from layer 1 at the full ET rate. This extraction from layer 1 continues until half of the available water has been taken to meet the ET demand. The available water is defined as the saturation content minus the wilting point content, multiplied by the bulk layer thickness. Second, soil water is extracted from layer 2 at the full ET rate. This extraction from layer 2 also continues until half of the available water has been taken. Third, the ET demand is applied equally to both layers until one of them reaches wilting point content. Finally, the ET demand is applied to the remaining layer until it also reaches wilting point content. Soil water below the wilting point content is never used for ET.

> Since this method allows for the extraction of infiltrated water, this method can be used for both event and continuous simulations.

The layered Green and Ampt loss method can be used for continuous simulation because it is built on a water balance of the two layers. Infiltrated water is added to the layers and percolated water is removed from the layers. Potential ET demand also removes water from the layers. The continuous simulation will include storm events from time to time. The Green and Ampt equation is used to compute the surface infiltration during each of these storm events. The initial condition of the Green and Ampt equation for each of these storms must be determined. The initial content is essentially a water content deficit which can be calculated as the saturated water content minus the current water content. The water content deficit is automatically calculated at the beginning of each storm based on current soil water storage in layer 1. The user can control the amount of time that must pass since the last precipitation in order for the initial condition to be recalculated, using the dry duration parameter. When the time since last precipitation is less than the dry duration, the precipitation is considered a continuation of the last storm event.

8.6.2 Required Parameters

Parameters that are required to utilize the layered Green and Ampt method include those which were previously mentioned for the “standard” Green and Ampt method in addition to: Layer 1 and 2 thicknesses [in or mm], field capacity [in/in or mm/mm], wilting point [in/in or mm/mm], layer 1 maximum seepage rate [in/hr or mm/hr], layer 2 maximum percolation rate [in/hr or mm/hr], and dry duration [days].

The layer 1 and 2 thicknesses define the bulk depth of soil and are typically estimated using soil maps.

The field capacity content specifies the point where the soil naturally stops seeping under gravity while the wilting point content specifies the amount of water remaining in the soil when plants are no longer capable of transpiring infiltrated water. Both parameters are typically estimated using predominant soil texture and literature values.

The dry duration sets the amount of time that must pass after a storm event in order to recalculate the initial condition. An initial estimate of 12 hours has been found to be reasonable for the dry duration. However, this parameter should be calibrated using observed data.

Finally, the percentage of the subbasin which is directly connected impervious area can be specified. Directly connected impervious areas are surfaces where runoff is conveyed directly to a waterway or stormwater collection system. These surfaces differ from disconnected impervious areas where runoff encounters permeable areas which may infiltrate some (or all) of the runoff prior to reaching a waterway or stormwater collection system. No loss calculations are carried out on the specified percentage of the subbasin; all precipitation that falls on that portion of the subbasin becomes excess precipitation and subject to direct runoff.

8.6.2.1 A Note on Parameter Estimation

The values presented here are meant as **initial estimates**. This is the same for all sources of similar data including Engineer Manual 1110-2-1417 Flood-Runoff Analysis[34] and the Introduction to Loss Rate Tutorials[35]. Regardless of the source, these initial estimates **must be calibrated and validated**.

8.7 Linear Deficit and Constant Model

8.7.1 Basic Concepts and Equations

The linear deficit and constant loss method (LC method) is a modification of the initial and constant method (see Schoener et al., 2021[36] and Schoener et al., 2023):

18) $$f_t = \left\{ \begin{array}{ll} mF_t + f_0 & \text{if } F_t < F_c \\ K_{\text{eff}} & \text{if } F_t \geq F_c \end{array} \right\} \quad (f_0 \geq K_{\text{eff}})$$

where f_t (mm/hr or in/hr) is the potential infiltration rate at time t, F_t (mm or in) is the cumulative infiltration at time t, F_c (mm or in) is the initial deficit, m (1/hr) is the infiltration rate decay factor with respect to cumulative infiltration, and K_{eff} (mm/hr or in/hr) is the constant infiltration rate or effective hydraulic conductivity.

8.7.2 Infiltration

The LC model lets the potential infiltration rate f start at an initial value f_0 (mm/hr or in/hr) and decrease linearly as a function of cumulative infiltration until reaching a constant rate K_{eff} when cumulative infiltration is equal to initial deficit F_c. Due to the linear relationship, only m and F_c need to be defined in addition to K_{eff}. Compared to other simple loss methods such as the initial and constant or curve number model, the LC method has the advantage that it does not use an initial abstraction term and will simulate runoff from the start of a rainfall event if precipitation intensity for a given time step exceeds potential infiltration rate.

8.7.3 Event-Based Simulation

The LC model accounts for a single, hypothetical soil layer, hereafter referred to as the *active soil layer*. The soil layer has a maximum capacity to hold water. Figure 1 below shows a conceptual representation of the linear deficit and constant loss method when the active soil layer is not completely saturated, i.e. the layer contains less water than the maximum storage capacity. The deficit, measured in mm or in, is the amount of water required at any point in time to bring the active layer to saturation. During event-based simulation (Figure 1, left), water will infiltrate into the soil at a rate determined by the initial deficit, decay factor, and cumulative infiltration since the onset of the storm. If at any point in time the precipitation rate exceeds the

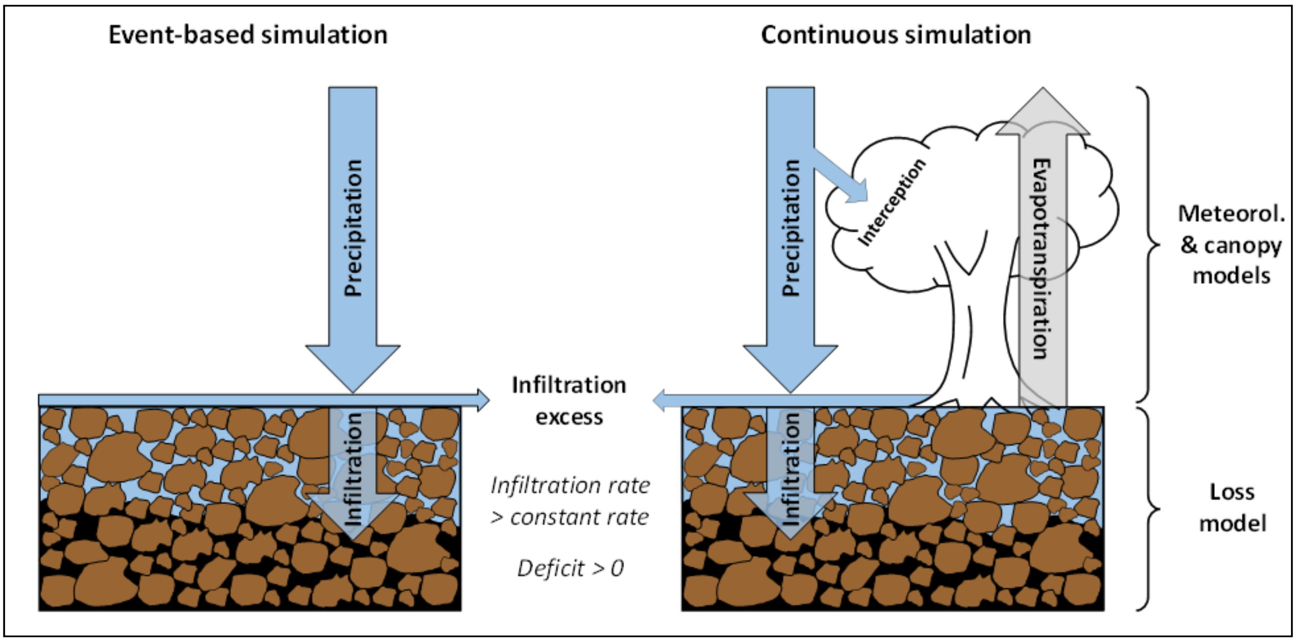
34 https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1417.pdf?ver=VFC-A5m2Q18fxZsnv19U8g%3d%3d
35 https://www.hec.usace.army.mil/confluence/display/HMSGUIDES/Introduction+to+the+Loss+Rate+Tutorials
36 https://doi.org/10.1016/j.jhydrol.2021.126490

potential infiltration rate, the difference (infiltration excess) will become runoff. If the precipitation rate at a given time is equal to or less than the potential infiltration rate, all rainfall infiltrates into the soil.

8.7.4 Continuous Simulation

The LC method also allows for continuous simulation (see Figure 1, right) when used in combination with a canopy method that allows extraction of water from the soil due to evapotranspiration. Continuous simulation requires the specification of another loss parameter, the maximum deficit. This value can be interpreted as the porosity multiplied by the thickness of the active layer and is measured in millimeters or inches.

For continuous simulation, the modeler must select a canopy method (under subbasin elements) and specify an evapotranspiration (ET) method (under meteorologic models). ET removes water from the active soil layer between and, depending on user setting, during storm events. The potential evapotranspiration rate is taken from the meteorologic model, where a variety of methods are available for representing that process. The ET rate is used as specified by the meteorologic model without any modification. There is no further evapotranspiration after the water in the soil layer is reduced to zero. ET will start again as soon as water is present in the soil layer. Unless a canopy and ET method are selected, no soil water extraction will occur. The canopy method also allows the modeler to simulate interception, the portion of precipitation intercepted by vegetation that never reaches the ground.



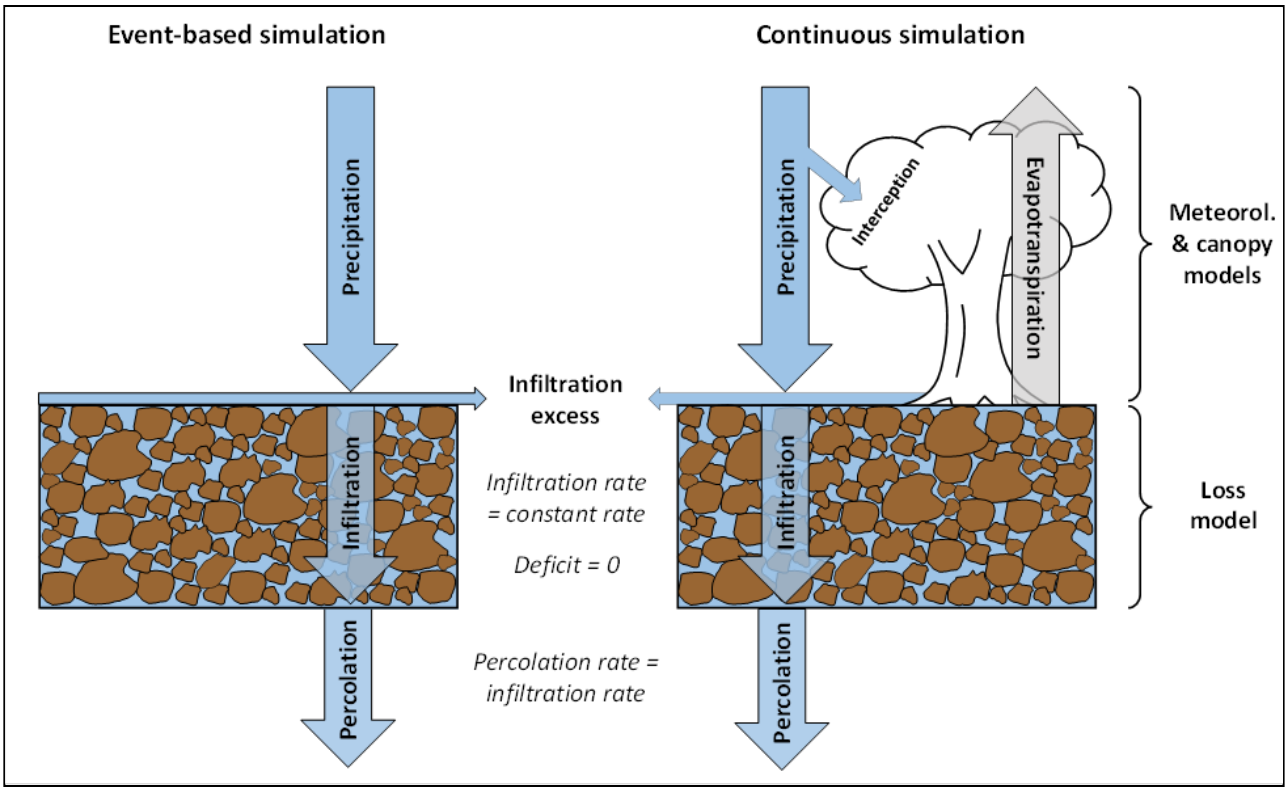
12 Figure 1: Conceptual representation of the linear deficit and constant loss method for event-based simulation (left) and continuous simulation (right) when the active soil layer has a deficit greater than zero.

8.7.5 Percolation

Once the active layer has saturated (the deficit is equal to zero), the potential infiltration rate becomes equal to the constant rate. Water will percolate out of the bottom of the active soil layer at a rate equal to the actual infiltration rate (see Figure 2). Percolation water is lost from the system. Percolation will continue as long as the soil layer is at maximum storage capacity, and precipitation continues. The linear deficit and constant method should therefore not be used for systems were:

- The water table is close to the surface, and the vadose zone could saturate completely during the analysis period; or
- An impermeable layer is present at a depth sufficiently shallow that that a perched aquifer could form during the analysis period.

In both cases, there would be no percolation once the active layer is saturated, and all additional precipitation would become runoff.



13 Figure 2: Conceptual representation of the linear deficit and constant loss method for event-based simulation (left) and continuous simulation (right) when the active soil layer has a deficit equal to zero.

8.7.6 Required Parameters

Parameters required to utilize this method within HEC-HMS include the initial deficit (mm or in), maximum deficit (mm or in), constant rate (mm/hr or in/hr), decay factor (1/hr) and directly connected impervious area (percent).

The **initial deficit** is the soil moisture deficit of the active soil layer at the onset of a storm event. The potential infiltration rate decreases linearly with cumulative infiltration until the initial deficit is satisfied. Once satisfied, the potential infiltration rate becomes constant (see constant rate below).

Table 1: Proposed values for initial deficit, sand, loamy sand, and sandy loam texture classes.

Antecedent soil moisture (m^3/m^3)	**Initial deficit (mm)**
0.02	48 (36-72)
0.06	36 (23-66)
0.10	23 (10-54)
0.14	10 (0-41)
0.18	0 (0-28)
0.22 *	0 (0-16)

* or presence of physical soil crust

Table 1 provides estimates for the initial deficit parameter for different antecedent soil moisture conditions and texture classes sand, sandy loam, and loamy sand. The values are based on plot-scale rainfall simulator testing (Schoener et al., Forthcoming) carried out at different test sites in New Mexico. The table contains median values, with reasonable ranges included in parenthesis. Rainfall simulator testing also revealed that physical soil crusts can substantially increase runoff, and crusted soils and crusted soils may be modeled equivalent to wet soils.

The **maximum deficit** specifies the maximum soil moisture deficit of the active soil layer. This value must be equal to or larger than the initial deficit. For event-based simulations, it will not impact model results. The maximum deficit can be interpreted as the porosity multiplied by the thickness of the active layer. Table 1 suggests a reasonable maximum deficit range for sands, sandy loams and loamy sands of 36-72 mm based on plot-scale rainfall simulator testing.

The **constant rate** determines the potential rate of infiltration that will occur after the initial deficit is satisfied. The same rate is applied regardless of the length of the simulation. The constant rate can be interpreted as the effective hydraulic conductivity of the active soil layer. Table 2 specifies proposed values of the constant rate parameter for texture classes sand, sandy loam, and loamy sand.

Table 2: Proposed values for constant rate parameter, sand, loamy sand, and sandy loam texture classes.

Texture class	***Constant rate*** **(mm/hr)**
Sand	31 (19-44)
Loamy sand	20 (14-26)

Texture class	***Constant rate* (mm/hr)**
Sandy loam	15 (11-21)

The **decay factor** is the rate of infiltration potential decay with respect to cumulative infiltration. The default value is -3, but users can change the decay factor values within the range of 0 to -8. Sensitivity analysis has shown that the LC loss method is substantially more sensitive to changes in the initial deficit parameter compared to decay factor, and that the latter may be held constant at -3 (paper under review).

Finally, the percentage of the subbasin comprised of directly connected impervious area can be specified. Directly connected impervious areas are surfaces where runoff is conveyed directly to a waterway or stormwater collection system. These surfaces differ from disconnected impervious areas where runoff encounters permeable areas which may infiltrate some (or all) of the runoff prior to reaching a waterway or stormwater collection system. No loss calculations are carried out on the specified percentage of the subbasin; all precipitation that falls on that portion of the subbasin becomes excess precipitation and subject to direct runoff.

8.7.7 A Note on Parameter Estimation

The parameter values presented in this section are intended as initial estimates only for the soil textures specified. Parameter guidance was developed for bare or sparsely vegetated soil plots in New Mexico under simulated rain. Evidence suggests that infiltration models can be parameterized successfully using plot-scale simulation under certain circumstances (Schoener et al., 2021). Nevertheless, initial estimates should be calibrated and validated using measured rainfall-runoff data whenever possible.

8.7.8 A Note on the Computational Algorithm in HEC-HMS

Potential infiltration rate over the duration of each simulation time step is calculated by averaging the potential infiltration rates at the beginning and at the end of the time step. Potential loss over the duration of the simulation time step is then calculated by assuming that the water infiltrates at the potential infiltration rate until the deficit is met. If the deficit is met before the end of the simulation time step, the water continues infiltrating at the percolation rate for the remainder of the simulation time step. The actual loss is then calculated as the minimum of the potential loss and the available precipitation. Deficit at the end of each time step is updated with the actual infiltration loss and any evaporation that happened over the time period.

8.8 SCS Curve Number Loss Model

8.8.1 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:

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

where Pe = accumulated precipitation excess at time t; P = accumulated rainfall depth at time t; Ia = 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 Ia and S:

$$I_a = 0.2 * S$$

Therefore, the cumulative excess at time t is:

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

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:

$$S = \begin{cases} \frac{1000-10CN}{CN} & (foot - pound\ system) \\ \frac{25400-254CN}{CN} & (\text{SI}) \end{cases}$$

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.

> Since no means for extracting infiltrated water is included, this method should only be used for event simulation.

8.8.2 Required Parameters

Parameters that are required to utilize the SCS curve number method include a curve number and directly connected impervious area [percent]. Optionally, I_a [in or mm] can be entered as well.

The curve number that is entered should be a “composite” curve number that represents all of the different soil group and land use combinations in the subbasin. This value should not include any impervious area that will be specified separately as the percentage of impervious area. Typically, curve numbers are derived from soils maps. I_a defines the amount of precipitation that must fall before excess precipitation results. If this value is not entered, it will be automatically calculated using:

$$I_a = 0.2 * S$$

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:

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

in which CNcomposite = the composite CN used for runoff volume computations; i = an index of watersheds subdivisions of uniform land use and soil type; CNi = the CN for subdivision i; and Ai = the drainage area of subdivision i.

> Users of the SCS model as implemented in the program should note that the tables in Appendix A[37] 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.

Finally, the percentage of the subbasin which is directly connected impervious area can be specified. Directly connected impervious areas are surfaces where runoff is conveyed directly to a waterway or stormwater collection system. These surfaces differ from disconnected impervious areas where runoff encounters permeable areas which may infiltrate some (or all) of the runoff prior to reaching a waterway or stormwater collection system. No loss calculations are carried out on the specified percentage of the subbasin; all precipitation that falls on that portion of the subbasin becomes excess precipitation and subject to direct runoff.

8.8.2.1 A Note on Parameter Estimation

The values presented here are meant as **initial estimates**. This is the same for all sources of similar data including Engineer Manual 1110-2-1417 Flood-Runoff Analysis[38] and the Introduction to Loss Rate Tutorials[39]. Regardless of the source, these initial estimates **must be calibrated and validated**.

8.9 Exponential Loss Model

8.9.1 Basic Concepts and Equations

The exponential loss method models the reduction in infiltration rate as an exponentially decreasing function of accumulated infiltration. This method is highly empirical and, in general, should not be used without prior calibration. Before using this method, consideration should be given to the Green and Ampt method because it produces a similar exponential decrease in infiltration and uses parameters with better physical interpretation. Also, this method should only be used for event simulation.

The potential loss rate [in/hr or mm/hr] at time t, f_t, is computed using:

37 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/cn-tables

38 https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1417.pdf?ver=VFC-A5m2Q18fxZsnv19U8g%3d%3d

39 https://www.hec.usace.army.mil/confluence/display/HMSGUIDES/Introduction+to+the+Loss+Rate+Tutorials

19) $$f_t = (AK + DLTK) * p_t^{ERAIN}$$

20) $$DLTK = 0.2 * DLTKR(1 - \frac{CUML}{DLTKR}))^2$$

21) $$AK = \frac{STRKR}{RTIOL^{0.1*CUML}}$$

where p_t = precipitation rate [in/hr or mm/hr] at time *t*, *ERAIN* = precipitation exponent, *AK* = loss rate coefficient at the beginning of the time interval, *DLTK* = incremental increase in the loss rate coefficient during the first *DLTKR* [in or mm] of accumulated loss, F_t. When F_t is greater than *DLTKR*, *DLTK* = 0. Note that there is no direct conversion between metric and English units for the coefficients used by this method. Consequently, separate calibrations excesses are required to derive site-specific coefficient for both unit systems.

8.9.2 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the initial range *DLTKR*, [in or mm], the initial coefficient *STRKR*, the coefficient ratio, the precipitation exponent *ERAI*N, and directly connected impervious area [percent]. The initial range (*DLTKR*) is the amount of initial accumulated infiltration during which the loss rate is increased. This parameter is considered to be a function primarily of antecedent soil moisture deficiency and is usually storm dependent. The initial coefficient (*STRKR*) specifies the starting loss rate coefficient on the exponential infiltration curve. It is assumed to be a function of infiltration characteristics and consequently may be correlated with soil type, land use, vegetation cover, and other properties of a subbasin. The coefficient ratio indicates the rate at which the exponential decrease in infiltration capability proceeds. It may be considered a function of the ability of the surface of a subbasin to absorb precipitation and should be reasonably constant for large, homogeneous areas. The precipitation exponent reflects the influence of precipitation rate on subbasin-average loss characteristics. It reflects the manner in which storms occur within an area and may be considered a characteristic of a particular region; this parameter varies between 0.0 and 1.0.

8.9.2.1 A Note on Parameter Estimation

The values presented here are meant as **initial estimates**. This is the same for all sources of similar data including Engineer Manual 1110-2-1417 Flood-Runoff Analysis[40] and the Introduction to Loss Rate Tutorials[41]. Regardless of the source, these initial estimates **must be calibrated and validated**.

40 https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1417.pdf?ver=VFC-A5m2Q18fxZsnv19U8g%3d%3d
41 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/introduction-to-the-loss-rate-tutorials

8.10 Smith Parlange

8.10.1 Basic Concepts and Equations

The Smith Parlange loss method is similar to the Green and Ampt method described previously. However, the Smith Parlange method approximates Richard's equation by assuming the wetting front can be represented with an exponential scaling of the saturated conductivity (Smith & Parlange, 1978). This linearization approach allows the infiltration computations to proceed very quickly while maintaining a reasonable approximation of the wetting front. The Smith Parlange loss method should only be used for event simulation. The governing equations of the Smith Parlange loss method are described in detail within Smith & Parlange (1978) and Smith, et al (2002).

8.10.2 Required Parameters

Parameters that are required to utilize the Smith Parlange method include the initial water content [in/in or mm/mm], the residual water content [in/in or mm/mm], the saturated water content [in/in or mm/mm], bubbling pressure [in or mm], the pore size distribution, the saturated hydraulic conductivity [in/hr or mm/hr], and directly connected impervious area [percent]. An optional temperature time series may be specified. If a temperature time series is selected, a beta zero parameter must also be specified.

The initial water content refers to the initial saturation of the soil at the beginning of a simulation and should be determined through model calibration. The residual water content specifies the amount of water remaining in the soil after all drainage by gravity has ceased. It should be specified in terms of volume ratio and is commonly estimated using the predominant soil texture. The saturated water content specifies the maximum water holding capacity in terms of volume ratio and is often assumed to be equivalent to the total porosity of the soil.

The bubbling pressure is similar to the wetting front suction head, which was previously discussed. The pore size distribution determines how the total pore space is distributed in different size classes. The saturated hydraulic conductivity was also previously described.

The optional temperature time series adjusts the water density, water viscosity, and matric potential based on temperature. If no temperature time series is specified, a default temperature of 25 degrees Celsius (75 deg Fahrenheit) is assumed to prevail. The beta zero parameter is used to correct the matric potential based on temperature. All of the aforementioned parameters are typically estimated using the predominant soil texture and literature values.

8.10.2.1 A Note on Parameter Estimation

The values presented here are meant as **initial estimates**. This is the same for all sources of similar data including Engineer Manual 1110-2-1417 Flood-Runoff Analysis[42] and the Introduction to Loss Rate Tutorials[43]. Regardless of the source, these initial estimates **must be calibrated and validated**.

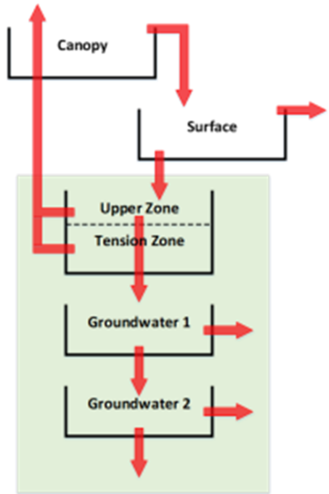
42 https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1417.pdf?ver=VFC-A5m2Q18fxZsnv19U8g%3d%3d

43 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/introduction-to-the-loss-rate-tutorials

8.11 Soil Moisture Accounting Loss Model

8.11.1 Basic Concepts and Equations

The soil moisture accounting (SMA) method simulates the movement and storage of water through the soil profile as well as multiple groundwater layers (Leavesley, Lichty, Troutman, & Saindon, 1983). The implementation of this method within HEC-HMS is described in detail within Bennett (1998). Using the soil moisture accounting method allows for continuous simulation when combined with a canopy method that will extract water from the soil in response to potential ET computed in the meteorologic model. When both a canopy and surface method are used in combination with the soil moisture accounting loss method, the system can be conceptualized as shown in the following figure.



14 Conceptual Representation of the Soil Moisture Accounting Method

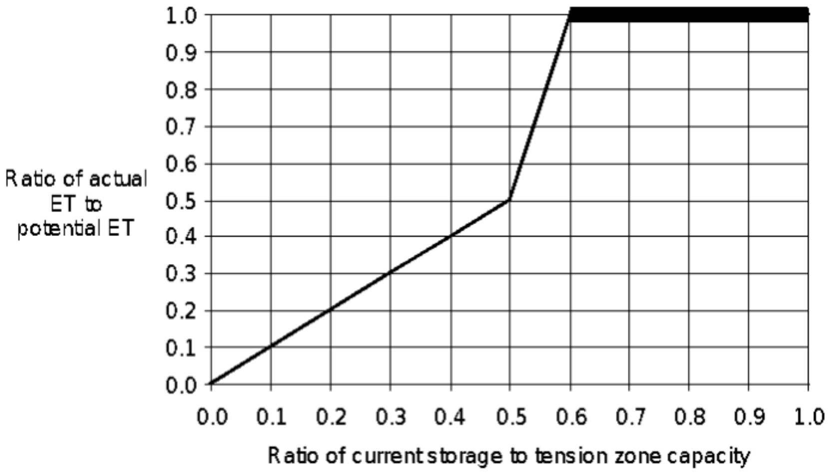
8.11.1.1 Storage Component

The SMA model represents the watershed with a series of storage layers, as illustrated above. 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 the figure below. 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.



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.

8.11.1.2 Flow Component

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

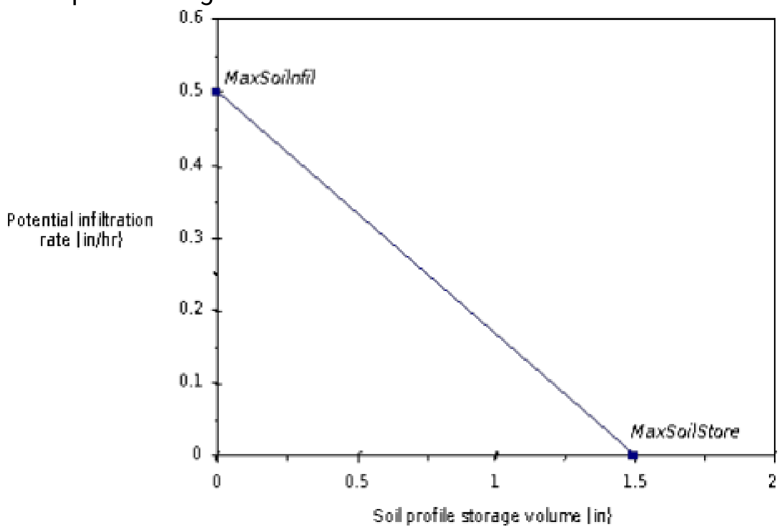
- Precipitation, which is an input to the system of storages. Precipitation first contributes to the canopy interception storage. If the canopy storage fills, the excess amount is then available for infiltration.

- Infiltration, which refers to the 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:

22)
$$PotSoiInfil = MaxSoilInfil - \frac{CurSoilStore}{MaxSoilStore} MaxSoilInfil$$

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.



The above figure 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, which refers to 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:

23)
$$PotSoilPerc = MaxSoilPerc(\frac{CurSoilStore}{MaxSoilStore})(1 - \frac{CurGW Store}{MaxGW Store})$$

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:

24) $$PotGWPerc = MaxPercGW(\frac{CurGWStore}{MaxGWStore})(1 - \frac{CurGWStore}{MaxGWStore})$$

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

25) $$PotSoilPerc = MaxSoilPerc\frac{CurSoilStore}{MaxSoilStore}$$

and

26) $$PotGWPerc = MaxPercGW\frac{CurGWStore}{MaxGWStore}$$

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

- Surface runoff, which is the water that exceeds the infiltration rate and overflows the surface storage. This volume of water is direct runoff.
- Groundwater flow, which 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:

27) $$GWFlow_{t+1} = \frac{ActSoilPerc + CurGW_iStore - PotGW_iPerc - \frac{1}{2}GWFlow_t * TimeStep}{RoutGW_iStore + \frac{1}{2}TimeStep}$$

where GwFlowt and GwFlowt+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; PotGwiPerc = potential percolation from groundwater layer i; RoutGwiStore = 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

28) $$GWVolume = \frac{1}{2}(GWFlow_{t+1} + GWFlow_t) * TimeStep$$

This volume may be treated as inflow to a linear reservoir model to simulate baseflow, as described in the Linear Reservoir Model[44] section.

- Evapotranspiration (ET), which 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:

29)
$$ActEvapSoil = PotEvapSoil * f(CurSoilStore, MaxTenStore)$$

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 8 is defined as follows:

- 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 onetoone rate, the same as losses from canopy and surface interception.
- 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.

8.11.1.3 Order of Model Computations

Flow into and out of storage layers is computed for each time step in the SMA model. 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

44 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/baseflow/linear-reservoir-model

removed from the upper-soil profile storage. The model then continues as described above for the precipitation periods.

8.11.2 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the amounts of storage within the soil, groundwater 1, and groundwater 2 layers that are initially filled [percent], the maximum infiltration rate [in/hr or mm/hr], directly connected impervious area [percent], the maximum soil storage [in or mm], tension storage [in or mm], the maximum soil percolation rate [in/hr or mm/hr], the maximum groundwater layer 1 storage [in], the maximum groundwater layer 1 percolation rate [in/hr or mm/hr], the groundwater layer 1 coefficient [hr], the maximum groundwater layer 2 storage [in], the maximum groundwater layer 2 percolation rate [in/hr or mm/hr], and the groundwater 2 layer coefficient [hr].

The amount of initial storage refers to the initial saturation of each layer at the beginning of a simulation and should be determined through model calibration. The maximum infiltration rate sets the upper bound on infiltration from the surface storage into the soil. This is the upper bound on infiltration; the actual infiltration in a particular time interval is a linear function of the surface and soil storage, if a surface method is selected. Without a selected surface method, water will always infiltrate at the maximum rate. Soil storage represents the total storage available in the soil layer. Tension storage specifies the amount of water storage in the soil that does not drain under the effects of gravity. Percolation from the soil layer to the upper groundwater layer will occur whenever the current soil storage exceeds the tension storage. Water in tension storage is only removed by ET. By definition, tension storage must be less than soil storage. The soil percolation sets the upper bound on percolation from the soil storage into the upper groundwater layer. The actual percolation rate is a linear function of the current storage in the soil and the current storage in the upper groundwater layer. The maximum groundwater layer 1 storage represents the total storage in the upper groundwater layer. The groundwater layer 1 percolation rate sets the upper bound on percolation from the upper groundwater into the lower groundwater layer. The groundwater layer 2 layer percolation rate sets the upper bound on deep percolation out of the system. The aforementioned parameters are typically estimated using the predominant soil texture and literature values. The groundwater layer 1 and groundwater layer 2 coefficients are used as the time lag on a linear reservoir for transforming water in storage to lateral outflow.

8.11.2.1 A Note on Parameter Estimation

The values presented here are meant as **initial estimates**. This is the same for all sources of similar data including Engineer Manual 1110-2-1417 Flood-Runoff Analysis[45] and the Introduction to Loss Rate Tutorials[46]. Regardless of the source, these initial estimates **must be calibrated and validated**.

8.12 Applicability and Limitations of the Runoff-Volume Methods

Selecting a loss method and estimating the required parameters are critical steps in developing program input. Not all loss methods can be used with all transforms. For instance, the gridded loss methods can only be used with the ModClark transform. Table 16 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 methods and the watershed. League and Freeze (1985) point out that "In many ways, hydrologic modeling is

45 https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1417.pdf?ver=VFC-A5m2Q18fxZsnv19U8g%3d%3d

46 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-loss-methods-within-hec-hms/introduction-to-the-loss-rate-tutorials

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 method 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 method 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."

The following table contains a list of various advantages and disadvantages regarding the aforementioned loss methods available for use within HEC-HMS. However, these are only guidelines and should be supplemented by knowledge of, and experience with, the methods and the watershed in question.

Method	Advantages	Disadvantages
Initial and Constant	• "Mature" method that has been used successfully in thousands of studies throughout the U.S. • Easy to set up and use. • Parameters can be related to predominant soil textures and estimated using multiple literature sources. • Method is parsimonious; it includes only a few parameters necessary to explain the variation of runoff volume.	• Difficult to apply to ungaged areas due to lack of direct physical relationship of parameters and watershed properties. • Method may be too simple to predict losses within event, even if it does predict total losses well. • Does not allow for continuous simulation.
Deficit and Constant	• Similar to advantages of the Initial and Constant method. • Method is scalable in that it allows for continuous simulation (but is not required for use).	• Similar to disadvantages of the Initial and Constant method.
Green and Ampt	• Parameters can be related to predominant soil textures and estimated using multiple literature sources. • Predicted values are in accordance with classical unsaturated flow theory (good for ungaged watersheds).	• Not widely used, so less mature. • Not as much experience in professional community as simpler methods. • Less parsimonious than simpler methods. • Does not allow for continuous simulation.
Layered Green and Ampt	• Similar to advantages of the Green and Ampt method. • Allows for continuous simulation.	• Similar to disadvantages of the Green and Ampt method. • Requires more parameters than the Green and Ampt method.

Method	Advantages	Disadvantages
SCS Curve Number	• 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 understood and well-documented. • Well established method widely accepted for use in U.S. and abroad. • Parameters can be related to predominant soil group/land use and estimated using multiple literature sources.	• Predicted values not in accordance with classical unsaturated flow theory (infiltration rate will approach zero during a storm of long duration rather than a constant rate). • Developed with data from small agricultural watersheds in midwestern U.S., so applicability elsewhere is uncertain. • Default initial abstraction (0.2*S) does not depend upon storm characteristics or timing. • Rainfall intensity is not considered (same loss for 1 in rainfall in 1 hour or 1 day). • Does not allow for continuous simulation.
Exponential	• Predicted values are in accordance with classical unsaturated flow theory.	• Similar to disadvantages of the Green and Ampt method. • Requires more parameters than Green and Ampt. • Parameters cannot be related to predominant soil textures and estimated using (not good for ungaged watersheds). • Does not allow for continuous simulation.
Smith Parlange	• Similar to advantages of the Green and Ampt method.	• Similar to disadvantages of the Green and Ampt method. • Requires more parameters than the Green and Ampt method. • Does not allow for continuous simulation.
Soil Moisture Accounting	• Parameters can be estimated for ungaged watersheds from information about soils. • Predicted values are in accordance with classical unsaturated flow theory (good for ungaged watersheds). • Allows for continuous simulation.	• Not widely used, so less mature, not as much experience in professional community. • Features not widely understood. • Less parsimonious than simple empirical methods.

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9 Transform

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 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 models included in the program are the Kinematic Wave model of overland flow and the Two-Dimensional (2D) Diffusion Wave model. They represent, 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, in the case of the Kinematic Wave transform).

9.1 Unit Hydrograph Basic Concepts

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 unit hydrograph 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:

30)

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

where Qn = storm hydrograph ordinate at time n Δt; P_m = rainfall excess depth in time interval m Δt to (m+1) Δt; M = total number of discrete rainfall pulses; and U_{n-m+1} = unit hydrograph ordinate at time (n-m+1) Δt. 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.

9.1.1 Parametric vs. Synthetic Unit Hydrographs

The alternative to specifying the entire set of unit hydrograph ordinates is to use a parametric UH. A parametric unit hydrograph defines all pertinent unit hydrograph 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 unit hydrograph ordinates. For example, to approximate the unit hydrograph with a triangle shape, all the ordinates can be described by specifying:

- Magnitude of the unit hydrograph peak.
- Time of the unit hydrograph peak.

The volume of the unit hydrograph 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 unit hydrograph can be computed through simple linear interpolation. Other parametric unit hydrograph are more complex, but the concept is the same.

A synthetic unit hydrograph relates the parameters of a parametric unit hydrograph model to watershed characteristics. By using the relationships, it is possible to develop a unit hydrograph 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 unit hydrograph model may relate the unit hydrograph peak of the simple triangular unit hydrograph to the drainage area of the watershed. With the relationship, an estimate of the unit hydrograph peak for any watershed can be made given an estimate of the drainage area. If the time of unit hydrograph peak and total time base of the unit hydrograph is estimated in a similar manner, the unit hydrograph can be defined "synthetically" for any watershed. That is, the unit hydrograph 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 unit hydrograph fall into three categories:

1. Those that relate unit hydrograph characteristics (such as unit hydrograph peak and peak time) to watershed characteristics. The Snyder unit hydrograph is such a synthetic UH.
2. Those that are based upon a dimensionless UH. The SCS unit hydrograph is such a synthetic UH.
3. Those that are based upon a quasi-conceptual accounting for watershed storage. The Clark unit hydrograph and the ModClark model do so.

All of these synthetic unit hydrograph models are included in the program.

9.2 Available Transform Methods

While a subbasin element conceptually represents infiltration, surface runoff, and subsurface processes interacting together, the actual surface runoff calculations are performed by a transform method contained within the subbasin. A total of eight different transform methods are provided including six that use unit hydrograph theory:

9.3 User-Specified Unit Hydrograph

9.3.1 Basic Concepts and Equations

The user-specified unit hydrograph method uses a defined time vs. discharge curve to route excess precipitation to the subbasin outlet using Equation 1.

31)

$$Q_n = \sum_{m=1}^{n \leq M} P_m U_{n-m+1}$$

When using this method, all ordinates of the unit hydrograph are explicitly defined by the user. Modifications to the effective duration of the specified unit hydrograph are required when the rate at which precipitation is applied (e.g. 15-minutes) differs from the effective duration of the derived unit hydrograph (e.g. 6-hours). S-curves (or S-graphs) are used to change the effective duration of unit hydrographs (Morgan & Hullinghorst, 1939). Within HEC-HMS, a cubic spline (along with a number of passes) can be used to smooth the S-curve.

However, in practical applications, shortening the duration of empirically-derived unit hydrographs (i.e. going from a longer duration to a shorter duration) is difficult due to the presence of numerical oscillations within S-curves (Hunt, 1985).

9.3.2 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the unit hydrograph (which is a paired data object) in addition to a number of passes. The unit hydrograph paired data object requires the specification of an observation interval [hours] in addition to a duration of uniform excess precipitation [hours].

9.3.2.1 Estimating the Model Parameters

The unit hydrograph for a watershed is properly derived from observed rainfall and runoff, using deconvolution—the inverse solution of the convolution equation. To estimate a unit hydrograph 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 a duration sufficient to ensure that the entire watershed is responding. This duration, T, is the duration of the unit hydrograph 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 unit hydrograph.

Chow, Maidment, and Mays (1988) present matrix algebra, linear regression, and linear programming alternatives to this approach. With any of these approaches, the unit hydrograph derived is appropriate only for analysis of other storms of duration T. To apply the unit hydrograph to storms of different duration, the unit hydrograph for these other durations must be derived. If the other durations are integral multiples of T, the new unit hydrograph can be computed by lagging the original unit hydrograph, 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.

9.3.3 Application of the User-Specified Unit Hydrograph

In practice, direct runoff computation with a specified-unit hydrograph is uncommon. The data necessary to derive the unit hydrograph in the manner described herein are seldom available, so the unit hydrograph 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 unit hydrograph-determination procedures described are difficult to apply. Finally, to provide information for many water resources development activities, a unit hydrograph for alternative watershed land use or channel conditions is often needed—data necessary to derive a unit hydrograph for these future conditions are never available.

9.4 User-Specified S-Graph

9.4.1 Basic Concepts and Equations

The user-specified S-graph (or S-curve) method uses a defined curve that represents the percentage of a summation hydrograph to route excess precipitation to the subbasin outlet. When used in this particular way, the S-curve is defined with the percentage of the watershed lag time, t_{lag}, as the independent variable (x-axis) and percentage of the cumulative runoff volume as the dependent variable (y-axis). t_{lag} [hours] is defined as the length of time between the centroid of precipitation mass and the peak flow of the resulting hydrograph. t_{lag} can either be directly specified or through the use of a regression equation which takes the following form:

32)
$$t_{lag} = C\left(\frac{L * L_{ca}}{S^p}\right)^m$$

where C = a coefficient that describes the hydraulic efficiency of the stream channel, L = length along the stream centerline from the outlet to the watershed boundary [miles or kilometers], L_{ca} = length along the stream centerline from the outlet to a point on the stream nearest the centroid of the watershed [mi or km], and S = slope of the main watercourse [feet/mile or km/m]. The exponents, *m* and *p*, must be derived from gaged data within the region of interest. Computationally, the S-curve is scaled by t_{lag} and successive differences are taken along the curve to compute a unit hydrograph. The resultant unit hydrograph is then used to route excess precipitation to the watershed outlet.

9.4.2 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the percentage curve (which is a paired data object), a lag time [hours] (when using the Standard method), and/or the aforementioned C, L, L_{ca}, and S coefficients and *m* and *p* exponents (when using the Regression method).

9.5 Clark Unit Hydrograph Model

The Clark unit hydrograph method utilizes the concept of an instantaneous unit hydrograph to route excess precipitation to the subbasin outlet. An instantaneous unit hydrograph is derived by instantaneously applying a unit depth (e.g. one inch) of excess precipitation over a watershed (Clark, 1945). The resultant unit hydrograph is entirely theoretical (i.e. real precipitation cannot be applied instantaneously to a watershed) but it has the distinct advantage of characterizing the watershed’s response to rainfall without reference to the duration of excess precipitation (Chow, Maidment, & Mays, 1988). This method explicitly represents two critical processes in the transformation of excess precipitation to runoff: 1) the translation (or movement) of excess precipitation from its origin throughout the watershed to the outlet and 2) the attenuation (or reduction) of the magnitude of the discharge as the excess precipitation is temporarily stored throughout the watershed (Hydrologic Engineering Center, 2000). Conceptually, water is translated from remote points to the watershed outlet with delay but without attenuation. Attenuation is then incorporated, conceptually speaking, at the watershed outlet. Three parameters are utilized within this method:

- Time of concentration (T_c), which is equivalent to the time it takes for excess precipitation to travel from the hydraulically-most remote point of the watershed to the outlet,
- Watershed storage coefficient (R), which is equivalent to attenuation due to storage effects throughout the watershed (Kull & Feldman, 1998), and
- Time-Area histogram, which represents the watershed area that contributes to flow at the outlet as a function of time.

9.5.1 Basic Concepts and Equations

The time-area histogram is used in conjunction with T_c to first transform excess precipitation into runoff. While a user-specified time-area histogram can be used, a smooth function fitted to a typical time-area relationship can oftentimes adequately represent the temporal distribution of flow (i.e. translation) for most watersheds and is provided as a default :

33)
$$\frac{A_t}{A} = \begin{cases} 1.414\left(\frac{t}{T_c}\right)^{1.5} & \text{for } t \le \frac{T_c}{2} \\ 1 - 1.414\left(1 - \frac{t}{T_c}\right)^{1.5} & \text{for } t \ge \frac{T_c}{2} \end{cases}$$

where A_t = cumulative watershed area contributing at time *t* and A = total watershed area. This typical time-area relationship was derived from an elliptically-shaped watershed. Through the use of this simplified time-area histogram, only T_c and R are required to completely define the instantaneous unit hydrograph for a watershed. As described within **Modified Clark Unit Hydrograph**, through the use of GIS and the Modified Clark method, watershed-specific time-area histograms can be efficiently created and used.

After translation, attenuation is incorporated using a linear reservoir model which begins with the continuity equation :

34)
$$\frac{dS}{dt} = I_t - O_t$$

where 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* can be related to outflow :

35)
$$S_t = RO_t$$

where R = a constant linear reservoir parameter. Combining and solving Equation 2 and Equation 3 using a finite difference approximation yields :

36)
$$O_t = C_A I_t + C_B O_{t-1}$$

where C_A and C_B = routing coefficients. The coefficients are then calculated according to :

37)
$$C_A = \frac{\Delta t}{R + 0.5\Delta t}$$
$$C_B = 1 - C_A$$

Finally, the average outflow during period *t* is then calculated as :

38)
$$\bar{O}_t = \frac{O_{t-1} + O_t}{2}$$

where O_{t-1} = outflow from the previous time step. Solving Equation 4 and Equation 6 recursively yields values of $\bar{O}_t$. However, if the inflow ordinates in Equation 4 are runoff from a unit of excess, the values of $\bar{O}_t$ are, in fact, a unit hydrograph. As the solution is recursive, outflow will theoretically continue for an infinite duration. Within HEC-HMS, computation of the unit hydrograph ordinates continues until the volume of the outflow exceeds 0.995 inches or mm. The unit hydrograph ordinates are then adjusted using a depth-weighted consideration to produce a unit hydrograph with a volume exactly equal to one unit of depth.

9.5.1.1 Variable Clark

According to Sherman (1932), the unit hydrograph of a watershed 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." This implies that ordinates of any hydrograph resulting from a quantity of runoff-producing rainfall of a defined duration would be equal to corresponding ordinates of a unit hydrograph for the same areal distribution of rainfall, multiplied by the ratio of excess precipitation values (U.S. Army Corps of Engineers, 1959). However, due to differences in areal distributions of rainfall and hydraulic reactions between large and small precipitation events, the corresponding unit hydrographs have not been found to be equal, as implied by unit hydrograph theory (Minshall, 1960), (U.S. Army Corps of Engineers, 1991), (Harrison, 1999), (England, Klawon, Klinger, & Bauer, 2006). Typically, as excess precipitation rates increase, translation of a flood wave becomes more rapid while attenuation decreases resulting in a more peaked unit response at the location of interest (Rodriguez-Iturbe & Valdes, 1979), (Meyersohn, 2016).

The "Variable" method can be used to avoid the aforementioned limitations by allowing both T_c and R to change as excess precipitation rates increase or decrease. Anticipated increases and decreases in translation time and/or attenuation can be simulated with excess precipitation-dependent T_c and R relationships. When used during extreme event simulations, this allows the unit response to vary. The runoff response achieved with the Variable Clark method has been shown to achieve results that are similar to those of much more complex two-dimensional routing methods in a fraction of the computational time (Bartles, 2016).

> Additional discussion regarding this method, including means by which Variable Clark parameters can be estimated for ungaged locations in California, can be found here: Bartles and Meyersohn (2023)[47]

9.5.1.2 Maricopa County, AZ Method

The "Maricopa County" method makes use of the following equations to estimate T_c and R :

39) $$T_c = 11.4 * L^{0.5} * K_b{}^{0.52} * S^{-0.31} * i^{-0.38}$$

40) $$R = 0.37T_c{}^{1.11} * A^{-0.57} * L^{0.8}$$

where *L* is the length of the hydraulically longest flow path [mi or km], *S* is the watercourse slope of the longest flow path [ft/mi or m/km], K_b is the resistance coefficient, and *i* is the average excess precipitation intensity [in/hr or mm/hr]. This approach should only be utilized for simulations within Maricopa County, AZ.

47 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstr/files/76908661/139730202/1/1684268892597/Bartles_Meyersohn_CA_Unit_Hydrograph_Regression_Equations.pdf

9.5.2 Required Parameters

Parameters that are required to utilize the “Standard” Clark method within HEC-HMS include T_c [hours], R [hours], and an optional time-area curve. T_c and R are commonly estimated using watershed characteristics and regression equations. If the “Variable Parameter” method is chosen, an index excess precipitation [inches], excess precipitation vs. T_c relationship, and excess precipitation vs. R relationship is required. The index excess precipitation is used to relate the specified T_c and R values against the variable parameter relationships. Typically, this value is set equal to 1 inch/hour (or 1 mm/hr). The excess precipitation vs. T_c relationships are commonly derived from two-dimensional simulations. If the “Maricopa County” method is chosen L, K_b, and S must be specified by the user. After running a preliminary simulation, the top ten highest five-minute time interval excess precipitation values will be extracted and used to compute T_c and R.

A tutorial describing an example application of this transform method, including parameter estimation and calibration, can be found here: Estimating Clark Unit Hydrograph Parameters[48].

9.5.2.1 Estimating Parameters

The Clark method employs several parameters including T_c, R, and a time-area histogram. Some of these parameters have physical meaning while others do not. Due to that fact, differing methods have been used in practice to initially estimate these parameters. Also, modifications to these parameters have been found to be warranted for use within extreme event simulations to account for non-linear routing phenomena that have been observed. Initial parameter estimates are typically made using GIS and various terrain, land use, hydrography, and watershed boundary layers.

The most commonly used terrain data is distributed by the USGS as the National Elevation Dataset (NED). The NED includes multiple layers at varying horizontal resolutions. Typically, a horizontal resolution of 1/3rd arc second (approximately 10 meters) is appropriate for use within hydrologic applications that encompass 100 to 1,000 square miles. However, the most appropriate horizontal resolution for each application is specific to the study and/or watershed physical characteristics.

The most commonly used hydrography and watershed boundary data is also distributed by the USGS as the National Hydrograph Dataset (NHD) and the Watershed Boundary Dataset (WBD). The NHD and WBD also include multiple versions at varying resolutions. The NED, NHD, and WBD can be accessed through the USGS National Map: “https://viewer.nationalmap.gov/basic/”.

9.5.2.1.1 Time of Concentration

T_c can be estimated for a watershed in multiple ways. The most commonly used methods include:

- Computing a flow length and assuming a constant velocity,
- Computing a flow length and discretizing changes in velocity, and
- Using regional regression equations which were developed from observed data in a similar region.

48 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/estimating-clark-unit-hydrograph-parameters

Travel time, T_t, refers to the amount of time necessary for runoff to move from one location to another within a watershed and can be computed using the following:

41) $$T_t = \frac{L}{3600 * V}$$

where T_t = travel time [hours], L = flow length [feet], 3600 = conversion factor from seconds to hours, and V = average velocity [feet/second]. An average velocity can be estimated using multiple approaches including simplistic nomographs (Natural Resources Conservation Service, 1999), Kinematic Wave Theory (Hydrologic Engineering Center, 1993), and/or Manning's equation.

However, as runoff moves down gradient, predominant flow regimes tend to change due to factors like channel shape, slope, roughness, and contributing drainage area. For instance, runoff that begins as sheet flow may transition to shallow concentrated flow after a short distance. As the flow regime changes, velocity tends to change as well. When non-uniform velocities are expected to occur within a watershed, it becomes computationally convenient to discretize the predominant length over which each flow regime tends to occur, thus allowing for more accurate computations of T_c. This method is summarized within NRCS Technical Release (TR) 55 (Natural Resources Conservation Service, 1999).

Within this method, three flow regimes are typically discretized: 1) sheet flow, 2) shallow concentrated flow, and 3) open channel flow. T_c can then be computed as:

42) $$T_c = T_{sheet} + T_{shallow} + T_{channel}$$

where T_{sheet} = sum of travel time in sheet flow segments, $T_{shallow}$ = sum of travel time in shallow flow segments (e.g. streets, gutters, shallow rills, etc), $T_{channel}$ = sum of travel time in open channel flow segments. Equation 7 assumes that T_c is derived from the longest flow path within the watershed.

Sheet flow can be conceptualized as flow that moves over planar surfaces with flow depths that are less than 0.1 feet (Natural Resources Conservation Service, 1999). An approximation of the Kinematic Wave equations can be used to estimate sheet flow travel time [hours]:

43) $$T_{sheet} = \frac{0.007(N * L)^{0.8}}{\sqrt{(P_{2-10})} * S_f^{0.4}}$$

where N = overland flow roughness coefficient, L = flow length [feet], P_{2-10} = ½ annual exceedance probability 24-hour duration rainfall [inches], and S_f = friction slope [ft/ft]. The overland flow roughness coefficient is typically much larger than an equivalent Manning's roughness coefficient for the same land use/channel material. Typical overland flow roughness coefficients for multiple land uses are presented in Technical Document 10 (Hydrologic Engineering Center, 1993) and (Natural Resources Conservation Service, 1999).

After a short distance, sheet flow transitions to shallow concentrated flow. The distance over which sheet flow occurs is commonly assumed to be less than 300 feet but can vary depending upon site-specific conditions (Natural Resources Conservation Service, 1999). Relationships presented in TR-55 can be used to estimate the average velocity for shallow concentrated flow:

44) $$V_{\text{shallow}} = \begin{cases} 16.1345 * \sqrt{S}, & \text{for unpaved surfaces} \\ 20.3282 * \sqrt{S}, & \text{for paved surfaces} \end{cases}$$

where $V_{shallow}$ = average velocity in shallow flow segments [ft/s] and S = watercourse slope [ft/ft]. Once $V_{shallow}$ has been calculated, the travel time of shallow concentrated flow can be estimated. The point at which shallow concentrated flow transitions to open channel flow is typically assumed to exist where evidence of channels can be obtained from field surveys, maps, or aerial photographs. However, this transition can vary depending upon site-specific conditions.

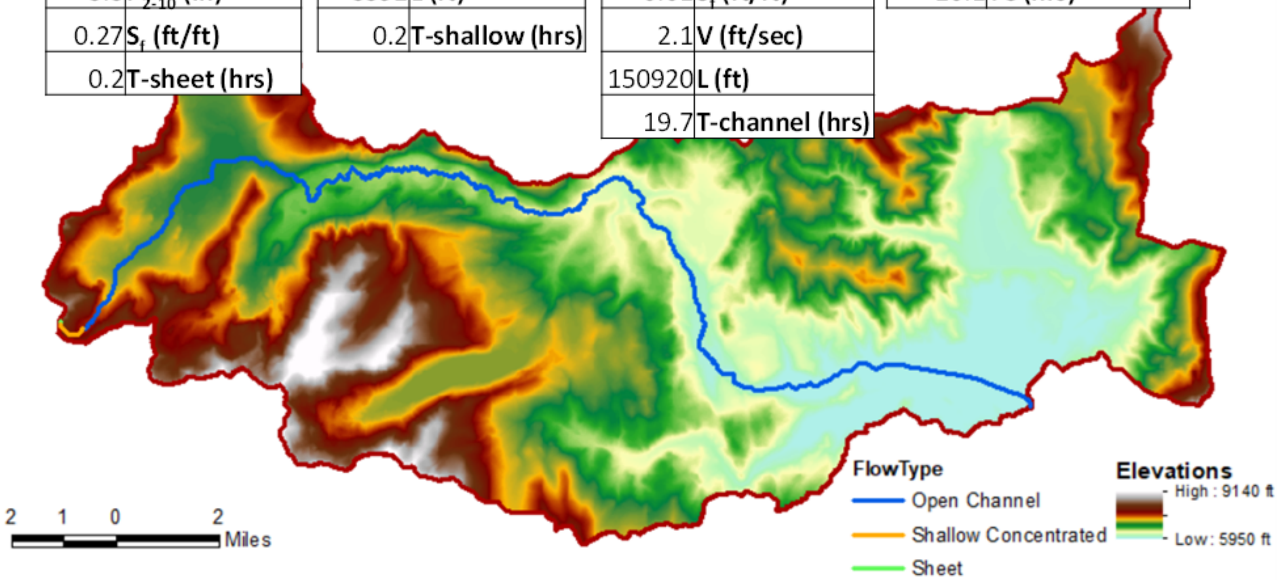
The average velocity for open channel flow can be estimated using Mannings Equation and a normal depth assumption. Manning's roughness coefficients for common channel and overbank materials are presented in numerous sources including Chow (1959). Once $V_{channel}$ has been calculated, the travel time of channel flow can also be estimated. Finally, once all travel times have been computed, T_c can be estimated. An example application of this methodology is shown in Figure below.

Sheet	
0.2	N
300	L (ft)
3.5	$P_{2\text{-}10}$ (in)
0.27	S_f (ft/ft)
0.2	T-sheet (hrs)

Shallow Conc.	
0.15	S (ft/ft)
6.23	V (ft/sec)
3392	L (ft)
0.2	T-shallow (hrs)

Open Channel	
0.045	n
0.95	R (ft)
0.01	S_f (ft/ft)
2.1	V (ft/sec)
150920	L (ft)
19.7	T-channel (hrs)

0.2	T-sheet (hrs)
0.2	T-shallow (hrs)
19.7	T-channel (hrs)
20.1	TC (hrs)



16 Example Tc Computation Using TR-55 Procedures

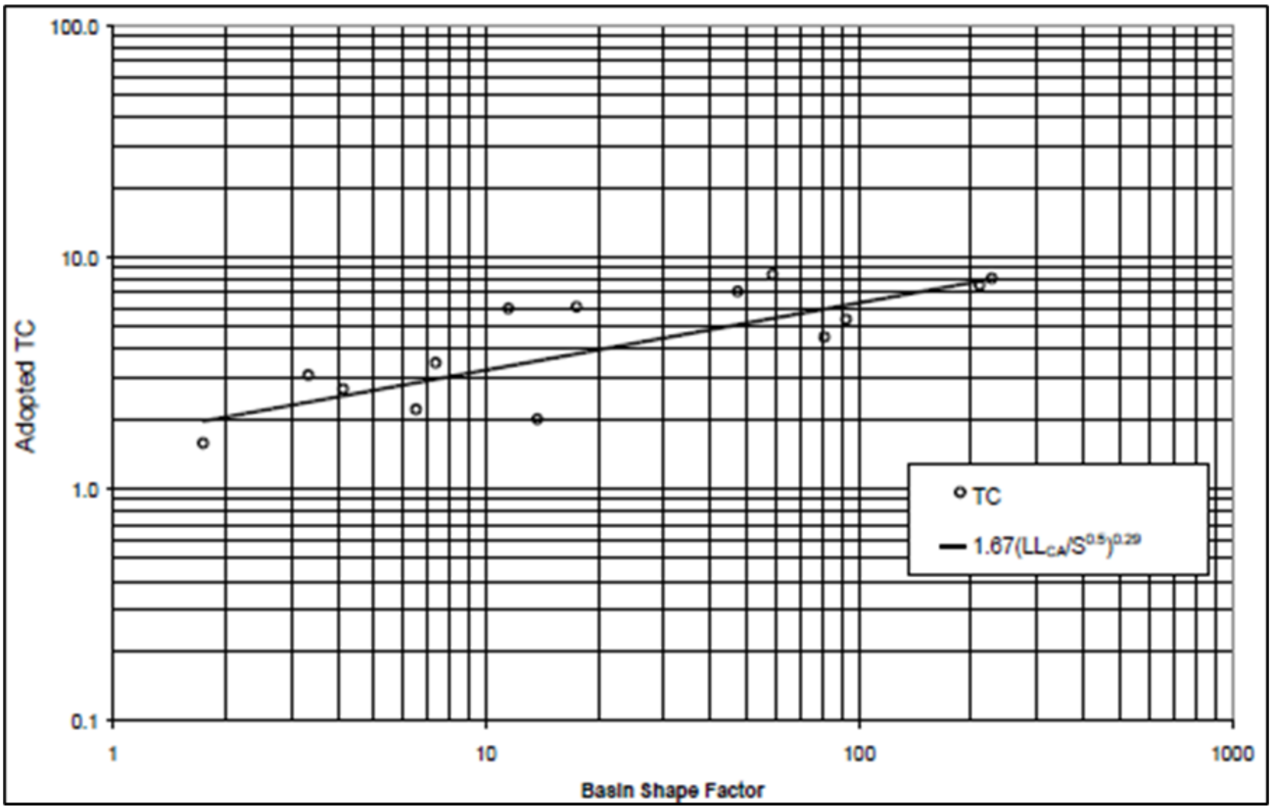
Finally, similar to the Snyder method, T_c can also be estimated through multi-linear regression analyses using various watershed physical characteristics and combinations. An example of this approach is shown within Figure below. The use of the results from regression analyses has the added advantage of allowing parameter estimation within ungaged watersheds. Typical relationships derived to estimate T_c [hours] follow the form:

45)

$$T_c = C\left(\frac{L * L_{ca}}{\sqrt{(S_{10-85})}}\right)^X$$

where C and X are parameters derived from gaged data in the same region, L = length along the stream centerline from the outlet to the watershed boundary [miles], L_{ca} = length along the stream centerline from the outlet to a point on the stream nearest the centroid of the watershed [miles], and $S_{10\text{-}85}$ = stream slope between points at 10- and 85-percent of the total distance [ft/mi].

When large standing bodies of water exist within the watershed of interest, flood waves may move downstream more rapidly than predicted within the aforementioned approach. As such, the application of the aforementioned approach may result in an over prediction of T_c. This potential over prediction can be addressed in one or more ways including: 1) refinement of initial estimates through model calibration and validation, 2) adding one or more additional flow regime transitions where flood waves are expected to travel through the standing bodies of water at a faster rate, 3) reducing the length over which open channel flow is assumed to occur, and/or 4) further discretizing the watershed (i.e. subdividing).



17 Using Multi-Linear Regression to Estimate Tc

Reproduced from (Hydrologic Engineering Center, 2001)

9.5.2.1.2   Watershed Storage Coefficient

Though R has units of time, there is only a qualitative meaning for it in a physical sense. Clark 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 (Clark, 1945). This parameter is commonly estimated through multi-linear regression analyses in conjunction with T_c using the following:

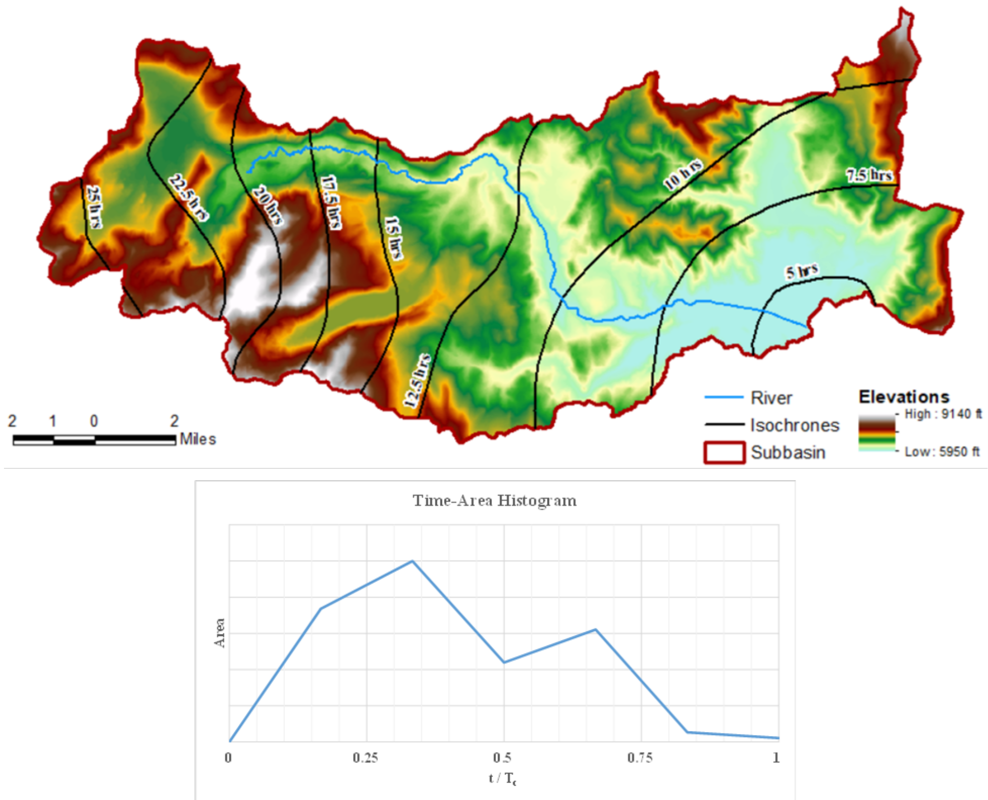
46)

$$X = \frac{R}{T_c + R}$$

where X = a coefficient that is determined through regional analyses. Smaller values of X result in short, steeply rising unit hydrographs and may be representative of urban watersheds. Larger values of X result in broad, slowly rising unit hydrographs and may be representative of flat, swampy watersheds. Values for X have been shown to vary due to factors like predominant channel shape, slope, and roughness; however, this coefficient has been found to be fairly constant on a regional basis (Hydrologic Engineering Center, 1988), (U.S. Army Corps of Engineers, 1994), and (Hydrologic Engineering Center, 2001). Regional regression equations for estimating watershed storage coefficients for the multiple hydrologic regions have been developed for California.

9.5.2.1.3 Time-Area Histogram

Studies at HEC have shown that a smooth function fitted to a typical time-area relationship can oftentimes adequately represent the temporal distribution of flow (i.e. translation) for most watersheds. A default time-area relationship is included within HEC-HMS and is shown in Equation 1. If a site-specific time-area histogram is required, it can be developed by demarcating lines of equal travel time, which are called "isochrones", to divide the watershed. Then, the watershed area encompassed by each isochrone and the watershed boundary is estimated. This process is demonstrated for an example watershed within Figure below.



18 Development of a Site-Specific Time-Area Histogram

As previously mentioned, when large standing bodies of water exist within the watershed of interest, flood waves may move downstream more rapidly than predicted within the aforementioned approach. As such, the temporal distribution of flow may be over predicted. This potential over prediction can be addressed in one or more ways including: 1) refinement of initial parameter estimates through model calibration and validation, 2) further discretizing the watershed (i.e. subdividing), and/or 3) modifying the time-area histogram to reduce the travel time through the standing body of water.

9.6 ModClark Model

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.

9.6.1 Basic Concepts and Equations

The Modified Clark (ModClark) method explicitly accounts for variations in travel time to the watershed outlet using a gridded representation of the watershed to route excess precipitation to the subbasin outlet (Kull & Feldman, 1998). While the method integrates spatial variability in travel time, the underlying flow velocity field is time invariant. The travel time index for each grid cell, $T_{t,cell}$, is set relative to the time of concentration for the watershed, $T_{c,watershed}$, using a distance index:

47)
$$T_{t,\text{ cell}} = T_{c,\text{ watershed}} \frac{D_{cell}}{D_{\max}}$$

where D_{cell} is the travel distance from a grid cell to the watershed outlet and D_{max} is the travel distance for the grid cell that is most distant from the watershed outlet. As long as the area for each grid cell is also specified, the volume of inflow can be computed as the product of area and excess precipitation. This, in essence, creates a watershed-specific time-area histogram instead of using a smooth function fitted to a typical time-area relationship. Inflows are then routed through a linear reservoir yielding an outflow hydrograph for each grid cell. The current implementation within HEC-HMS assumes the watershed storage coefficient, R, is uniform throughout each grid cell for a watershed (Hydrologic Engineering Center, 2018). The individual outflow hydrographs from each grid cell are then summed to determine the total direct runoff hydrograph.

9.6.2 Required Parameters

Parameters that are required to utilize the ModClark method within HEC-HMS include T_c [hours], R [hours], and a gridded representation of the watershed. This gridded representation must use one of the following systems:

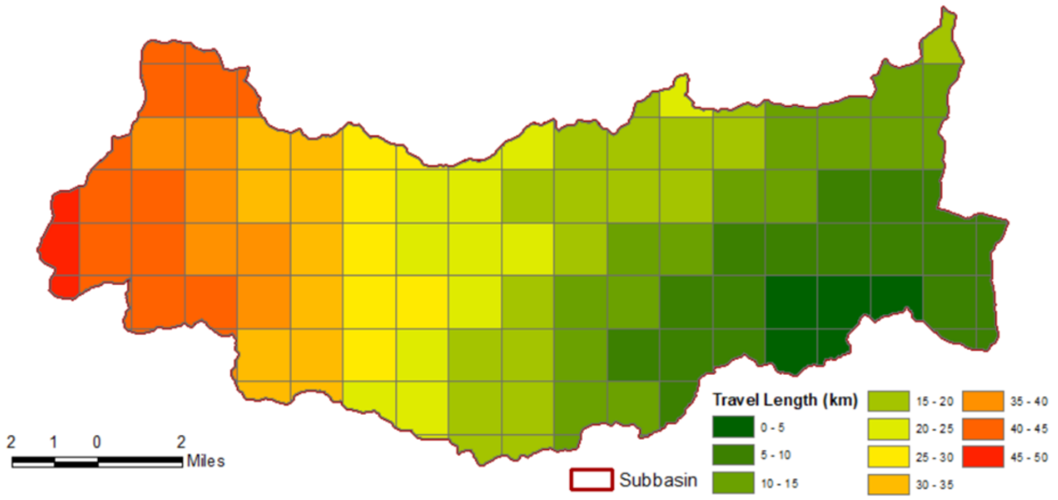
- Hydrologic Rainfall Analysis Project (HRAP), which is based on the Polar Stereographic map projection and is most commonly used by the National Weather Service
- Standard Hydrologic Grid (SHG), which is defined for the conterminous United States and is based on the Albers Equal Area Conic map projection
    - SHG grid resolutions of 10, 20, 50, 100, 200, 500, 1000, 2000, 5000, and 10000 meters are supported.
- Universal Transverse Mercator (UTM), which uses a transverse Mercator projection and divides the Earth into 60 zones, each being six degrees longitude in width (Hydrologic Engineering Center, 2013)

The gridded representation of the watershed must contain the following four items:

- X-coordinate of the cell centroid,
- Y-coordinate of the cell centroid,
- Travel Length, and
- Area

HEC-HMS version 4.4 (or later) should be used to create this gridded file. An example of this process can be found here: Applying Gridded Precipitation to a Non-Georeferenced Project - Structured Discretization[49].

An example of this gridded data, using the SHG system with 2000 meter x 2000 meter grid cells, is shown below.



19 Gridded Information Required for the ModClark Method (1)

49 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/working-with-gridded-boundary-condition-data/applying-gridded-precipitation-to-a-non-georeferenced-project-structured-discretization

```
Parameter Order: Xcoord Ycoord TravelLength Area
End:

Subbasin: Example
     GridCell: -1024 1000 35.712330137160279 0.12803066423745729
     GridCell: -1023 1000 36.110547371094746 0.36819121314518377
     GridCell: -1024 1001 34.095179546609096 2.3459891648941413
     GridCell: -1023 1001 34.602972599695207 3.6470611489421354
     GridCell: -1022 1001 33.518025304800609 0.089274433594147093
     GridCell: -1025 1002 29.855231616713237 0.064628453062595612
     GridCell: -1024 1002 31.400492125984258 3.6375099423355679
     GridCell: -1023 1002 31.440721043942091 3.9999959114353243
     GridCell: -1022 1002 30.40322501270003 1.0927904847475651
     GridCell: -1025 1003 29.447851314452635 1.6276540712933425
     GridCell: -1024 1003 28.584308324866651 4.0000122954586015
     GridCell: -1023 1003 28.360065563881133 4.0000122954586015
     GridCell: -1022 1003 27.933730473710952 2.9794879903052478
     GridCell: -1021 1003 28.097963233426473 0.44198323704966058
```

20 Gridded Information Required for the ModClark Method (2)

9.7 Snyder Unit Hydrograph Model

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

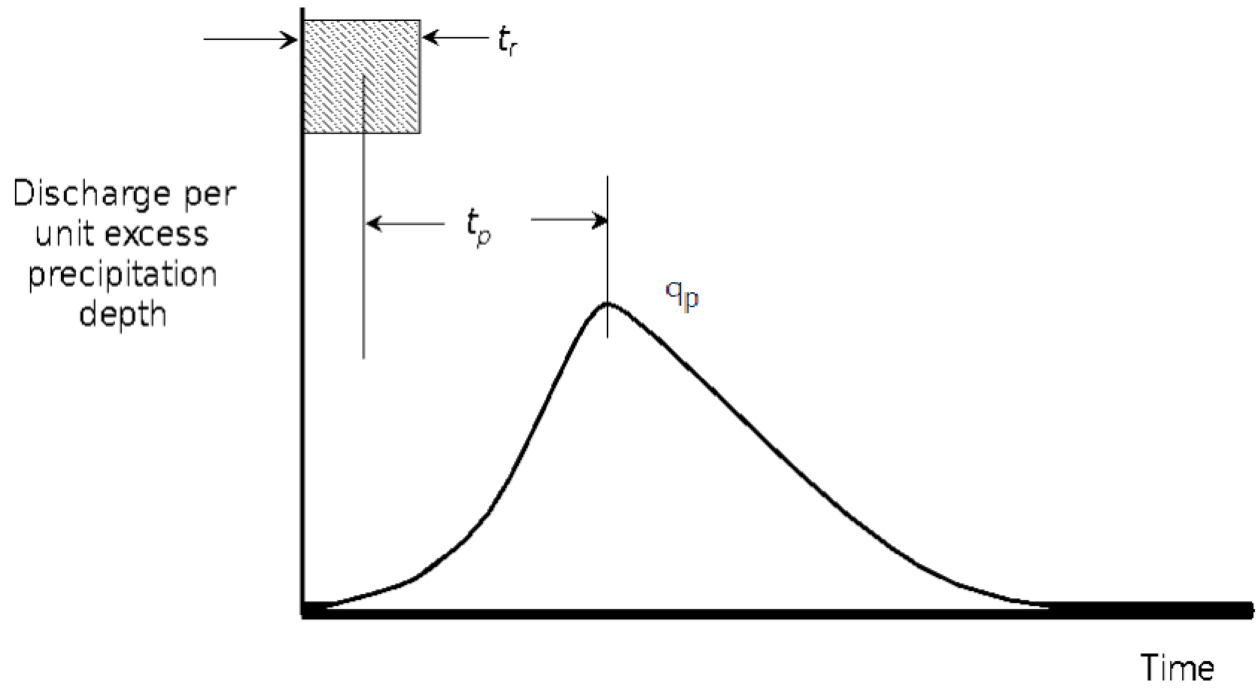
9.7.1 Basic Concepts and Equations

The Snyder unit hydrograph method utilizes multiple relationships to estimate several key ordinates of a unit hydrograph to route excess precipitation to the subbasin outlet. Snyder selected the basin lag, t_p, peak discharge per unit area, q_p, and total time base as the critical characteristics of a unit hydrograph (Snyder, 1938). A "standard unit hydrograph" is defined as one whose rainfall duration, t_r, is related to the basin lag, t_p, by :

48)

$$t_p = 5.5t_r$$

Conceptually, t_p is the difference in time between the centroid of excess precipitation and the time of q_p, as shown below.



Thus, if t_r is specified, t_p can be calculated. If the duration of the desired unit hydrograph for the watershed of interest is significantly different than Equation 1, the following relationship can be used:

49)
$$t_{pR} = t_p - \frac{t_r - t_R}{4}$$

where t_R = lag of the desired unit hydrograph [hours]. For the standard case, Snyder found that q_p can be computed as:

50)
$$q_p = \frac{640 * C_p}{t_p}$$

where C_p = a coefficient derived from gaged data in the same region. For the non-standard case, the peak discharge per unit area of a desired unit hydrograph, q_{pR}, can be computed as:

51)
$$q_{pR} = \frac{640 * C_p}{t_R}$$

In the case of a standard unit hydrograph, Equation 1 and Equation 3 can then be solved to determine t_p and q_p. In the non-standard case, Equation 2 and Equation 4 can be used to determine t_p and q_p. As a final step, a curve with a unit depth of runoff must be fit through the previously computed ordinates. Snyder proposed a relationship with which the total time base of the unit hydrograph could be defined.

Instead of this relationship, within HEC-HMS, an equivalent Clark synthetic unit hydrograph is determined through an optimization routine and utilized in subsequent precipitation-runoff computations.

9.7.2 Required Parameters

Parameters that are required to utilize the “Standard” Snyder method within HEC-HMS include t_p [hours] and C_p. The most typical approach used to estimate the aforementioned parameters is through multi-linear regression analyses using various watershed physical characteristics and combinations. To utilize the “Ft Worth District” method, the total length along the stream centerline to the most hydraulically remote point [mi or km], the length along the stream centerline to the subbasin centroid [mi or km], the weighted slope between points along the stream centerline at 10- and 85-percent of the total length [ft/mi or m/km], percent urbanization, and percent sand must also be specified. To utilize the “Tulsa District” method, the total length along the stream centerline to the most hydraulically remote point [mi or km], the length along the stream centerline to the subbasin centroid [mi or km], the weighted slope between points along the stream centerline at 10- and 85-percent of the total length [ft/mi or m/km], and the percent channelization must also be specified. Typically, these values are estimated using GIS.

9.7.2.1 Estimating Parameters

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

52)
$$t_p = CC_t(LL_c)^{0.3}$$

where Ct = basin coefficient; L = length of the main stream from the outlet to the divide; Lc = 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 Ct of Equation 34 and Cp of Equation 32 are best found via calibration, as they are not physically-based parameters. Bedient and Huber (1992) report that Ct 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 Cp ranges from 0.4 to 0.8, where larger values of Cp are associated with smaller values of Ct.

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

53)
$$t_p = CC_t\left(\frac{LL_c}{\sqrt{S}}\right)^N$$

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 tp as a function of tc , 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. Various studies estimate tp as 50-75% of tc.

9.8 SCS Unit Hydrograph Model

The Soil Conservation Service (SCS) proposed a parametric Unit Hydrograph (UH) model. 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.

9.8.1 Basic Concepts and Equations

The SCS unit hydrograph method makes use of a dimensionless, curvilinear unit hydrograph to route excess precipitation to the subbasin outlet. This dimensionless, curvilinear unit hydrograph expresses discharge, q, as a ratio of the peak discharge, q_p, for any time *t*, as a fraction of the time of rise, T_p. Given the peak discharge and lag time for the duration of excess precipitation, all ordinates of the unit hydrograph can then be estimated. The "standard" SCS curvilinear unit hydrograph contains 37.5-percent of the total runoff before T_p. T_p can be related to the duration excess precipitation as:

54)

$$T_p = \frac{t_r}{2} + t_p$$

in which t_r = duration of excess precipitation (or computational time step) and t_p = the basin lag which is defined as the time difference between the center of mass of excess precipitation and the peak of the unit hydrograph. Furthermore, the peak discharge of the unit hydrograph, Q_p [cubic feet / second] can be related to the watershed area, A [square miles], and T_p [hours] using the following relationship:

55)

$$Q_p = \frac{PRF * A}{T_p}$$

where PRF is a constant which is usually termed the "peak rate factor". Given t_p, Equation (33) can be solved to determine T_p. Then, given a PRF, Equation 2 can be solved to find Q_p. The entire unit hydrograph can then be found from the dimensionless curvilinear form using multiplication.

The standard dimensionless SCS curvilinear unit hydrograph is created by setting the PRF equal to approximately 484. However, the PRF constant has been shown to vary from about 600 in steep terrain to 100 or less in flat areas. Various dimensionless unit hydrographs with predefined peak rate factors are presented in the NRCS National Engineering Handbook (2007). A change in the peak rate factor causes a change in the percent of runoff occurring before T_p, which is typically not uniform across all watersheds because it depends on flow length, ground slope, and other properties of the watershed. By changing PRF, alternate unit hydrographs can be computed for watersheds with varying topography and other conditions that effect runoff.

9.8.2 Required Parameters

Parameters that are required to utilize the SCS method within HEC-HMS include a PRF and a lag time [minutes]. Research has shown that t_p can be related to the watershed time of concentration, T_c, using (Natural Resources Conservation Service, 1999):

56) $$t_p = 0.6 * T_c$$

9.8.2.1 Estimating Parameters

The SCS UH lag can be estimated via calibration for gaged headwater subwatersheds. Time of concentration is a quasi-physically based parameter that can be estimated as:

57) $$t_c = t_{\text{sheet}} + t_{\text{shallow}} + t_{\text{channel}}$$

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:

58) $$V = \frac{CR^{2/3}S^{1/2}}{n}$$

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:

59) $$t_{\text{channel}} = \frac{L}{V}$$

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:

60) $$t_{\text{sheet}} = \frac{0.007(NL)^{0.8}}{(P_2)^{0.5}S^{0.4}}$$

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. The table below 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:

61) $$V = \left\{ \begin{array}{ll} 16.1345\sqrt{S} & \text{for unpaved surface} \\ 20.3282\sqrt{S} & \text{for paved surface} \end{array} \right\}$$

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

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.

9.9 Kinematic Wave Transform Model

The kinematic wave method is 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).

9.9.1 Basic Concepts and Equations

The kinematic wave method utilizes simplifications of the open channel flow equations to route excess precipitation to the subbasin outlet. This approximation starts with a one-dimensional approximation of the momentum equation :

62)
$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t}$$

where S_f = energy gradient (or friction slope) [ft/ft or m/m], S_0 = channel slope [ft/ft or m/m], V = velocity [ft/sec or m/sec], y = hydraulic depth [ft or m], x = distance along the flow path [ft or m], t = time [sec]; g = acceleration due to gravity [ft/sec^2 or m/sec^2], ($\frac{\partial y}{\partial x}$) = pressure gradient [ft/ft or m/m], (V/g)($\frac{\partial V}{\partial x}$) = convective acceleration [ft/sec^2 or m/sec^2], and ($1/g$)($\frac{\partial V}{\partial t}$) = local acceleration [ft/sec^2 or m/sec^2]. S_f can be approximated using Manning's equation:

63)
$$Q = \frac{K}{N} R^{2/3} A \sqrt{S_f}$$

where K = 1.486 or 1.0 when using English units or metric units, respectively, Q = flow [ft^3/sec or m^3/sec], N = overland flow roughness coefficient, R = hydraulic radius [ft], and A = cross-sectional area [ft^2 or m^2]. S_f can be set equal to S_0 when acceleration effects are negligible (i.e. steady, unvaried flow). Thus, Equation (2) can be simplified to:

64)
$$Q = \alpha A^m$$

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

The one-dimensional continuity equation begins as :

65)
$$A\frac{\partial V}{\partial x} + VB\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} = q$$

where B = top width [ft or m], A($\frac{\partial V}{\partial x}$) = prism storage, VB($\frac{\partial y}{\partial x}$) = wedge storage, B($\frac{\partial y}{\partial t}$) = rate of rise, and q = lateral inflow per unit length [ft^3/sec/ft or m^3/sec/m]. Simplifying to shallow flow over a planar surface reduces Equation 4 to:

66)
$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q$$

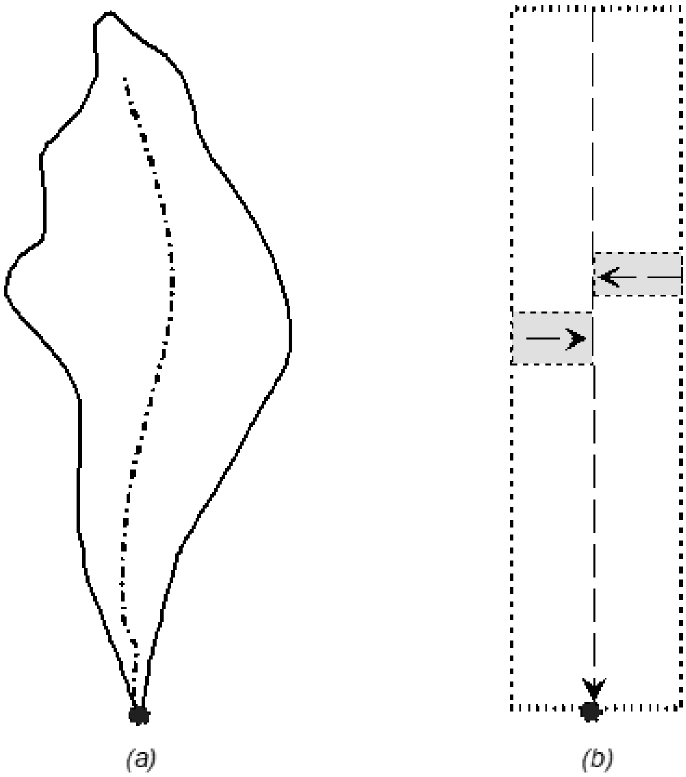
Combining Equation 3 with Equation 5 produces:

67)
$$\frac{\partial A}{\partial t} + \alpha m A^{m-1}\frac{\partial A}{\partial x} = q$$

Equation 6 represents the kinematic wave approximation of the equations of motion. HEC-HMS represents the overland flow component as a wide, rectangular channel of unit width such that *m* = 5/3, and:

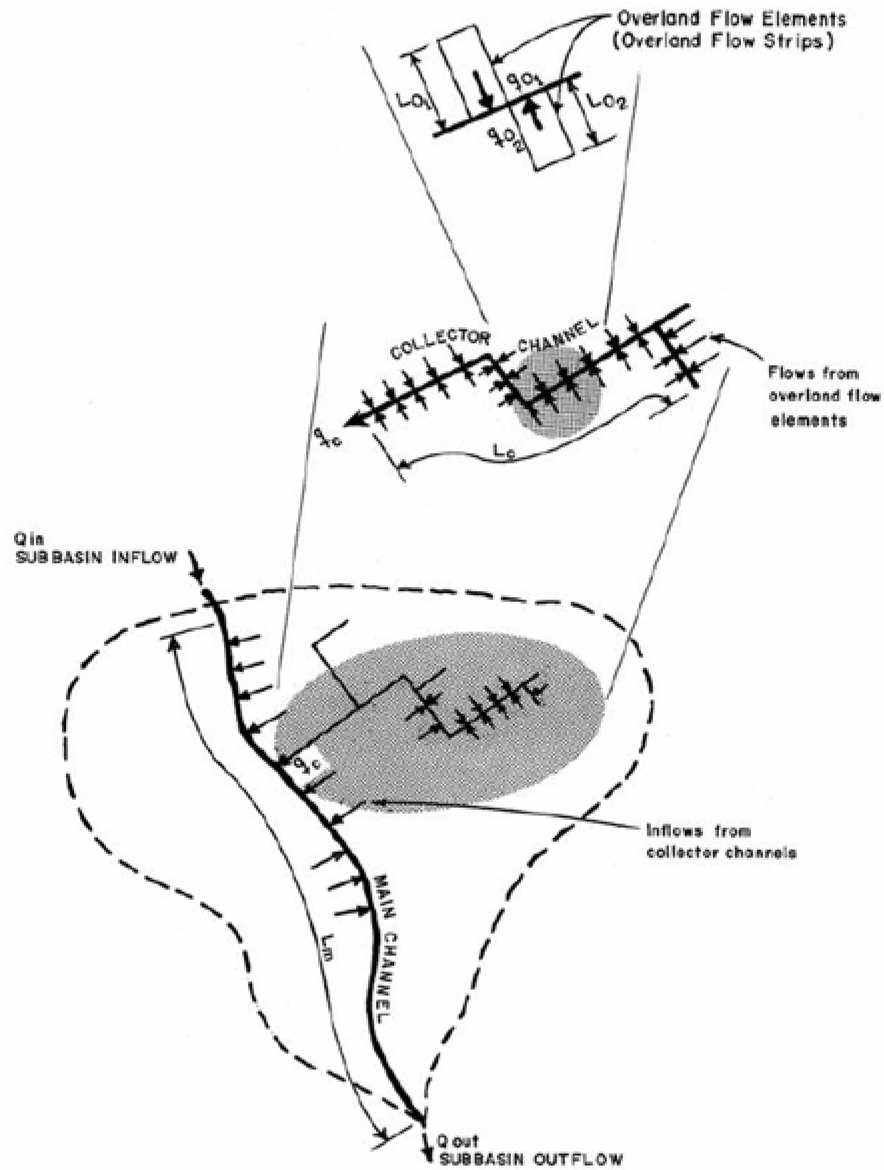
68)
$$\alpha = K * \sqrt{S_f} * \frac{1}{N}$$

The kinematic wave method allows for the representation of variable scales of complexity within a watershed. A simple representation is shown in Figure below where one or two overland planes and a channel are included.



21 Simple Representation of a Watershed Using Kinematic Wave

A complex representation is shown in Figure below where overland planes, subcollectors, collectors, and a channel are included. For a detailed discussion of this method, the reader is directed to HEC (1993).



22 Complex Representation of a Watershed Using Kinematic Wave Reproduced from (Hydrologic Engineering Center, 1993)

9.9.1.1 Channel Flow Model

For certain classes of channel flow, conditions are such that the momentum equation can be simplified. In those cases, the kinematic-wave approximation of Equation 6 is an appropriate model of channel flow. In the case of channel flow, the inflow in Equation 6 may be the runoff from watershed planes or the inflow from upstream channels. Figure below 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.

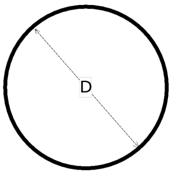
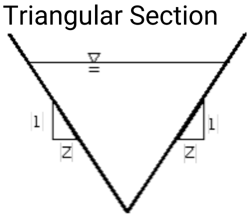
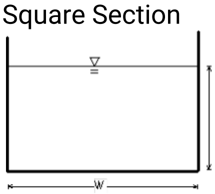
9.9.1.2 Solution of Equations

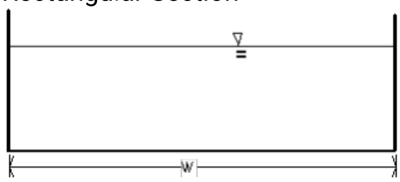
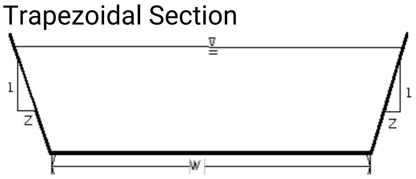
The kinematic-wave approximation is solved in the same manner for either overland or channel flow:

- 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 6, 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.

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

Circular Section 	$\alpha = \frac{0.804}{n} S^{\frac{1}{2}} D^{\frac{1}{6}}$ $m = 5/4$
Triangular Section 	$\alpha = \frac{0.94}{n} S^{\frac{1}{2}} \left(\frac{Z}{1+Z^2}\right)^{\frac{1}{3}}$ $m = 4/3$
Square Section 	$\alpha = \frac{0.72}{n} S^{\frac{1}{2}}$ $m = 4/4$

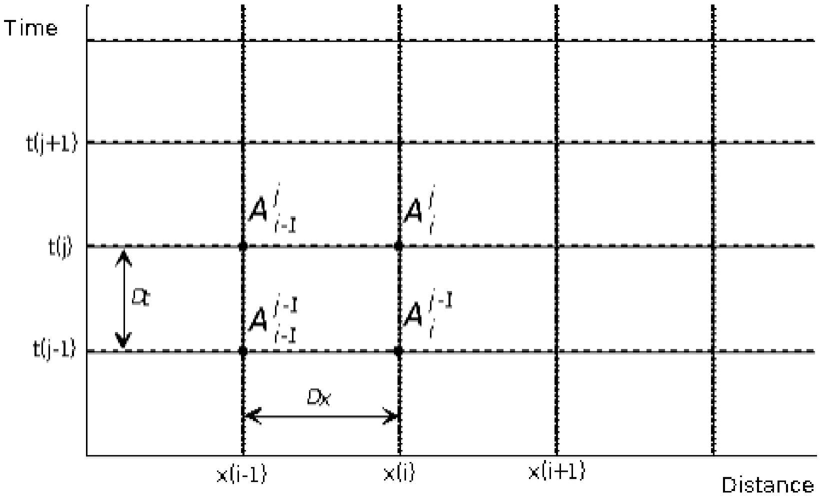
Rectangular Section 	$\alpha = \frac{1.49}{n} S^{\frac{1}{2}} W^{-\frac{2}{3}}$ $m = 5/3$
Trapezoidal Section 	$Q = \frac{1.49}{n} S^{\frac{1}{2}} A^{\frac{5}{3}} \left(\frac{1}{W + 2Y\sqrt{1+Z^2}} \right)^{\frac{2}{3}}$

In Equation 6, 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 :

69)

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

Equation 8 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 below. 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.



With the solution scheme proposed, the only unknown value in Equation 8 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 :

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

The flow is computed as :

71) $$Q_i^j = \alpha\left[A_i^j\right]^m$$

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):

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

or

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

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

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

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

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

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

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

9.9.1.3 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 Δx = $c\Delta t_m$ 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.

9.9.1.4 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 the table below 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 the table below 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 the table below 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 the table below 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 17, for overland flow planes and the tables in Chow (1959) and other texts for channel n values.

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

9.9.2 Required Parameters

Parameters that are required to utilize the kinematic wave method within HEC-HMS include length [ft or m], slope [ft/ft or m/m], an overland flow roughness coefficient, percentage of the total area, and number of routing steps for each overland plane. Optional parameters for subcollectors, collectors, and channel elements may be used as well.

9.10 2D Diffusion Wave Model

9.10.1 Basic Concepts

The 2D Diffusion Wave Transform method explicitly routes excess precipitation throughout a subbasin element using a combination of the continuity and momentum equations. Unlike unit hydrograph transform methods, this transform method can be used to simulate the non-linear movement of water throughout a subbasin when exposed to large amounts of excess precipitation (Minshall, 1960). This Transform Method can be combined with all Canopy, Surface, and Loss methods that are currently within HEC-HMS.

> However, only the None, Linear Reservoir, and Constant Monthly Baseflow Methods can be used with this transform method.

9.10.1.1 2D Mesh

The 2D Diffusion Wave Method represents the subbasin using a 2D mesh which is comprised of both grid cells and cell faces. Grid cells do not have to have a flat bottom and cell faces do not have to be straight lines with a single elevation. Instead, each grid cell and cell face is comprised of hydraulic property tables that are developed using the details of the underlying terrain. This type of model is often referred to as a “high resolution subgrid model” (Casulli, 2008). The term “subgrid” implies the use of a detailed underlying terrain (subgrid) to develop the geometric and hydraulic property tables that represent the grid cells and the cell faces. Currently, users must create a 2D mesh (and any associated connections) within HEC-RAS (version 5.0.7 or newer) and then import to HEC-HMS[50]. In the future, users will be able to create and modify both 2D meshes and boundary conditions entirely within HEC-HMS. The 2D mesh preprocessor within HEC-RAS creates: 1) an elevation-volume relationship for each grid cell and 2) cross sectional information (e.g. elevation-wetted perimeter, area, roughness, etc) for each cell face. The net effects of using a subgrid model such as this are fewer computations, faster run times, greater stability, and improved accuracy. For more information related to the development of a 2D mesh, users are referred to the HEC-RAS 2D Modeling User's Manual[51].

The 2D Diffusion Wave Transform can only be used with Unstructured or File-Specified Discretizations. An Unstructured Discretization can be created by importing a 2D mesh from an HEC-RAS Unsteady Plan HDF file using the File | Import | HEC-RAS HDF File option. Unsteady Plan HDF files have extensions of ".p##.hdf" where "p##" corresponds to the specific plan of interest. When importing a 2D mesh from an HEC-RAS Unsteady Plan HDF file, any accompanying boundary conditions for the selected 2D mesh (except for precipitation time series) will be imported and used to create new 2D Connections with the same parameterization. If a File-Specified Discretization is used, the backing file must be in an HDF 5 format and created using either HEC-RAS or HEC-HMS.

9.10.1.2 2D Engine

HEC’s 2D engine solves the St. Venant Equations using physically measurable characteristics to route water on the overland surface (U.S. Army Corps of Engineers, 2022). This engine makes use of an implicit finite volume algorithm which allows for advantages such as:

- Larger computational time steps than explicit methods,
- Improved stability and robustness over traditional finite difference and finite element techniques,
- Efficient wetting and drying of 2D cells, and
- Subcritical, supercritical, and mixed flow regimes.

Unstructured or structured computational meshes can be utilized within this engine that include triangular, square, rectangular, or even eight-sided elements. Computational cells and cell faces are pre-processed to contain detailed hydraulic property tables including elevation-volume and elevation-conveyance relationships, amongst others. This type of model is often referred to as a "high resolution subgrid model" (Casulli, 2008).

The 2D engine can be used to better recreate anticipated non-linear runoff responses when subjected to large amounts of precipitation when compared to unit hydrograph transform methods (Bartles, 2017). However, 2D overland transform methods require additional data and are more computationally intensive than unit hydrograph transform methods.

50 https://www.hec.usace.army.mil/confluence/display/HMSUM/.Importing+HEC-RAS+HDF+Files+v4.8
51 https://www.hec.usace.army.mil/software/hec-ras/documentation/HEC-RAS%205.0%202D%20Modeling%20Users%20Manual.pdf

For additional details regarding the fundamental equations and solution schemes employed within this transform method please see the HEC-RAS Documentation Page[52].

9.10.2 Required Parameters

Parameters that are required to utilize the 2D Diffusion Wave method within HEC-HMS include implicit weighting factor, water surface tolerance [ft or m], volume tolerance [ft or m], maximum iterations, time step method, use warm up period, and number of cores. If the Adaptive Time Step method is selected, additional parameters are required including the maximum Courant number and maximum time step [seconds]. If the Fixed Time Step method is selected, the maximum time step [sec] must is also required. If the warm up period option is enabled, additional parameters are required including the warm up period [hours] and warm up period fraction.

> A tutorial describing a simple example application of this transform method, including parameter estimation and calibration, can be found here: Creating a Simple 2D Flow Model within HEC-HMS[53].
> A tutorial describing a complex example application of this transform method, including parameter estimation and calibration, can be found here: Creating a Complex 2D Flow model within HEC-HMS[54].

9.11 Applicability and Limitations of Transform Models

The choice of a transform method should be coordinated with the purposes of the study and desired level of accuracy. When hydrologic data are carefully selected and used, the results obtained through the use of unit hydrograph transform methods are generally acceptable for practical purposes. In fact, throughout the world, hydrologic modeling guidelines specify or recommend the use of unit hydrograph theory (Institute of Hydrology, 1999) & (The Institution of Engineers, Australia, 2001).

Initial estimates of transform parameters should be subjected to a model calibration process where computed outputs are compared against observed data and model parameters are modified in order to achieve an adequate fit. Also, best estimate parameters derived through the aforementioned model calibration process should be tested through a model validation process where computed results, without any further parameter modifications, are used to compute outputs which are compared against observed data for independent events that were not considered during model calibration. Storm events used during both model calibration and validation should be approximately equal to the magnitude of events that are being considered within the particular application.

The following table contains a list of various advantages and disadvantages regarding the aforementioned transform methods available for use within HEC-HMS. However, these are only guidelines and should be supplemented by knowledge of, and experience with, the methods and the watershed in question.

52 https://www.hec.usace.army.mil/confluence/rasdocs
53 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/using-2d-flow-within-hec-hms/creating-a-simple-2d-flow-model-within-hec-hms
54 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/using-2d-flow-within-hec-hms/creating-a-complex-2d-flow-model-within-hec-hms

Method	Advantages	Disadvantages
User-Specified Unit Hydrograph	• Well established and documented method.	• Requires the use of observed data to derive an empirical unit hydrograph. • Difficult to calibrate derived unit hydrographs. • Difficult to apply to ungaged areas due to lack of direct physical relationship of parameters and watershed properties. • Shortening the duration of excess precipitation is difficult and can lead to numerical oscillations.
User-Specified S-Graph	• Parameters can be regionalized allowing for estimation within ungaged watersheds.	• Similar to disadvantages of the user-specified unit hydrograph method. • <u>Not</u> well documented nor widely used.
Clark	• "Mature" method that has been used successfully in thousands of studies throughout the U.S. • Well established, widely accepted for use, easy to set up and use. • Parameters can be regionalized and related to measurable basin characteristics. • Parameters <u>can</u> be varied with excess-precipitation rate for use within extreme event simulation when using the Variable Clark option.	• Default time-area histogram may be inappropriate for use within some watersheds (though, a user-specified time-area histogram can be used). • Cannot be used with gridded snowmelt processes. • When using the Variable Clark option, requires development of Variable Tc and R curves.
Modified Clark	• Similar to advantages of the Clark method. • Uses a site-specific time-area histogram. • <u>Can</u> be used with gridded snowmelt processes.	• Similar to disadvantages of the Clark method. • Requires a gridded definition of travel time. • Parameters <u>cannot</u> be varied with excess-precipitation rate for use within extreme event simulation.

Method	Advantages	Disadvantages
Snyder	• Similar to advantages of the Clark method.	• Similar to disadvantages of the Clark method. • Only key unit hydrograph characteristics are defined; HEC-HMS implementation determines equivalent Clark unit hydrograph parameters in order to define a "complete" unit hydrograph. • Parameters <u>cannot</u> be varied with excess-precipitation rate for use within extreme event simulation.
SCS	• Similar to advantages of the Clark method.	• Similar to disadvantages of the Clark method. • Parameters <u>cannot</u> be varied with excess-precipitation rate for use within extreme event simulation. • Only a limited number of unit hydrograph shapes can be derived using this method.
Kinematic Wave	• Predicted values are in accordance with open channel flow theory. • Parameters can be related to measurable basin parameters.	• Method is <u>less</u> parsimonious than unit hydrograph methods; it requires many more parameters. • Not as widely used as unit hydrograph methods. • Only appropriate for use in steep watersheds (slopes > 10 ft/mi) • Relatively small modeling elements are required to adequately determine representative parameters.

2D Diffusion Wave	• Predicted values are in accordance with open channel flow theory.
	• Can be much more accurate than unit hydrograph theory, especially when extrapolating.
	• Allows for:
	    • Unstructured and structured meshes.
	    • Large cell sizes while still retaining detailed terrain information.
		• Much more computationally intense when compared to other transform unit hydrograph theory

9.12 Surface Runoff References

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10 Baseflow

As water is infiltrated to the subsurface, some volume can be lost to deep aquifer storage. However, some volume is only temporarily stored and will return relatively quickly to the surface. The combination of this baseflow and direct runoff results in a total runoff hydrograph.

10.1 Available Baseflow Methods

While a subbasin element conceptually represents infiltration, surface runoff, and subsurface processes interacting together, the actual subsurface calculations are performed by a baseflow method contained within the subbasin. A total of five different baseflow methods are provided. Some of the methods are designed primarily for simulating events while others are intended for continuous simulation. These methods include:

- Constant Monthly[55]
- Recession[56]
- Bounded Recession[57]
- Linear Reservoir[58]
- Nonlinear Boussinesq[59]

10.2 Constant Monthly Model

10.2.1 Basic Concepts and Equations

The Constant Monthly baseflow method allows the specification of a constant baseflow for each month of the year.

Unless parameters are carefully chosen, this method is not guaranteed to conserve mass (e.g., precipitation losses < baseflow volume).

This method is primarily intended for continuous simulation in subbasins where the baseflow is easily approximated by a constant flow for each month.

55 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/baseflow/constant-monthly-model
56 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/baseflow/recession-model
57 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/baseflow/bounded-recession-model
58 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/baseflow/linear-reservoir-model
59 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/baseflow/nonlinear-boussinesq-model

10.2.2 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the rate of baseflow for all twelve months throughout the year [ft^3/sec or m^3/sec]. These constant monthly baseflow rates are best estimated empirically using 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, baseflow contributions may be negligible.

A tutorial describing an example application of this baseflow method, including parameter estimation and calibration, can be found here: Applying the Constant Monthly Baseflow Method[60].

10.3 Recession Model

10.3.1 Basic Concepts and Equations

The Recession baseflow method is designed to approximate the typical behavior observed in watersheds when baseflow recedes exponentially after an event (Chow, Maidment, & Mays, 1988). This method is intended primarily for event simulation. However, it does have the ability to automatically reset after each storm event and consequently may be used for continuous simulation.

Unless parameters are carefully chosen, this method is not guaranteed to conserve mass (e.g., precipitation losses < baseflow volume).

This method defines the baseflow at time *t*, Q_t [ft^3/sec or m^3/sec], as:

77)
$$Q_t = Q_o * k^t$$

where Q_0 = initial baseflow at time zero [ft^3/sec or m^3/sec] and k = recession constant. Within HEC-HMS, k is defined as the ratio of the baseflow at time t to the baseflow one day earlier and must be positive and less than one.

Required Parameters

60 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-baseflow-methods-in-hec-hms/applying-the-constant-monthly-baseflow-method

Parameters that are required to utilize this method within HEC-HMS include the initial baseflow type and value, recession constant, and threshold type and value. The initial discharge type can be specified as either a discharge rate [ft^3/sec or m^3/sec] or a discharge rate per area [ft^3/sec/mi^2 or m^3/sec/km^2]. The discharge rate method is most appropriate when there is observed streamflow data at the outlet of the subbasin for determining the initial flow in the channel. The discharge rate per area method is better suited when regional information is available. The threshold type can be specified as either a ratio to peak or a threshold discharge [ft^3/sec or m^3/sec]. If the threshold type is set to ratio to peak, the baseflow will be reset when the current flow divided by the peak flow falls to the specified value. If the threshold type is set to a threshold discharge, the baseflow will be reset when the receding limb of the hydrograph falls to the specified value, regardless of the peak flow during the previous storm event.

A tutorial describing an example application of this baseflow method, including parameter estimation and calibration, can be found here: Applying the Recession Baseflow Method[61].

The recession constant, k, depends upon the source of baseflow. If k = 1.0, the baseflow contribution will be constant, with $Q_t = Q_0$. Otherwise, to model the exponential decay typical of natural undeveloped watersheds, k must be less than 1.0. The following table shows typical values proposed by Pilgrim and Cordery (1992) for basins ranging in size from 120 to 6500 square miles (300 to 16,000 square kilometers) in the U.S., eastern Australia, and several other regions.

Flow Component	**Recession Constant, *k***
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 ratio to peak or discharge value can be estimated from examination of a hydrograph of observed flows. The flow at which the recession limb is approximated well by a straight line defines the threshold ratio to peak or discharge value.

61 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-baseflow-methods-in-hec-hms/applying-the-recession-baseflow-method

10.4 Bounded Recession Model

10.4.1 Basic Concepts and Equations

The Bounded Recession baseflow method is very similar to the Recession method. The principal difference is that monthly baseflow limits can be used to limit the baseflow magnitude. In effect, baseflow is first computed according to the previously described recession methodology and then monthly limits are imposed. Though there are many similarities with the recession method, one important difference is that this method does not reset the baseflow after a storm event.

> Unless parameters are carefully chosen, this method is not guaranteed to conserve mass (e.g., precipitation losses < baseflow volume).

Required Parameters

Parameters that are required to utilize this method within HEC-HMS include those which were previously mentioned for the recession method in addition to limiting baseflow values for all twelve months of the year.

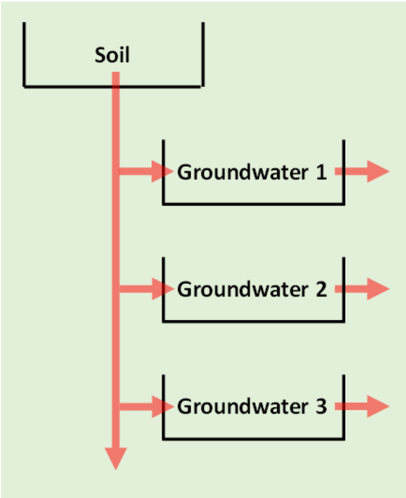
10.5 Linear Reservoir Model

10.5.1 Basic Concepts and Equations

As implied by its name, the Linear Reservoir baseflow method uses one to three linear reservoirs (layers) to simulate the recession of baseflow after a storm event.

> Unlike the other baseflow methods contained within HEC-HMS, this method is guaranteed to conserve mass (i.e., the baseflow volume cannot exceed precipitation losses).

The volume of infiltrated water is used as inflow to the Linear Reservoir method. Inflow can be partitioned to each layer in addition to deep aquifer recharge. As such, during periods of high infiltration, more baseflow will be generated. Conversely, during periods of little to no infiltration, less baseflow will be generated. When three groundwater reservoirs are used within this method, the system can be conceptualized as shown in the following figure.



23 Conceptual Representation of the Linear Reservoir Method

As described within the Soil Moisture Accounting Loss Method (SMA) section[62], when the Linear Reservoir method is used in conjunction with the SMA infiltration method, special behavior is produced. The lateral outflow from the SMA groundwater layer 1 is connected as inflow to the Linear Reservoir groundwater layer 1. The lateral outflow from the SMA groundwater layer 2 is connected as inflow to the Linear Reservoir groundwater layer 2. The percolation out of the SMA groundwater layer 2 is connected as inflow to the Linear Reservoir groundwater layer 3. Partition fractions are not used for the Linear Reservoir groundwater 1 and 2 layers because their inflow is determined by the respective lateral outflow from the SMA groundwater layers. However, a partition fraction should be used with the Linear Reservoir groundwater layer 3 in order to define how much percolation from SMA groundwater layer 2 is lost to deep aquifer recharge how much goes towards inflow to the Linear Reservoir groundwater layer 3.

The Linear Reservoir Baseflow method can be used with any loss method; it **DOES NOT** require the use of the SMA loss method.

Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the number of reservoirs/layers, the initial baseflow type and value, the partition fraction, the routing coefficient [hours], and the number of routing steps for each layer. The initial discharge type can be specified as either a discharge rate [ft^3/sec or m^3/sec] or a discharge rate per area [ft^3/sec/mi^2 or m^3/sec/km^2]. Using the discharge rate method is appropriate when there is observed streamflow data at the outlet of the subbasin for determining the initial flow in the channel. The discharge rate per area method is better suited when regional information is available. However, the same method must be used for specifying the initial condition for all layers. The partition fraction is used to determine the amount of inflow going to each layer. Each fraction must be greater than 0.0 and the sum of the fractions must be less than or equal to 1.0. If the sum of the fractions is less than 1.0, the remaining volume will be removed from the system (i.e. deep aquifer recharge). If the sum of the fractions is exactly equal to 1.0, then all percolation will become baseflow and there will be no deep aquifer recharge. The routing coefficient is the time constant for each layer. Similar to the estimation of parameters for the Clark unit hydrograph transform, this parameter can be estimated using measurable

62 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/infiltration-and-runoff-volume/soil-moisture-accounting-loss-model

watershed characteristics. The number of routing steps can be used to subdivide the routing through each layer and is related to the amount of attenuation during the routing. Minimum attenuation is achieved when only one routing step is selected. Attenuation of baseflow increases as the number of steps increases.

A tutorial describing an example application of this baseflow method, including parameter estimation and calibration, can be found here: Applying the Linear Reservoir Baseflow Method[63].

10.6 Nonlinear Boussinesq Model

10.6.1 Basic Concepts and Equations

The Nonlinear Boussinesq baseflow method is similar to the Recession baseflow method but assumes that the channel overlies an unconfined aquifer which is itself underlain by a horizontal impermeable layer (Szilagyi & Parlange, 1998). Through the use of the one-dimensional Boussinesq equation, an assumption that capillarity above the water table can be neglected, and the Dupuit-Forcheimer approximation, it is possible to parameterize the method using measurable field data. This method is intended primarily for event simulation. However, it does have the ability to automatically reset after each storm event and consequently may be used for continuous simulation.

Unless parameters are carefully chosen, this method is not guaranteed to conserve mass (e.g., precipitation losses < baseflow volume).

Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the initial baseflow type and value, threshold type and value, the characteristic subsurface flow length [ft or m], saturated hydraulic conductivity of the aquifer [in/hr or mm/hr], and drainable porosity of the aquifer [ft/ft or m/m].

The initial discharge type can be specified as either a discharge rate [ft^3/sec or m^3/sec] or a discharge rate per area [ft^3/sec/mi^2 or m^3/sec/km^2]. The discharge rate method is most appropriate when there is observed streamflow data at the outlet of the subbasin for determining the initial flow in the channel. The discharge rate per area method is better suited when regional information is available. The threshold type can be specified as either a ratio to peak or a threshold discharge [ft^3/sec or m^3/sec]. If the threshold type is set to ratio to peak, the baseflow will be reset when the current flow divided by the peak flow falls to the specified value. If the threshold type is set to a threshold discharge, the baseflow will be reset when the receding limb of the hydrograph falls to the specified value, regardless of the peak flow during the previous storm event. The characteristic subsurface flow length corresponds to the mean distance from the subbasin boundary to the stream, which can be estimated using GIS. The saturated hydraulic conductivity of the

63 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-baseflow-methods-in-hec-hms/applying-the-linear-reservoir-baseflow-method

aquifer can be estimated from field tests or from the predominant soil texture. An upper limit of the drainable porosity of the aquifer corresponds to the total porosity minus the residual porosity. The actual drainable porosity depends on site-specific conditions.

10.7 Applicability and Limitations of Baseflow Models

The choice of a baseflow method hinges upon the purposes of the study and desired level of detail. Typically, the amount of baseflow generated during extreme events (like those simulated within dam safety studies) pales in comparison to the amount of direct runoff. However, in locations with voluminous underground flow paths, direct runoff may not be the predominant source of streamflow. Also, studies that require the simulation of short-duration events (e.g., days to weeks) tend to focus more upon direct runoff generation while studies that require the simulation of long-duration events (e.g., months to years) use continuous simulation techniques that focus on both direct runoff and baseflow generation.

Initial estimates of baseflow parameters should be subjected to a model calibration process where computed outputs are compared against observed data and model parameters are modified in order to achieve an adequate fit. Also, best estimate parameters derived through the aforementioned model calibration process should be tested through a model validation process where computed results, without any further parameter modifications, are used to compute outputs which are compared against observed data for independent events that were not considered during model calibration. Storm events used during both model calibration and validation should be approximately equal to the magnitude of events that are being considered within the particular application.

The following table contains a list of various advantages and disadvantages regarding the aforementioned baseflow methods available for use within HEC-HMS. However, these are only guidelines and should be supplemented by knowledge of, and experience with, the methods and the watershed in question.

Method	Advantages	Disadvantages
Constant Monthly	• Simple, parsimonious method.	• Method may be too simple to predict baseflow especially during large storm events. • Does not automatically conserve mass. • No connection to infiltrated water.
Recession	• "Mature" method that has been used successfully in thousands of studies throughout the U.S. • Parameters can be regionalized. • Baseflow can be reset after a storm event.	• Does not automatically conserve mass. • No connection to infiltrated water.

Method	Advantages	Disadvantages
Bounded Recession	• Similar to advantages of the Recession method.	• Similar to disadvantages of the Recession method. • Baseflow cannot be reset after a storm event. • Not well documented nor widely used.
Linear Reservoir	• This method automatically conserves mass. • Method is scalable; can be used for single event or continuous simulation. • Infiltrated water can be partitioned to volume that returns to the stream and volume that is lost to deep aquifer storage. • Baseflow generation corresponds to infiltration (more infiltration = more baseflow; less infiltration = less baseflow). • Parameters can be regionalized and estimated using watershed characteristics.	• Requires more parameters than the Recession method. • Less parsimonious than simpler, empirical methods.
Nonlinear Boussinesq	• Similar to advantages of the Recession method. • Parameters can be estimated using watershed characteristics.	• Similar to disadvantages of the Recession method. • Requires more parameters than the Recession method. • Not well documented nor widely used.

10.8 Baseflow References

Chow, V.T., Maidment, D.R., and Mays, L.W. (1988). Applied hydrology. McGraw-Hill, New York, NY.

Linsley, R.K., Kohler, M.A., and Paulhus, J.L.H. (1982). Hydrology for engineers. McGraw-Hill, New York, NY.

Pilgrim, D.H, and Cordery, I. (1992). "Flood runoff." D.R. Maidment, ed., Handbook of hydrology, McGraw-Hill, New York, NY.

Szilagyi, J., & Parlange, M. B. (1998). Baseflow Separation Based on Analytical Solutions of the Boussinesq Equation. *Journal of Hydrology*, 251 - 260.

11 Channel Flow

As the total runoff from subbasins reaches defined channels, the depth of water increases and the predominant flow regime begins to transition to open channel flow. At this point, open channel flow approximations are used to represent translation and attenuation effects as flood waves move downgradient. This section describes the models of channel flow that are included in the program; these are also known as routing models. 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.

11.1 Channel Flow Basic Concepts, Equations, and Solution Techniques

11.1.1 Fundamental 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:

78)

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

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:

79)

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

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.

11.1.2 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 2) 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. The following table 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:

80) $$S_f = S_0$$

If this simplified momentum equation is combined with the continuity equation, the result is the kinematic wave approximation, which is described here: Kinematic Wave Channel Routing Model[64].

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, which is described here: Muskingum-Cunge Model (see page 210).

64 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/kinematic-wave-channel-routing-model

81)
$$S_f = S_0 - \frac{\partial y}{\partial x}$$

- 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 HEC-RAS (USACE, 2023).

82)
$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x}$$

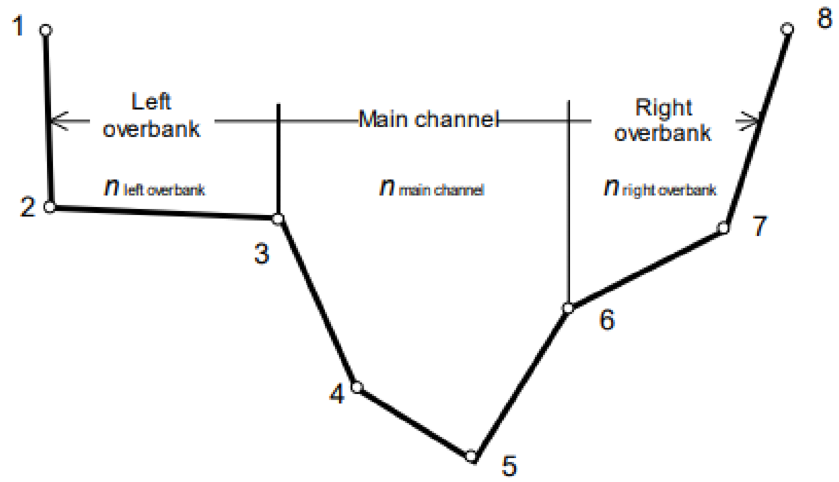
11.1.3 Solution Schemes

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.

11.1.4 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. The 8-point cross-section configuration is one of the cross section shapes available to describe the channel. The 8 pairs of x, y (distance, elevation) values are described spatially in the figure below. Coordinates 3 and 6 represent the left and right banks of the channel, respectively. Coordinates 4 and 5 are located within the channel. Coordinates 1 and 2 represent the left overbank and coordinates 7 and 8 represent the right overbank.



- 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 initial inflow, the baseflow within the channel at the start of the simulation, or the 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.

11.2 Available Channel Routing Methods

A total of eight different channel routing methods are available for use within HEC-HMS. These methods include:

- Kinematic Wave[65]
- Lag[66]
- Lag and K[67]
- Modified Puls[68]
- Muskingum[69]
- Muskingum-Cunge[70]
- Normal Depth[71]

65 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/kinematic-wave-channel-routing-model
66 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/lag-model
67 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/lag-and-k-model
68 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/modified-puls-model
69 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/muskingum-model
70 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/muskingum-cunge-model
71 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/normal-depth-model

- Straddle Stagger[72]

The following sections detail their unique concepts and uses.

11.3 Kinematic Wave Channel Routing Model

11.3.1 Basic Concepts and Equations

The derivation of the kinematic wave routing method is detailed within the Kinematic Wave Transform Model[73] section. In summary, this method approximates the full unsteady flow equations by neglecting inertial and pressure forces. Specifically, the pressure gradient, convective acceleration, and local acceleration terms within the momentum equation are ignored. This results in the following simplification:

83)
$$S_f = S_0$$

As shown within the previous equation, the energy gradient is assumed to be equal to the bottom slope.

> As such, this method is only appropriate for use in steep channels (i.e. 10 ft/mi or greater) and does not recreate backwater effects.

11.3.2 Required Parameters

The parameters that are required to utilize this method within HEC-HMS are the initial condition, the reach length [ft or m], the bottom slope [ft/ft or m/m], Manning's n roughness coefficient, the number of subreaches, an index method and value, and a cross-section shape and parameters/dimensions. An optional invert can also be specified.

Two options for specifying the initial condition are included: outflow equals inflow and specified discharge [ft^3/sec or m^3/sec]. The first option assumes that the initial outflow is the same as the initial inflow to the reach from the upstream elements which is equivalent to the assumption of a steady-state initial condition. The second option is most appropriate when there is observed streamflow data at the end of the reach.

The reach length should be set as the total length of the reach element while the bed slope should be set as the average bed slope for the entire reach. If the slope varies significantly throughout the stream represented by the reach, it may be necessary to use multiple reaches with different slopes. The Manning's n roughness coefficient should be set as the average value for the whole reach. This value can be estimated using "reference" streams with established roughness coefficients or through calibration.

The number of subreaches is used in concert with the index method to determine the minimum distance step to use during routing calculations. Two options for specifying an index method are included: flow [ft^3/s or m^3/s] and celerity [ft/s or m/s]. When index flow is selected, the user-entered flow rate is converted to an equivalent celerity using the cross-section shape of the reach. When the index celerity method is selected, the travel time is computed directly from the specified value. The distance step is first estimated from the

72 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/straddle-stagger-model
73 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/transform/kinematic-wave-transform-model

travel time. If the distance step is greater than the reach length divided by the number of subreaches, then the distance step is decreased.

Five options are provided for specifying the cross-section shape: circle, deep, rectangle, trapezoid, and triangle. The circle shape is not meant to be used for pressure flow or pipe networks, but is suitable for representing a free surface inside a pipe. The deep shape should only be used for flow conditions where the flow depth is approximately equal to the flow width. Depending on the shape you choose, additional information will have to be entered to describe the size of the cross-section shape. This information may include a diameter (circle) [ft or m], bottom width (deep, rectangle, and/or trapezoid) [ft or m], or side slope (trapezoid and triangle) [ft/ft or m/m]. In all cases, cross-section shapes must be defined in such a way that all possible flow depths that will be simulated will be completely confined within the defined shape.

Many of the aforementioned parameters are typically estimated using GIS. However, field survey data may be necessary to accurately determine reach lengths, bed slopes, and/or cross-section shape parameters.

11.4 Lag Model

11.4.1 Basic Concepts and Equations

This is the simplest of the routing models in HEC-HMS. Using the Lag model, 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 method does not include any representation of attenuation or diffusion processes. Consequently, it is best suited to short stream segments with a predicable travel time that doesn't vary with changing conditions.

Mathematically, the downstream ordinates are computed by:

84)

$$O_t = \left\{ \begin{array}{ll} I_t & t < \text{lag} \\ I_{t+\,lag} & t \geq \text{lag} \end{array} \right\}$$

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.

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 = Δt in the Muskingum model, the computed outflow hydrograph will equal the inflow hydrograph lagged by K.

11.4.2 Required Parameters

The parameters that are required to utilize this method within HEC-HMS are the initial condition and a lag time [minutes]. Two options for specifying the initial condition are included: outflow equals inflow and specified discharge [ft^3/sec or m^3/sec]. The first option assumes that the initial outflow is the same as the initial inflow to the reach from the upstream elements which is equivalent to the assumption of a steady-state initial condition. The second option is most appropriate when there is observed streamflow data at the end of the reach. Lag time is the amount of time that the inflow hydrograph will be translated.

A tutorial describing an example application of this channel routing method, including parameter estimation and calibration, can be found here: Applying the Lag Routing Method[74].

11.5 Lag and K Model

11.5.1 Basic Concepts and Equations

The Lag and K routing method is a hydrologic storage routing method based on a graphical routing technique that is extensively used by the National Weather Service (NWS) (Linsley, Kohler, & Paulhus, 1982). Within this method, lag and K represent translation and attenuation, respectively. The method is a special case of the Muskingum[75] method where channel storage is represented by the prism component alone with no wedge storage (i.e. Muskingum X = 0). The following equation is combined with inflow vs. translation and outflow vs. attenuation functions in order to solve for outflow:

85)
$$\frac{dS}{dt} = I_t - O_t$$

where *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*.

The lack of wedge storage means that the method should only be used for slowly varying flood waves. Also, this method does not account for complex flow conditions such as backwater effects and/or hydraulic structures.

74 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-reach-routing-methods-within-hec-hms/applying-the-lag-routing-method
75 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/channel-flow/muskingum-model

11.5.2 Required Parameters

The parameters that are required to utilize this method within HEC-HMS are the initial condition, a lag method and value or function, and a K method and value or function. Two options for specifying the initial condition are included: outflow equals inflow and specified discharge [ft^3/sec or m^3/sec]. The first option assumes that the initial outflow is the same as the initial inflow to the reach from the upstream elements which is equivalent to the assumption of a steady-state initial condition. The second option is most appropriate when there is observed streamflow data at the end of the reach. Two options for specifying a lag method are included: Constant Lag [hours] and Variable Lag. When using the Variable Lag option, an Inflow-Lag function (which is a paired data object) must be specified. Similarly, two options for specifying a K method are included: Constant K [hours] and Variable K. When using the Variable K option, an Outflow-Attenuation function (which is a paired data object) must be specified. These relationships are typically derived through evaluation of historical flood hydrographs. Care must be exercised when using lag functions with multiple intercepts (i.e., lag is the same for more than one flow rate) as this may result in numerically attenuated peak flow rates.

11.6 Modified Puls Model

11.6.1 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

86) $$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0$$

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:

87) $$\overline{I_t} - \overline{O_t} = \frac{\Delta S_t}{\Delta t}$$

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:

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

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 3. Once that function is established, it is substituted into Equation 3, 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.

> If the storage vs. discharge relationships are carefully constructed using a hydraulic model that includes bridges and/or other hydraulic structures, this method can simulate backwater effects and the impacts of hydraulic structures so long as the effects/impacts are fully contained within the reach.

11.6.2 Required Parameters

The parameters that are required to utilize this method within HEC-HMS are the initial condition, a storage vs. discharge relationship, and the number of subreaches. An optional elevation vs. discharge function can be selected in addition to an optional invert.

Two options for specifying the initial condition are included: outflow equals inflow and specified discharge [ft^3/sec or m^3/sec]. The first option assumes that the initial outflow is the same as the initial inflow to the reach from the upstream elements which is equivalent to the assumption of a steady-state initial condition. The second option is most appropriate when there is observed streamflow data at the end of the reach. In either case, the initial storage will be computed from the first inflow to the reach and corresponding storage vs. discharge function.

The storage vs. discharge relationship (which is a paired data object) must define the amount of outflow for a specific amount of storage in the reach. Entered storage values should cover the entire range of expected storages that will be encountered during a simulation; the first storage value should be set to zero. Typically, hydraulic routing simulations that include bridges and/or hydraulic structures are used to generate the storage vs. discharge relationships (Hydrologic Engineering Center, 2023).

The specified number of subreaches affects attenuation. One subreach results in the maximum amount of attenuation and increasing the number of subreaches approaches zero attenuation. For an idealized channel, the travel time through a subreach should be approximately equal to the simulation time step. An initial estimate of this parameter can be obtained by dividing the actual reach length by the product of the wave celerity and the simulation time step. For natural channels that vary in cross-section dimension, slope, and storage, the number of subreaches can be treated as a calibration parameter. The number of subreaches may be used to introduce numerical attenuation which can be used to better represent the movement of flood waves through the natural system.

The optional elevation vs. discharge function should represent the depth of water for any given outflow from the reach. When an elevation vs. discharge function is specified, the optional invert elevation should also be specified.

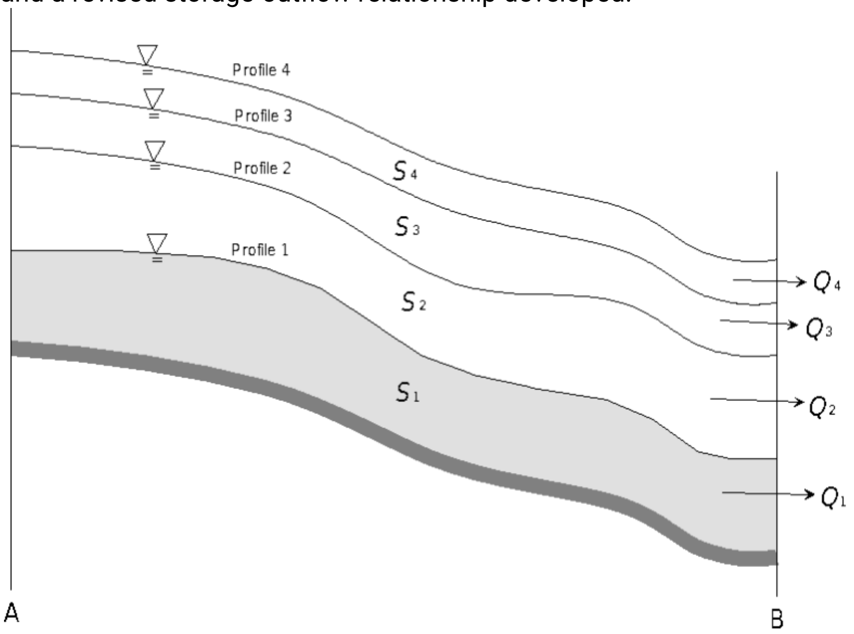
11.6.2.1 Defining the Storage-Outflow Relationship

The storage-outflow relationship required for the Modified Puls routing model can be determined using several techniques.

11.6.2.1.1 Using Hydraulic Model Outputs

Water-surface profiles can be computed with a hydraulic model for a range of discharges. Hydraulic modeling applications like HEC-RAS (USACE, 2023) include automated tools that can be used to define a relationship of storage to flow between two channel cross sections using computed water-surface profiles.

The following figure illustrates 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.



11.6.2.1.2 Using Historical Observations

Storage-outflow relationships can also be determined using 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 values may be better suited to calibrate a water-surface profile-model for the channel reach of interest. Then the calibrated model can be used to establish the storage-outflow relationship as described above.

11.6.2.1.3 Trial and Error

Finally, storage-outflow relationships can be calibrated 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 3. 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 computed outflow hydrograph is compared with the observed hydrograph. If the match is not adequate, the function is adjusted, and the process is repeated.

11.6.2.2 Estimating Other Model Parameters

The Kinematic Wave Transform Model[76] section 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 also applies 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:

89)
$$\text{steps} = \frac{L}{c\Delta t}$$

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:

90)
$$\text{steps} = 2L\frac{S_0}{y_0}$$

where y_o = normal depth associated with baseflow in the channel. Engineer Manual 1110-2-1417 Flood-Runoff Analysis (U.S. Army Corps of Engineers, 1994) indicates that this parameter, however, is best determined by calibration, using observed inflow and outflow hydrographs.

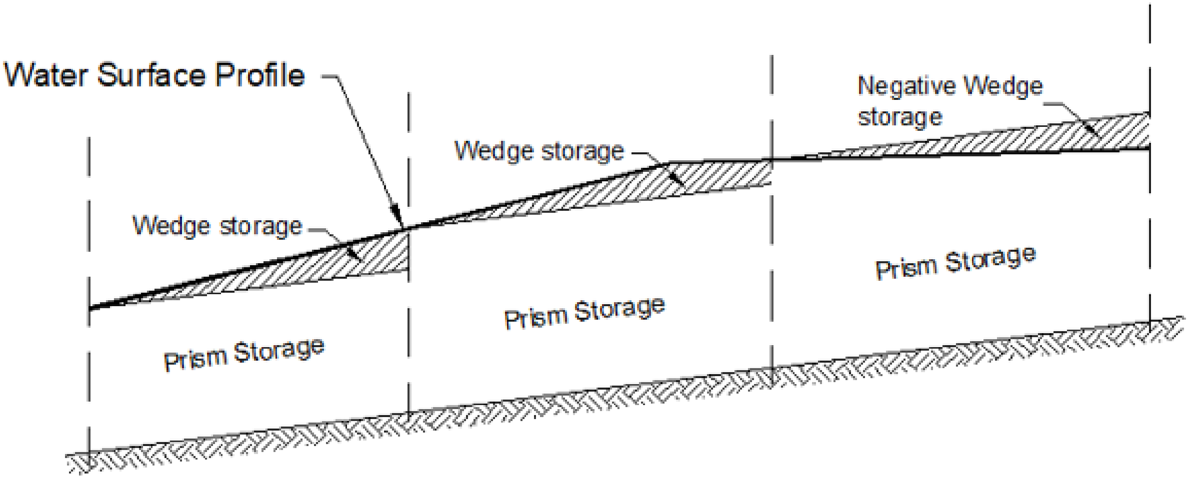
11.7 Muskingum Model

11.7.1 Basic Concepts and Equations

The Muskingum routing method is similar to the Modified Puls method in that a conservation of mass approach is used to route flow through the stream reach. However, the Muskingum method accounts for

76 https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/transform/kinematic-wave-transform-model

"looped" storage vs. outflow relationships that commonly exist in most rivers (i.e., hysteresis). This can simulate the commonly observed increased channel storage during the rising side and decreased channel storage during the falling side of a passing flood wave. To do so, the total storage in a reach is conceptualized as the sum of prism (i.e., rectangle) and wedge (i.e., triangle) storage, as shown in the following figure. During rising stages on the leading edge of a flood wave, wedge storage is positive and added to the prism storage. Conversely, during falling stages on the receding side of a flood wave, wedge storage is negative and subtracted from the prism storage. Through the inclusion of a travel time for the reach and a weighting between the influence of inflow and outflow, it is possible to approximate attenuation.



This method begins with the following form of the continuity equation:

91) $$\overline{I_t} - \overline{O_t} = \frac{\Delta S_t}{\Delta t}$$

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 finite difference approximation and rearrangement yields:

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

where I_{t-1} and I_t = inflow to the reach at times t-1 and t, respectively, O_{t-1} and O_t = outflow from the reach at times t-1 and t, respectively, and S_{t-1} and S_t = storage within the reach at times t-1 and t, respectively.

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 method defines total storage as:

93) $$S_t = KO_t + KX(I_t - O_t)$$

Further simplification yields:

94) $$S_t = KO_t + KX(I_t - O_t) = K[XI_t + (1 - X)O_t]$$

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

The quantity $X\left(I_t - O_t\right) = K\left[XI_t + (1-X)O_t\right]$ 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 4 resolves to a linear reservoir model:

95)
$$S_t = KO$$

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 2 is substituted into Equation 4 and the result is rearranged to isolate the unknown values at time t, the result is:

96)
$$O_t = \left(\frac{\Delta t - 2KX}{2K(1-X)+\Delta t}\right)I_t + \left(\frac{\Delta t + 2KX}{2K(1-X)+\Delta t}\right)I_{t-1} + \left(\frac{2K(1-X)-\Delta t}{2K(1-X)+\Delta t}\right)O_{t-1}$$

HEC-HMS solves Equation 6 recursively to compute O_t given inflow (I_t and I_{t-1}), an initial condition ($O_{t=0}$), K, and X.

11.7.2 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the initial condition, K [hours], X, and the number of subreaches. Two options for specifying the initial condition are included: outflow equals inflow and specified discharge [ft^3/sec or m^3/sec]. The first option assumes that the initial outflow is the same as the initial inflow to the reach from the upstream elements which is equivalent to the assumption of a steady-state initial condition. The second option is most appropriate when there is observed streamflow data at the end of the reach. In either case, the initial storage will be computed from the first inflow to the reach and corresponding storage vs. discharge function.

K is equivalent to the travel time through the reach. Initial estimates of this parameter can be made using observed streamflow data or through approximations of flood wave celerity. One such approximation is Seddon's Law (Ponce, 1983):

97)
$$c = \frac{1}{B}\frac{dQ}{dy}$$

where c = flood wave celerity [ft/s or m/s], B = top width of the water surface [ft or m], and dQ/dy = slope of the discharge vs. stage relationship (i.e. rating curve). K can then be estimated using:

98)
$$K = \frac{L}{c}$$

where L = length of the reach [ft or m].

X is a dimensionless coefficient that lacks a strong physical meaning. This parameter must range between 0.0 (maximum attenuation) and 0.5 (no attenuation). For most stream reaches, an intermediate value is found through calibration.

The specified number of subreaches affects attenuation. One subreach results in the maximum amount of attenuation and increasing the number of subreaches approaches zero attenuation. For an idealized channel, the travel time through a subreach should be approximately equal to the simulation time step. An initial estimate of this parameter can be obtained by dividing the actual reach length by the product of the wave celerity and the simulation time step. For natural channels that vary in cross-section dimension, slope, and storage, the number of subreaches can be treated as a calibration parameter. The number of subreaches may be used to introduce numerical attenuation which can be used to better represent the movement of flood waves through the natural system.

A tutorial describing an example application of this loss method, including parameter estimation and calibration, can be found here: Applying the Muskingum Routing Method[77].

11.8 Muskingum-Cunge Model

11.8.1 Basic Concepts and Equations

The Muskingum-Cunge routing method builds upon concepts within the Muskingum method which was previously presented. This method uses a combination of the continuity equation and a simplified form of the momentum equation. The Muskingum-Cunge method is sometimes referred to as a “variable coefficient” method since the routing parameters are recalculated every time step based upon channel properties and the flow depth.

This method begins by neglecting the convective and local acceleration terms within the momentum equation which yields:

99)
$$S_f = S_o - \frac{\partial y}{\partial x}$$

Equation 1 is then combined with the following equation:

100)
$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_L$$

After combining Equation 2 and Equation 3, a linear approximation can be used to yield the convective diffusion equation (Miller & Cunge, 1975):

101)
$$\frac{\partial Q}{\partial t} + c\frac{\partial Q}{\partial x} = \mu\frac{\partial^2 Q}{\partial x^2} + cq_L$$

where c = flood wave celerity, μ = hydraulic diffusivity, and q_L = lateral inflow. Flood wave celerity and hydraulic diffusivity can be expressed as:

77 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-reach-routing-methods-within-hec-hms/applying-the-muskingum-routing-method

102) $$c = \frac{dQ}{dA}$$

103) $$\mu = \frac{Q}{2BS_o}$$

where B = top width of the water surface.

Recall that HEC-HMS solves the the following equation recursively to compute O_t given inflow (I_t and I_{t-1}), an initial condition ($O_{t=0}$), K, and X when using the Muskingum method:

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

Using a finite difference approximation of the partial derivatives in Equation 3 and combination with Equation 6 yields:

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

The C_1, C_2, C_3, and C_4 coefficients are:

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

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

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

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

Within the previously mentioned Muskingum method, the X parameter is a dimensionless coefficient that lacks a strong physical meaning. Cunge (1969) evaluated the numerical diffusion that is produced through the use of Equation 6 and set this equal to the physical diffusion represented by Equation 3. This yielded the following representations for K and X (Ponce & Yevjevich, 1978):

110) $$K = \frac{\Delta x}{c}$$

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

Since c, Q, and B can change during the passage of a flood wave, the coefficients C_1, C_2, C_3, and C_4 also change. As such, the C_1, C_2, C_3, and C_4 coefficients are recomputed each time and distance step (Δt and Δx) using the algorithm proposed by Ponce (1986).

11.8.2 Required Parameters

The parameters that are required to utilize this method within HEC-HMS are the initial condition, the reach length [ft or m], the friction slope [ft/ft or m/m], Manning's n roughness coefficient, a space-time interval method and value, an index method and value, and a cross-section shape and parameters/dimensions. An optional invert can also be specified.

Two options for specifying the initial condition are included: outflow equals inflow and specified discharge [ft^3/sec or m^3/sec]. The first option assumes that the initial outflow is the same as the initial inflow to the reach from the upstream elements which is equivalent to the assumption of a steady-state initial condition. The second option is most appropriate when there is observed streamflow data at the end of the reach.

The reach length should be set as the total length of the reach element while the friction slope should be set as the average friction slope for the entire reach. If the friction slope varies significantly throughout the stream represented by the reach, it may be necessary to use multiple reaches with different slopes. If no information is available to estimate the friction slope, the bed slope can be used as an approximation. The Manning's n roughness coefficient should be set as the average value for the whole reach. This value can be estimated using "reference" streams with established roughness coefficients or through calibration.

The choices of space and time steps (Δx and Δt) are critical to ensure accuracy and stability. Three options for specifying a space-time method are provided within HEC-HMS: 1) Auto DX Auto DT, 2) Specified DX Auto DT, and 3) Specified DX Specified DT. When the Auto DX Auto DT method is selected, space and time intervals that attempt to maintain numerical stability will automatically be selected. Δt is selected as the minimum of either the user-specified time step or the travel time through the reach (rounded to the nearest multiple or divisor of the user-specified time step). Once Δt is computed, Δx is computed as:

112)

$$\Delta x = c\Delta t$$

When the Specified DX Auto DT method is selected, the specified number of subreaches will be used while automatically varying the time interval to take as long a time interval as possible while also maintaining numerical stability. When the Specified DX Specified DT method is selected, the specified number of subreaches and subintervals (rounded to the nearest multiple or divisor of the user-specified time step) will be used throughout the entire simulation.

Upon completion of a simulation, the minimum and maximum celerity of the routed hydrograph will be displayed as notes. Also, a reference space step, Δx_{ref}, will be computed using methodology presented in Engineer Manual 1110-2-1417 Flood-Runoff Analysis (U.S. Army Corps of Engineers, 1994):

113)

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

where Q_0 = reference flow, which is computed from the inflow hydrograph as:

114)

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

where Q_B = baseflow and Q_{peak} = inflow peak. If Δx_{ref} is not equal to the Δx that was used during the routing computations, a note will be issued.

The index method is used in conjunction with the physical properties of the channel and the previously mentioned Δt and Δx interval selection to discretize the routing reach in both space and time. Appropriate reference flows and flood wave celerities are dependent upon the physical properties of the channel as well as the flood event(s) in question. Experience has shown that a reference flow (or celerity) based upon average values of the hydrograph in question (i.e. midway between the base flow and the peak flow) is, in general, the most suitable choice. Reference flows (or celerities) based on peak values tend to numerically accelerate the flood wave more than would occur in nature while the converse is true if a low reference flow (or celerity) is used (Ponce, 1983).

Six options are provided for specifying the cross-section shape: circle, eight-point, rectangle, tabular, trapezoid, and triangle. The circle shape is not meant to be used for pressure flow or pipe networks, but is suitable for representing a free surface inside a pipe. Depending upon the chosen shape, additional information will have to be entered to describe the size of the cross-section shape. This information may include a diameter (circle) [ft or m], bottom width (deep, rectangle, and/or trapezoid) [ft or m], or side slope (trapezoid and triangle) [ft/ft or m/m]. In all cases, cross-section shapes must be defined in such a way that all possible flow depths that will be simulated will be completely confined within the defined shape.

The tabular shape option allows for the use of user-defined elevation vs. discharge, elevation vs. area, and elevation vs. width relationships (which are all paired data objects). This option is typically used when relationships derived from hydraulic simulations are available. When the tabular shape is selected, no Manning's n roughness coefficients need to be entered.

When using the eight-point shape, a simplified cross-section (which is a paired data object) with eight station vs. elevation values must be selected. The cross-section is typically configured to represent the main channel plus left and right overbank areas. As such, separate Manning's n values are required for each overbank.

Many of the aforementioned parameters are typically estimated using GIS. However, field survey data may be necessary to accurately determine reach lengths, friction or bed slopes, and/or cross-section shape parameters.

A tutorial describing an example application of this loss method, including parameter estimation and calibration, can be found here: Applying the Muskingum-Cunge Routing Method[78].

11.9 Normal Depth Model

11.9.1 Basic Concepts

The Normal Depth method is very similar to the aforementioned Modified Puls method. Specifically, storage within a reach is assumed to be primarily dependent upon outflow. However, the Normal Depth method automatically develops storage vs. discharge relationships using Manning’s equation, a normal depth assumption, and user-defined channel properties. This method allows for more efficient parameterization,

78 https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/applying-reach-routing-methods-within-hec-hms/applying-the-muskingum-cunge-routing-method

but also loses the ability to simulate backwater effects and the impacts of hydraulic structures since hydraulic simulations are no longer used to develop storage vs. discharge relationships.

11.9.2 Required Parameters

The parameters that are required to utilize this method within HEC-HMS are the initial condition, the reach length [ft or m], bottom slope [ft/ft or m/m], Manning's n roughness coefficient, index flow [ft^3/s or m^3/s], and cross-section shape and parameters/dimensions. An optional invert can also be specified.

Two options for specifying the initial condition are included: outflow equals inflow and specified discharge [ft^3/sec or m^3/sec]. The first option assumes that the initial outflow is the same as the initial inflow to the reach from the upstream elements which is equivalent to the assumption of a steady-state initial condition. The second option is most appropriate when there is observed streamflow data at the end of the reach.

The reach length should be set as the total length of the reach element while the bed slope should be set as the average bed slope for the entire reach. If the slope varies significantly throughout the stream represented by the reach, it may be necessary to use multiple reaches with different slopes. The Manning's n roughness coefficient should be set as the average value for the whole reach. This value can be estimated using "reference" streams with established roughness coefficients or through calibration.

The index flow should represent the expected maximum flow within the reach. Storage-discharge for the reach will be created ranging from zero to 1.5 times the index flow. The index flow is also used in combination with the reach length and channel geometry to compute the travel time through the reach. The number of subreaches is then computed by dividing the travel time by the simulation time interval.

Five options are provided for specifying the cross-section shape: circle, eight-point, rectangle, trapezoid, and triangle. The circle shape is not meant to be used for pressure flow or pipe networks but is suitable for representing a free surface inside a pipe. Depending upon the shape you choose, additional information will have to be entered to describe the size of the cross-section shape. This information may include a diameter (circle) [ft or m], bottom width (deep, rectangle, and/or trapezoid) [ft or m], or side slope (trapezoid and triangle) [ft/ft or m/m]. In all cases, cross-section shapes must be defined in such a way that all possible flow depths that will be simulated will be completely confined within the defined shape.

When using the eight-point shape, a simplified cross-section (which is a paired data object) with eight station vs. elevation values must be selected. The cross-section is typically configured to represent the main channel plus left and right overbank areas. As such, separate Manning's n values are required for each overbank. Storage vs. discharge relationships (ranging from zero to 1.5 times the index flow) will be automatically generated for the given channel properties using Manning's equation and a normal depth assumption.

Many of the aforementioned parameters are typically estimated using GIS. However, field survey data may be necessary to accurately determine reach lengths, bed slopes, and/or cross-section shape parameters.

11.10 Straddle Stagger Model

11.10.1 Basic Concepts

The Straddle Stagger method (or progressive average lag method) uses empirical representations of translation and attenuation processes to route water through a reach. Specifically, inflow to the reach is lagged in time and then averaged over a specified duration to produce the final outflow.

11.10.2 Required Parameters

The parameters that are required to utilize this method within HEC-HMS are the initial condition, lag [minutes], and duration [minutes]. Two options for specifying the initial condition are included: outflow equals inflow and specified discharge [ft^3/sec or m^3/sec]. The first option assumes that the initial outflow is the same as the initial inflow to the reach from the upstream elements which is equivalent to the assumption of a steady-state initial condition. The second option is most appropriate when there is observed streamflow data at the end of the reach. The lag parameter specifies the travel time through the reach; inflow to the reach is delayed in time by an amount equal to the specified lag. The duration parameter specifies the amount of spreading in a flood peak as it travels through the reach. The delayed inflows are averaged over this specified time duration. The duration parameter loses physical meaning when it is greater than twice the lag time. These parameters are most often estimated using observed data and refined through calibration.

11.11 Applicability and Limitations of Channel Routing Models

The choice of a channel routing method depends upon the purposes of the study, the desired level of detail, and the physical characteristics of the watershed in question. In some applications, the transformation of precipitation to streamflow at a particular point of interest can be adequately represented without channel routing. Also, some of the aforementioned methods utilize parameters that cannot be estimated using measurable channel characteristics; they can only be determined through the use of observed data. Moreover, only one of the aforementioned methods (Modified Puls) can simulate backwater effects or the impacts of hydraulic structures. Finally, some of the aforementioned routing methods are only appropriate for use in steep streams with bed slopes > 10 ft/mi.

Initial estimates of channel routing parameters should be subjected to a model calibration process where computed outputs are compared against observed data and model parameters are modified in order to achieve an adequate fit. Also, best estimate parameters derived through the aforementioned model calibration process should be tested through a model validation process where computed results, without any further parameter modifications, are used to compute outputs which are compared against observed data for independent events that were not considered during model calibration. Storm events used during both model calibration and validation should be approximately equal to the magnitude of events that are being considered within the particular application.

The following table contains a list of various advantages and disadvantages regarding the aforementioned channel routing methods available for use within HEC-HMS. However, these are only guidelines and should be supplemented by knowledge of, and experience with, the methods and the watershed in question.

Method	Advantages	Disadvantages
Kinematic Wave	• Predicted values are in accordance with open channel flow theory (for steep channels). • Parameters can be estimated using measurable channel characteristics. • Can use multiple cross-section shapes.	• Method is <u>less</u> parsimonious than simpler routing methods; it requires many more parameters. • <u>Cannot</u> simulate backwater effects or impacts of hydraulic structures. • Only appropriate for use in steep streams (bed slopes > 10 ft/mi). • HEC-HMS implementation cannot use cross-section shapes that include overbank areas.
Lag	• Simple, parsimonious method.	• Method may be too simple; no attenuation effects are simulated by this method. • Parameters <u>cannot</u> be estimated using measurable channel characteristics. • Only appropriate for use in streams that experience no attenuation. • <u>Cannot</u> simulate backwater effects or impacts of hydraulic structures.
Lag and K	• Well established and documented method through its use by NWS. • Simple, parsimonious method. • Can simulate variable translation and attenuation.	• Similar to disadvantages of the Muskingum method. • Numerical errors can arise through the use of flawed inflow vs translation functions.
Modified Puls	• <u>Can</u> simulate backwater effects and impacts of hydraulic structures.	• Method is <u>less</u> parsimonious than simpler methods; it requires many more parameters. • Requires hydraulic simulations to derive accurate storage vs. outflow relationships; consequently, this method can be difficult to parameterize and calibrate.

Method	Advantages	Disadvantages
Muskingum	• "Mature" method that has been used successfully in thousands of studies throughout the U.S. • Method is parsimonious; it requires only a few parameters.	• Method may be too simple to accurately predict floodwave translation and attenuation. • Only appropriate for use in moderately steep streams (bed slopes > 2 ft/mi). • Cannot simulate variable translation and attenuation. • Cannot simulate backwater effects or impacts of hydraulic structures.
Muskingum-Cunge	• Similar to advantages of the Muskingum method. • Predicted values are in accordance with open channel flow theory. • Parameters can be estimated using measurable channel characteristics. • Can use cross-section shapes that include overbank areas.	• Similar to disadvantages of the Muskingum method. • Method is less parsimonious than Muskingum; it requires many more parameters.
Normal Depth	• Similar to advantages of the Modified Puls method. • Does not require the use of hydraulic simulations to derive storage vs. outflow relationships. • Parameters can be estimated using measurable channel characteristics.	• Similar to disadvantages of the Modified Puls method. • Since normal depth is assumed to derive storage vs. outflow relationships, backwater effects or impacts of hydraulic structures cannot be simulated.
Straddle Stagger	• Similar to advantages of the Lag method. • This method can simulate attenuation effects.	• Similar to disadvantages of the Lag method. • Not well documented nor widely used.

11.12 Channel Flow References

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12 Erosion and Sediment Transport (Under Construction)

Surface erosion, reservoir sedimentation, channel sediment transport (including erosion and deposition) are integral components of watershed management, natural resources conservation planning, evaluation of water quality best management practices (BMPs), and total maximum daily loads (TMDLs) studies. These processes, encompassing surface soil erosion and sediment transport, have far-reaching implications for various critical aspects, including agricultural land productivity, the functioning of aquatic ecosystems, the recreational quality of rivers and reservoirs, the navigability of channels, and the operational flexibility of reservoirs in relation to water supply, environmental flows, and flood risk management objectives.

In this section, the utilization of HEC-HMS emerges as a valuable and indispensable tool for enhancing modelers' understanding of the potential impacts stemming from surface erosion and sediment transport within channel and reservoir systems on watershed dynamics. By leveraging the capabilities of HEC-HMS, modelers can achieve more reasonable predictions of peak flows and sediment transport, facilitating the simulation of diverse scenarios while incorporating essential hydrological and sediment data. This software provides users with a powerful tool to conduct thorough evaluations of heightened risks related to sediment transport. These risks encompass erosion and deposition within the system. By doing so, it facilitates the creation of robust strategies for erosion control, the maintenance of water quality, safeguarding critical infrastructure, and the establishment of sustainable water supplies.

This section delineates the techniques for addressing surface erosion, reservoir sedimentation, and channel sediment transport incorporated within the program. Especially, each of these methods of reach and reservoir elements calculates a sediment transport/routing downstream, utilizing an upstream sediment distribution as an initial boundary condition. The mechanism for achieving this involves solving both the continuity and momentum equations. Within this chapter, we provide a concise overview of the foundational equations, simplifications, and alternative model solutions.

12.1 Erosion Methods

A Subbasin Element represents a drainage area where precipitation induces surface runoff, influenced by burn or unburn conditions. Within this catchment, erosion occurs due to various physical processes, notably in post-fire scenarios. Raindrops initiate erosion by impacting the ground, dislodging soil particles, which are carried by overland flow. This flow also imparts erosive energy to the terrain, potentially further disrupting the topsoil layer. As overland flow intensifies, it becomes channeled into rills, concentrating erosive energy and exacerbating surface erosion. The extent of erosion closely correlates with precipitation rate, land surface slope, and surface condition. Occasionally, soil eroded from higher up in the catchment may settle before reaching the subbasin outlet.

All Surface Erosion Methods for the subbasin element share certain simulation features. Each method calculates the total sediment load transported out of the subbasin during a storm, repeating this process for each storm within the simulation time window. The computed sediment load is then distributed into a time-series of sediment discharge from the subbasin. Each method was developed with distinct purposes and applications, as outlined below:

Erosion Method	Urban Watershed	Natural Watershed	Burn Watershed
Modified USLE[79]		👍	
Build-up Wash-off[80]	👍		
LA Debris Equation 1[81]		👍	👍
LA Debris Equations 2-5[82]		👍	👍
Multi-Sequence Debris Prediction Method (MSDPM) (see page 230)		👍	👍
USGS Emergency Assessment Debris Model[83]		👍	👍
USGS Long-Term Debris Model[84]		👍	👍
2D Sediment Transport[85]		👍	

Another common feature among these methods is the treatment of Grain Size Distribution. Initially, all methods compute the overall Sediment Discharge, encompassing all grain sizes. Subsequently, a Gradation Curve specifies the proportion of the total sediment discharge allocated to each grain size class or subclass. Users define and select a gradation curve for each subbasin, allowing for distinctions in erosion, deposition, and resuspension processes within each subbasin. These processes are often collectively represented by an Enrichment Ratio.

12.1.1 2D Sediment Transport

A brief description of the Finite-Volume discretization of the total-load transport equation is provided here without any derivation or details. For a more comprehensive understanding, including information on advection schemes, gradient operators, and related aspects, please refer to the "HEC-RAS 2D Sediment Transport Technical Reference Manual" by Sánchez et al. (2019) (2D Sediment Manual[86]). This reference is relevant because HEC-HMS shares the same 2D Sediment Transport engine as HEC-RAS. The 2D Transport Module in both applications employs explicit and implicit Finite-Volume methods to solve generic Advection-Diffusion equations. The final form of the discretized total-load advection-diffusion equation is given by

79 https://www.hec.usace.army.mil/confluence/display/HMSTRM/Modified+USLE?src=contextnavpagetreemode
80 https://www.hec.usace.army.mil/confluence/display/HMSTRM/Build-up+Wash-off?src=contextnavpagetreemode
81 https://www.hec.usace.army.mil/confluence/display/HMSTRM/LA+Debris+Equation+1?src=contextnavpagetreemode
82 https://www.hec.usace.army.mil/confluence/display/HMSTRM/LA+Debris+Equations+2-5?src=contextnavpagetreemode
83 https://www.hec.usace.army.mil/confluence/display/HMSTRM/USGS+Emergency+Assessment+Debris+Model?src=contextnavpagetreemode
84 https://www.hec.usace.army.mil/confluence/display/HMSTRM/USGS+Long-Term+Debris+Model?src=contextnavpagetreemode
85 https://www.hec.usace.army.mil/confluence/display/HMSTRM/2D+Sediment+Transport?src=contextnavpagetreemode
86 https://www.hec.usace.army.mil/confluence/display/RAS/2D+Sediment+Manual

$$\frac{\Omega_P^{n+1} C_{tk,P}^{n+1}}{\Delta t \beta_{tk,P}^{n+1}} = \frac{\Omega_P^{n} C_{tk,P}^{n}}{\Delta t \beta_{tk,P}^{n}} + \sum_f \left[\frac{A_f \varepsilon_{tk,f}}{r_{PN}} \left(C_{tk,N} - C_{tk,P} \right) - F_f C_{tk,f} \right]^{n+\theta} + \left(E_{tk}^{HF} - D_{tk}^{HF} + S_{tk} \right)_P \Delta A_P^W$$

where Ω = cell volume

P = subscript indicating cell

f = subscript indicating face between cells *P* and *N*

N = subscript indicating neighboring cell to *P* and sharing face *f*

n = superscript indicating time step

C_{tk} = total-load sediment concentration of the k^{th} grain class [M/L^3]

B_{tk} = total-load correction factor for the k^{th} grain class

ε_{tk} = total-load diffusion coefficient corresponding to the k^{th} grain class

A_f = face vertical area [L^2/T]

F_f = face-normal water flow [L^3/T]

E_{tk}^{HF}= total-load erosion rate in hydraulic flow [M/T/L^2]

D_{tk}^{HF} = total-load deposition rate in hydraulic flow [M/T/L^2]

S_{tk} = total-load source/sink term [M/T/L^2]

ΔA_p^W = cell wetted horizontal area [L^2]

r_{pn} = distance between cell points *N* and *P* [L]

θ = implicit weighting factor [-]

n + θ = superscript representing the temporal weighting (Generalized Euler scheme)

When the implicit weighting factor is equal to 1, the scheme reduces to the first-order fully implicit Backward Euler scheme. When the implicit weighting factor is equal to 0.5, the scheme is the second-order Crank-Nicholson scheme. An implicit discretization is utilized for robustness. However, future versions will have the option to use an explicit scheme.

12.1.1.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the total load scaling factor, critical mobility scaling factor, sheet & splash erodibility coefficient, sediment total roughness factor, and adaptation coefficient.

A tutorial using the 2D Sediment Transport simulation can be found here: TBD.

12.1.1.1.1 A Note on Parameter Estimation

For comprehensive details regarding input parameters, kindly consult the "HEC-RAS 2D Sediment Transport Technical Reference Manual (2D Sediment Manual[87])" authored by Sánchez et al. in 2019.

12.1.2 Build-up Wash-off

The Build-Up Wash-Off (BUWO) method is a hydrological modeling approach used to simulate the accumulation and removal of pollutants, such as sediment, nutrients, and contaminants, from impervious surfaces in urban areas during rainfall events. This method is commonly employed in the field of stormwater management and urban water quality assessment. It helps estimate the quantity and quality of runoff from urban areas and the subsequent pollutant loads entering receiving water bodies. Build up may be a function of time, traffic flow, dry fallout and street sweeping. During a storm event, the material is then washed off into the drainage system. Although the bulid-up Wash-off option is conceptually appealing, the reliability and credibility of simulation may be difficult to establish without local data for calibration and validation (Huber and Dickinson, 1988).

The Michaelis-Menten Build-Up Equation is a mathematical model used to describe the accumulation of pollutants on impervious surfaces in urban areas over time. In the context of urban stormwater management and water quality modeling, the Michaelis-Menten Build-Up Equation is used to estimate how a specific pollutant, such as sediment, heavy metals, or nutrients, accumulates on impervious surfaces (e.g., roads, parking lots) as a function of time, especially during dry weather periods. The equation is often used in conjunction with the Build-Up Wash-Off (BUWO) method to model the buildup and subsequent wash-off of pollutants during rainfall events. The general form of the Michaelis-Menten Build-Up Equation is as follows (Huber and Dickinson, 1988 and Neitsch, Arnold, Kiniry, and Williams, 2009) :

$$\mathrm{SED} = \frac{\mathrm{SED}_{\max} \times t_d}{(t_{\mathrm{half}} + t_d)}$$

$$\begin{aligned}
&\mathrm{SED} = \text{ Solids Build Up (kg/curb km) td days since } SED = 0 \\
&\mathrm{SED}_{\max} = \text{ Maximum Accumulation of Solids Possible for the Land Type (kg/curb km)} \\
&\mathrm{t}_{\mathrm{half}} = \text{ Length of Time needed for Solids to Increase from 0 to half of } \mathrm{SED}_{\max} \text{ (days)} \\
&\mathrm{t}_{\mathrm{d}} = \text{Time dry in days}
\end{aligned}$$

The Huber-Dickinson equation is a commonly used mathematical model for simulating the wash-off of pollutants from impervious surfaces in urban areas during rainfall events. It's named after its developers, W.C. Huber and R.E. Dickinson, who introduced the model in the 1988 publication titled "Stormwater Management Model User's Manual, Version III." The Huber-Dickinson equation is particularly associated with the United States Environmental Protection Agency's (EPA) Storm Water Management Model (SWMM), which is widely used for stormwater management and urban hydrology modeling. The equation estimates the wash-off of pollutants as a function of various factors, including rainfall characteristics, land use, and the pollutant load on impervious surfaces. The general form of the Huber-Dickinson Wash-Off equation is as follows (Huber and Dickinson, 1988 and Neitsch, Arnold, Kiniry, and Williams, 2009):

87 https://www.hec.usace.army.mil/confluence/display/RAS/2D+Sediment+Manual

$$Y_{sed} = SED_0 \times \left(1 - e^{-kk \cdot t}\right)$$

$$\begin{aligned}
&Y_{sed} = \text{Cumulative Amount of Solids Washed Off at Time t (kg/curb km)}\\
&SED_0 = \text{Amount of Solids Build Up on the Impervious Area at the Beginning of the Precipitaiton Event (kg/curb km)}\\
&kk = urb_{coef} \times q_{peak}\\
&\quad urb_{coef} = \text{Wash Off Calibration Coefficient } \left(0.039 - 0.39 \text{ mm}^{-1}\right)\\
&\quad q_{peak} = \text{Peak Event Flow Rate (mm/hr)}
\end{aligned}$$

$$\begin{aligned}
&Y_{sed} \times L_{curb} = \text{Sediment Load}\\
&\quad L_{curb} = \text{Length of Curb (curb km)}
\end{aligned}$$

Street cleaning is a common practice in urban areas to manage the accumulation of solid debris and litter in street gutters. While it has traditionally been believed that street cleaning positively impacts the quality of urban runoff, there has been a scarcity of data available to quantitatively assess this influence. Previous studies, such as those conducted under the EPA Nationwide Urban Runoff Program (NURP), have generally indicated limited improvements in runoff quality as a result of street sweeping unless carried out on a daily basis (EPA, 1983b). The equation or model for street sweeping or pollutant removal during street cleaning operations can vary and is typically developed based on local practices, equipment types, cleaning frequencies, and specific pollutant removal efficiencies associated with the street sweeping process. These equations or models may not have a standardized or widely accepted form, as they often depend on the unique characteristics of the street cleaning operations in a particular area.

This BUWO Method includes four parameters to describe the street sweeping operations within the subbasin. The Density specifies the total length of street curb whether or not the curb is subject to sweeping operations. The density should consider whether the street has curbs on one side or both sides of the street. The Sweeping Percentage specifies the percentage of the curb length subject to sweeping. The percentage should account for the possible presence of parked cars which result in missed curb. The Efficiency Percentage specifies the efficiency of the sweeping equipment at removing accumulated sediment. Finally, the Interval specifies the number of days between scheduled sweeping operations.

The general form of the Huber-Dickinson street sweeping removal equation is as follows (Huber and Dickinson, 1988 and Neitsch, Arnold, Kiniry, and Williams, 2009):

$$SED = SED_0 \times (1 - fr_{av} \times reff)$$

$$\begin{aligned}
&SED = \text{Amount of Solids Remaining after Sweeping (kg/curbkm)}\\
&SED_0 = \text{Amount of Solids Present prior to Sweeping (kg/curbkm)}\\
&fr_{av} = \text{Fraction of the Curb Length Available for Sweeping}\\
&reff = \text{Removal Efficiency of the Sweeping Equipment}
\end{aligned}$$

12.1.2.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the initial time, half time, maximum solid amount, density, sweeping percentage, efficiency percentage, interval, and wash-off coefficient.

A tutorial using the BUWO simulation can be found here: TBD.

12.1.2.1.1 A Note on Parameter Estimation

Street sweeping parameters exhibit variability and are typically tailored to local conditions, accounting for factors like equipment types, cleaning frequencies, and the efficiency of pollutant removal during the street sweeping process.

12.1.3 LA Debris Equation 1

The Los Angeles District Debris Method - Equation 1 (Gatwood, Pedersen, and Casey, 2000) is employed to simulate events in watersheds ranging from 0.1 mi² to 3 mi² in size, where peak flow data is unavailable . This equation was derived from a comprehensive dataset comprising 349 observations collected across 80 watersheds in Southern California. All the factors included in this equation demonstrated statistical significance at a confidence level of 0.99. It is worth noting that the LA Debris Method Equation 1 exhibits its highest efficacy in arid or semi-arid regions, precisely the same geographical area where it was originally developed.

$$\log D_y = 0.65(\log P) + 0.62(\log RR) + 0.18(\log A) + 0.12(FF)$$

$D_y =$ Unit Debris Yield $\left(yd^3/\mathrm{mi}^2\right)$

$P =$ Maximum 1-Hour Precipitation (inch)

$RR =$ Relief Ration (ft/mi)

$$RR = \frac{h_2 - h_1}{L}$$

$h_2 =$ Highest Elevation in the Watershed (ft)

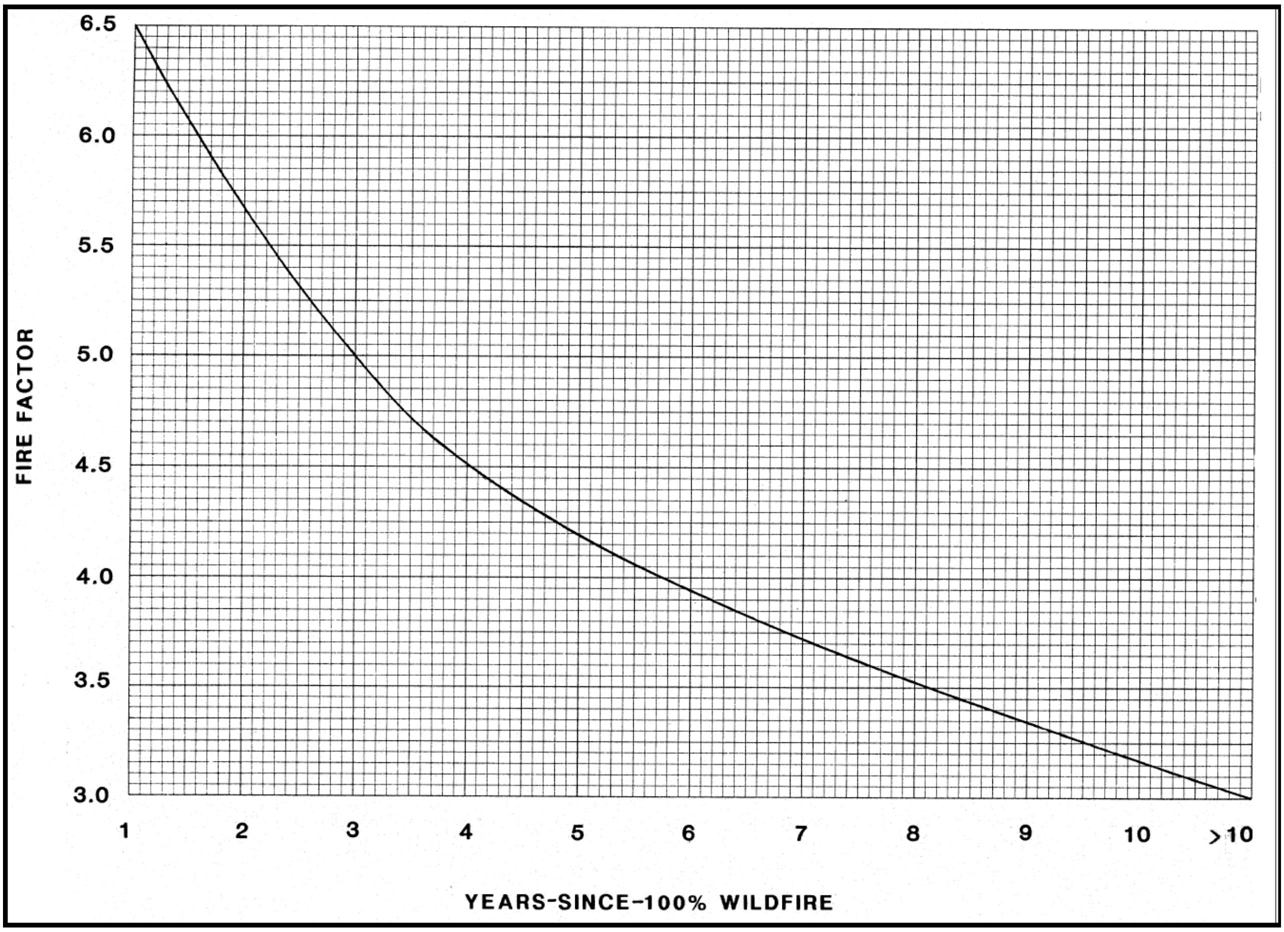
$h_1 =$ Lowest Elevation in the Watershed (ft)

$L =$ Maximum Stream Length (mi)

$A =$ Drainage Area (ac)

$FF =$ Non-Dimensional Fire Factor

The Fire Factor (FF) can be approximated using the Factor Factor Curve (watersheds ranging from 0.1 mi² to 3 mi²) provided below, which illustrates a scenario of 100% combustion. An illustration of how to calculate the Fire Factor in cases of partial combustion can be found in the Los Angeles Debris Method Manual (Gatwood, Pedersen, and Casey, 2000).



12.1.3.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the maximum 1-hour precipitation [inches or millimeters], relief ratio [ft/mi or m/km], and non-dimensional fire factor.

A tutorial using the Los Angeles District Debris Method - Equation 1 in an event simulation can be found here: Applying Debris Yield Methods in HEC-HMS[88].

12.1.3.1.1 A Note on Parameter Estimation

HEC-HMS initially assigns a default value of 1.0 to the Adjustment-Transposition (A-T) factor. However, it's essential to fine-tune and verify this value by taking into account the disparities in geomorphological characteristics between the specific watershed under consideration and the original watershed (San Gabriel Mountains, CA) from which the regression equation was originally derived.

88 https://www.hec.usace.army.mil/confluence/display/HMSGUIDES/Applying+Debris+Yield+Methods+in+HEC-HMS

The **Flow Rate Threshold** parameter was introduced as an independent variable to segment storm events for continuous simulation. It establishes the lower boundary for direct runoff flow rate, marking the commencement of a debris flow event when the direct runoff exceeds this threshold. Conversely, the event concludes when the direct runoff drops below the specified threshold. This parameter assumes particular significance in the calibration process, especially for continuous simulations.

12.1.4 LA Debris Equations 2-5

The Los Angeles District Debris Method - Equation 2 Through 5 (Gatwood, Pedersen, and Casey, 2000) was developed from statistical analysis of data from watersheds with an area from 3.0 mi² to 200.0 mi². This method may also be used for drainage areas less than 3 mi2. The Los Angeles District Debris Method EQ 2-5 works best in arid or semi arid regions of Southern California where it was developed. The Component Editor is shown in the next two figures.

12.1.4.1 Los Angeles District Debris Method - Equation 2

The Los Angeles District Debris Method - Equation 2 (Gatwood, Pedersen, and Casey, 2000) is employed to simulate events in watersheds ranging from 3 mi² to 10 mi² in size.

$$\log D_y = 0.85(\log Q) + 0.53(\log RR) + 0.04(\log A) + 0.22(FF)$$

$$D_y = \text{ Unit Debris Yield } \left(yd^3/\text{mi}^2\right)$$
$$Q = \text{ Unit Peak Runoff } \left(ft^3/s/\text{mi}^2\right)$$
$$RR = \text{ Relief Ration (ft/mi)}$$
$$RR = \frac{h_2 - h_1}{L}$$
$$h_2 = \text{ Highest Elevation in the Watershed (ft)}$$
$$h_1 = \text{ Lowest Elevation in the Watershed (ft)}$$
$$L = \text{ Maximum Stream Length (mi)}$$
$$A = \text{ Drainage Area (ac)}$$
$$FF = \text{ Non-Dimensional Fire Factor}$$

12.1.4.2 Los Angeles District Debris Method - Equation 3

The Los Angeles District Debris Method - Equation 3 (Gatwood, Pedersen, and Casey, 2000) of the Los Angeles Debris Method is employed to simulate events in watersheds ranging from 10 mi² to 25 mi² in size.

$$\log D_y = 0.88(\log Q) + 0.48(\log RR) + 0.06(\log A) + 0.20(FF)$$

$$\begin{aligned} D_y &= \text{ Unit Debris Yield } \left(yd^3/\text{mi}^2\right) \\ Q &= \text{ Unit Peak Runoff } \left(ft^3/s/\text{mi}^2\right) \\ RR &= \text{ Relief Ration (ft/mi)} \\ RR &= \frac{h_2 - h_1}{L} \\ h_2 &= \text{ Highest Elevation in the Watershed (ft)} \\ h_1 &= \text{ Lowest Elevation in the Watershed (ft)} \\ L &= \text{ Maximum Stream Length (mi)} \\ A &= \text{ Drainage Area (ac)} \\ FF &= \text{ Non-Dimensional Fire Factor} \end{aligned}$$

$$

12.1.4.3 Los Angeles District Debris Method - Equation 4

The Los Angeles District Debris Method - Equation 4 (Gatwood, Pedersen, and Casey, 2000) of the Los Angeles Debris Method is employed to simulate events in watersheds ranging from 25 mi² to 50 mi² in size.

$$\log D_y = 0.94(\log Q) + 0.32(\log RR) + 0.14(\log A) + 0.17(FF)$$

$$\begin{aligned} D_y &= \text{ Unit Debris Yield } \left(yd^3/\text{mi}^2\right) \\ Q &= \text{ Unit Peak Runoff } \left(ft^3/s/\text{mi}^2\right) \\ RR &= \text{ Relief Ration (ft/mi)} \\ RR &= \frac{h_2 - h_1}{L} \\ h_2 &= \text{ Highest Elevation in the Watershed (ft)} \\ h_1 &= \text{ Lowest Elevation in the Watershed (ft)} \\ L &= \text{ Maximum Stream Length (mi)} \\ A &= \text{ Drainage Area (ac)} \\ FF &= \text{ Non-Dimensional Fire Factor} \end{aligned}$$

$$

12.1.4.4 Los Angeles District Debris Method - Equation 5

The Los Angeles District Debris Method - Equation 5 (Gatwood, Pedersen, and Casey, 2000) of the Los Angeles Debris Method is employed to simulate events in watersheds ranging from 50 mi² to 200 mi² in size.

$$\log D_y = 1.02(\log Q) + 0.23(\log RR) + 0.16(\log A) + 0.13(FF)$$

$$\begin{aligned}
&D_y = \text{ Unit Debris Yield } \left(yd^3/\text{mi}^2\right)\\
&Q = \text{ Unit Peak Runoff } \left(ft^3/s/\text{mi}^2\right)\\
&RR = \text{ Relief Ration (ft/mi)}\\
&RR = \frac{h_2 - h_1}{L}\\
&h_2 = \text{ Highest Elevation in the Watershed (ft)}\\
&h_1 = \text{ Lowest Elevation in the Watershed (ft)}\\
&L = \text{ Maximum Stream Length (mi)}\\
&A = \text{ Drainage Area (ac)}\\
&FF = \text{ Non-Dimensional Fire Factor}
\end{aligned}$$

The Fire Factor (FF) can be approximated using the Factor Factor Curve (watersheds ranging from 3.0 mi² to 200.0 mi²) provided below, which illustrates a scenario of 100% combustion. An illustration of how to calculate the Fire Factor in cases of partial combustion can be found in the Los Angeles Debris Method Manual (Gatwood, Pedersen, and Casey, 2000).

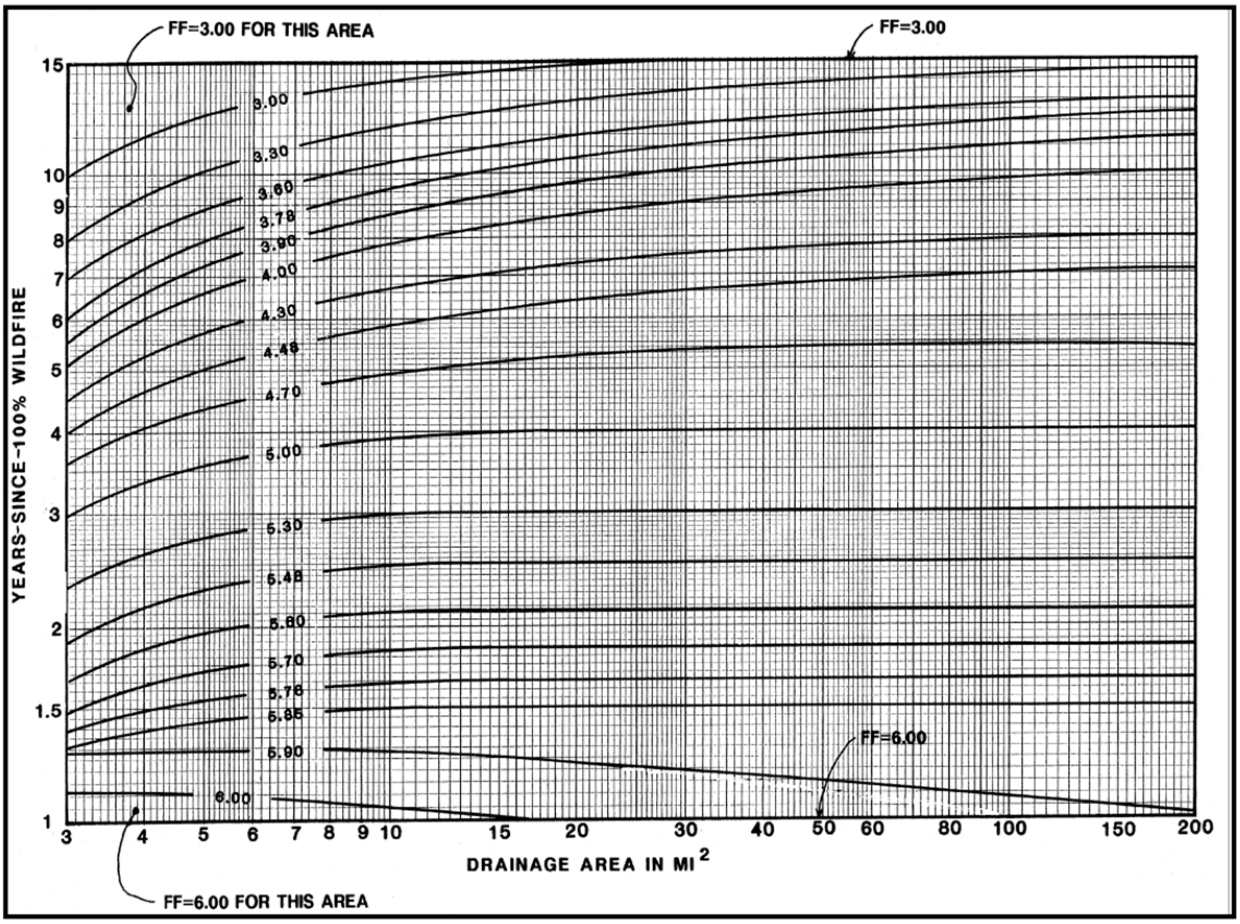


Figure 3: Fire Factor Curves for Watersheds 3.0 to 200 mi²

12.1.4.5 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the maximum 1-hour precipitation [inches or millimeters], relief ratio [ft/mi or m/km], and non-dimensional fire factor.

A tutorial using the Los Angeles District Debris Method - Equations 2 - 5 can be found here: Hydrologic Modeling and Debris Flow Estimation for Post Wildfire Conditions[89].

12.1.4.5.1 A Note on Parameter Estimation

HEC-HMS initially assigns a default value of 1.0 to the Adjustment-Transposition (A-T) factor. However, it's essential to fine-tune and verify this value by taking into account the disparities in geomorphological characteristics between the specific watershed under consideration and the original watershed (San Gabriel Mountains, CA) from which the regression equation was originally derived.

The **Flow Rate Threshold** parameter was introduced as an independent variable to segment storm events for continuous simulation. It establishes the lower boundary for direct runoff flow rate, marking the commencement of a debris flow event when the direct runoff exceeds this threshold. Conversely, the event concludes when the direct runoff drops below the specified threshold. This parameter assumes particular significance in the calibration process, especially for continuous simulations.

Calibrating and validating peak runoff is a crucial step in the process, particularly in the context of post-wildfire conditions. This calibration accounts for several key factors: the diminished precipitation interception by the vegetation canopy and forest litter/duff zone, the heightened susceptibility of the soil to raindrop impact, and the reduced soil infiltration capacity due to alterations in soil properties.

12.1.5 Modified USLE

The MUSLE, or Modified Universal Soil Loss Equation (Williams, 1975), is a mathematical model used in soil science and hydrology to estimate soil erosion. It was developed as an extension and modification of the original Universal Soil Loss Equation (USLE). The MUSLE takes into account factors such as land use, topography, soil erodibility, and climate conditions to predict the potential erosion rate in a particular area. The modifications to the original USLE equation changed the formulation to calculate erosion from surface runoff instead of precipitation. The other components of the original formulation remained the same. The method works best in agricultural environments where it was developed. However, some users have adapted it to construction and urban environments.

89 https://www.hec.usace.army.mil/confluence/display/HMSGUIDES/Hydrologic+Modeling+and+Debris+Flow+Estimation+for+Post+Wildfire+Conditions

$$\text{Sed} = 11.8\left(Q_{\text{surf}} \times q_{\text{peak}}\right)^{0.56} \times K \times LS \times C \times P$$

$$\begin{aligned}
&\text{Sed} = \text{Sediment Yield per Event (metric tons)}\\
&\text{O}_{\text{surf}} = \text{Surface Runoff Volume } \left(\text{m}^3\right)\\
&\text{q}_{\text{peak}} = \text{Peak Runoff Rate } \left(\text{m}^3/\text{s}\right)\\
&\text{K} = \text{Soil Erodibility Factor}\\
&\text{LS} = \text{Topographic Factor}\\
&\text{C} = \text{Cover and Management Factor}\\
&\text{P} = \text{Support Practice Factor}
\end{aligned}$$

12.1.5.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the erodibility factor, topographic factor, cover factor, and practice factor.

A tutorial using the MUSLE in an event simulation can be found here: TBD.

12.1.5.1.1 A Note on Parameter Estimation

The **Threshold** parameter was introduced as an independent variable to segment storm events for continuous simulation. It establishes the lower boundary for direct runoff flow rate, marking the commencement of a sediment flow event when the direct runoff exceeds this threshold. Conversely, the event concludes when the direct runoff drops below the specified threshold. This parameter assumes particular significance in the calibration process, especially for continuous simulations.

12.1.6 Multi-Sequence Debris Prediction Method (MSDPM)

The Multi-Sequence Debris Prediction Method (MSDPM), as outlined by Pak and Lee in 2008, emerged from an extensive analysis of debris clean-out data spanning from 1938 to 2002. This dataset was meticulously collected from 80 debris basins located across Southern California. MSDPM has proven effective for the continuous simulation of debris yield within relatively modest watersheds, covering areas ranging from 0.1 mi^2 to 3.0 mi^2.

The underpinnings of MSDPM are grounded in three fundamental physical processes. First, it considers the critical conditions necessary for sediment entrainment. Second, it factors in the transport capacity required to convey sediment toward the focal point of concentration, which typically corresponds to a debris basin. Finally, MSDPM integrates the antecedent precipitation state in conjunction with subsequent rainfall events as critical variables in its predictive model.

It is important to note that MSDPM performs most effectively in arid or semi-arid regions, particularly within the specific geographic context of Southern California, where it was initially developed and refined.

$$\sum_{i=1}^{N}\left(D_y\right)_i = 0.25\sum_{i=1}^{N}\left(1+\frac{|(I_m)_i - I_C|}{((I_m)_i - I_C)}\right)\left(1+\frac{|(P)_i - P_c|}{((P)_i - P_C)}\right)(I_m)_i^{0.541} S^{0.134} A^{1.023} e^{0.290FF}$$

$D_y =$ Sediment Yield per Event $\left(\mathrm{m}^3\right)$
$I_m =$ Maximum 1-hour Rainfall Intensity per Event (mm/h)
$I_c =$ Threshold Maximum 1-hour Rainfall Intensity (TMRI)(mm/h)
$P =$ Total Rainfall Amount per Event (mm)
$P_c =$ Total Minimum Rainfall Amount ($TMRA$)(mm)
$S =$ Relief Ratio (m/km)

$$\mathrm{S} = \frac{h_2 - h_1}{L}$$

$h_2 =$ Highest Elevation in the Watershed (m)
$h_1 =$ Lowest Elevation in the Watershed (m)
L= Maximum Stream Length (km)
$A =$ Drainage Area (ha)
$FF =$ Non-Dimensional Fire Factor
$P \neq P_c$ and $I_m \neq I_c$

This Fire Factor (FF) equation has been meticulously designed to cater specifically to the continuous, long-term simulation, allowing for the dynamic incorporation of the recovery process following a fire event, in tandem with the influence of rainfall events over time. In the course of this research, a comprehensive fire factor (FF) was ingeniously formulated by taking into account several key parameters. These parameters include the extent of watershed burned, the temporal elapsed since the fire incident, and the count of antecedent precipitation events surpassing a predefined threshold value since the fire occurrence.

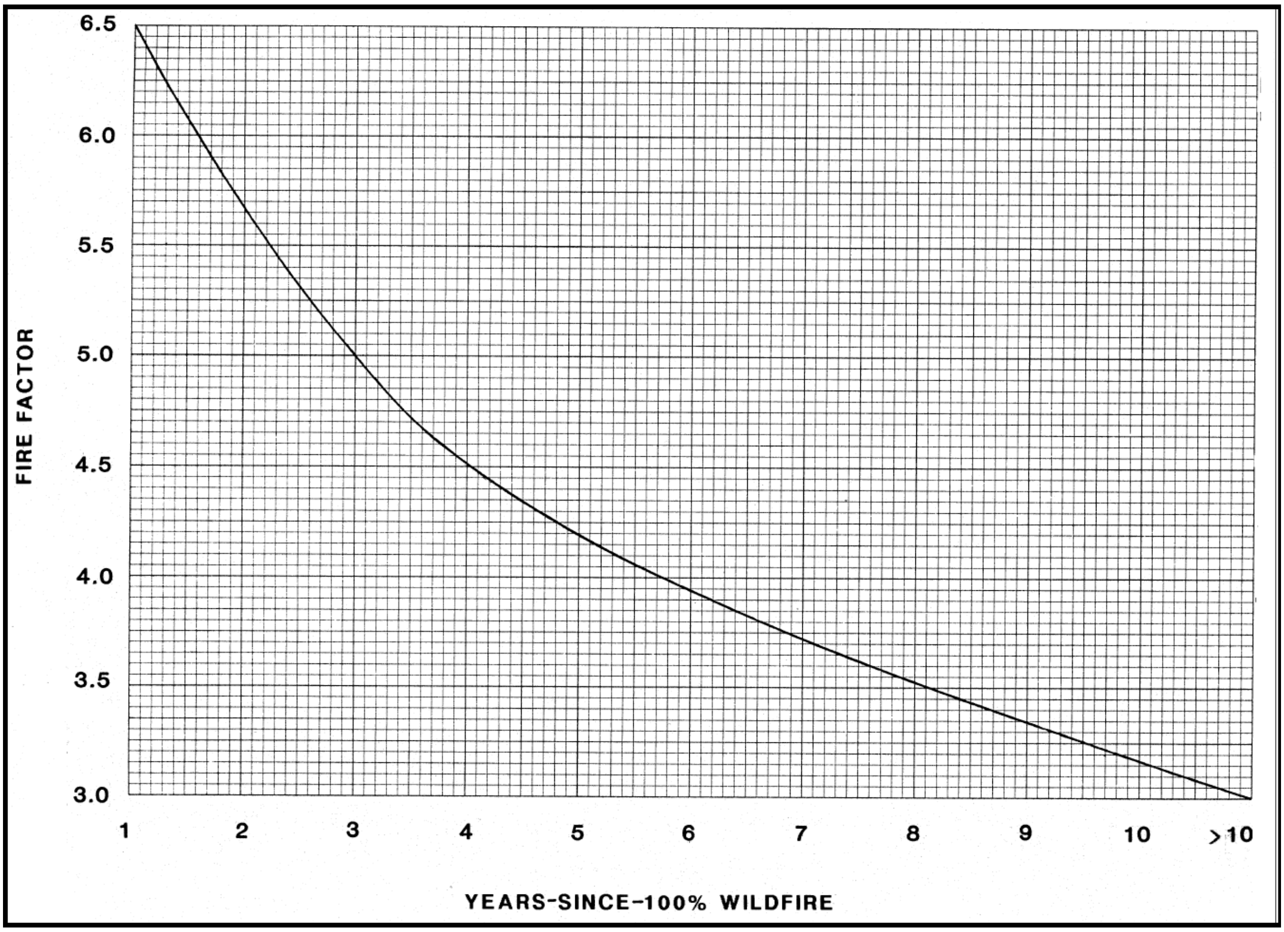
Furthermore, it's important to note that the Fire Factor (FF) equation is applicable in conjunction with LA Debris Equation 1 for continuous simulation. Nonetheless, an essential consideration arises when extending its usage to LA Debris Equations 2-5, particularly when dealing with areas exceeding 3.0 mi². This is due to the fact that the original equation was established using the Fire Factor Curve within the range of 0.1 mi² to 3.0 mi². Therefore, for reliable application beyond this range, the equation necessitates calibration with empirically measured data.

$$FF = \text{ Fire Factor, } 3.0 \leq FF \leq 6.5(\text{ dimensionless })$$

$$FF = 6.5 \times \left(B_p \times B_y^{-0.29} + \left(1 - B_p\right) \times \left(20 - B_y\right)^{-0.29}\right) \times \left(2 - e^{\left(A_p/200\right)}\right)$$

$A_p =$ Number of antecedent ef fective precipitation events that have enough energy to generate sediment yield
$B_p =$% of, $\left(0 \leq B_p \leq 1\right)$
$B_y =$ Number of years since burn, $\left(1 \leq B_y \leq 10yr\right)$

Additionally, should the need arise, the below Fire Factor Curve (watersheds ranging from 0.1 mi² to 3 mi²) can be effectively employed in conjunction with MSDPM for event-based simulations.



12.1.6.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the maximum 1-hour precipitation [inches or millimeters], Threshold Maximum 1-hour Rainfall Intensity (TMRI) [inched/hour or millimeters/hour], Total Rainfall Amount per Event [inches or millimeters], Total Minimum Rainfall Amount (TMRA) [inches or millimeters], relief ratio [ft/mi or m/km], and non-dimensional fire factor.

A tutorial using the Multi-Sequence Debris Prediction Method (MSDPM) in an event simulation can be found here: Applying Debris Yield Methods in HEC-HMS[90].

12.1.6.1.1 A Note on Parameter Estimation

HEC-HMS initially assigns a default value of 1.0 to the Adjustment-Transposition (A-T) factor. However, it's essential to fine-tune and verify this value by taking into account the disparities in geomorphological

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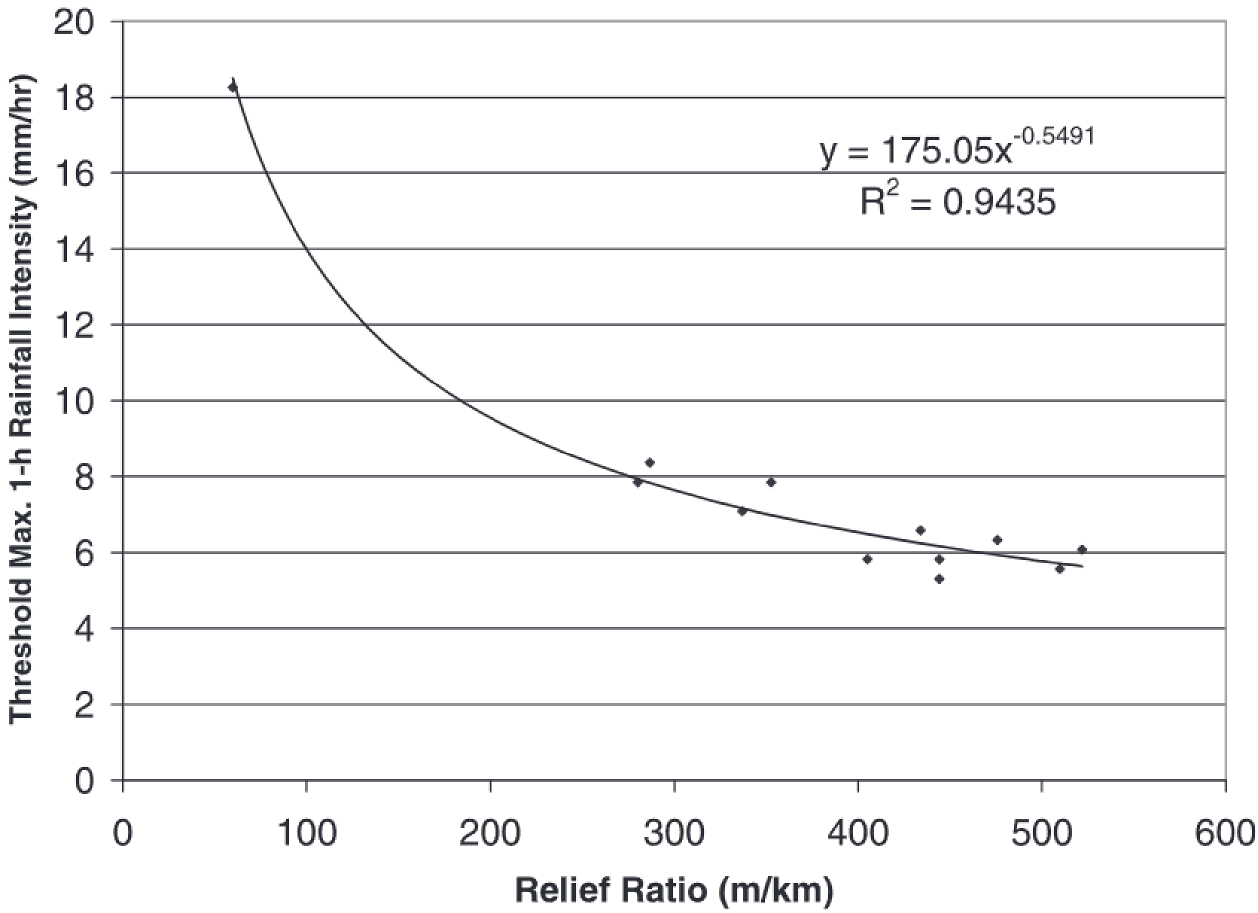
characteristics between the specific watershed under consideration and the original watershed (San Gabriel Mountains, CA) from which the regression equation was originally derived.

The **Flow Rate Threshold** parameter was introduced as an independent variable to segment storm events for continuous simulation. It establishes the lower boundary for direct runoff flow rate, marking the commencement of a debris flow event when the direct runoff exceeds this threshold. Conversely, the event concludes when the direct runoff drops below the specified threshold. This parameter assumes particular significance in the calibration process, especially for continuous simulations.

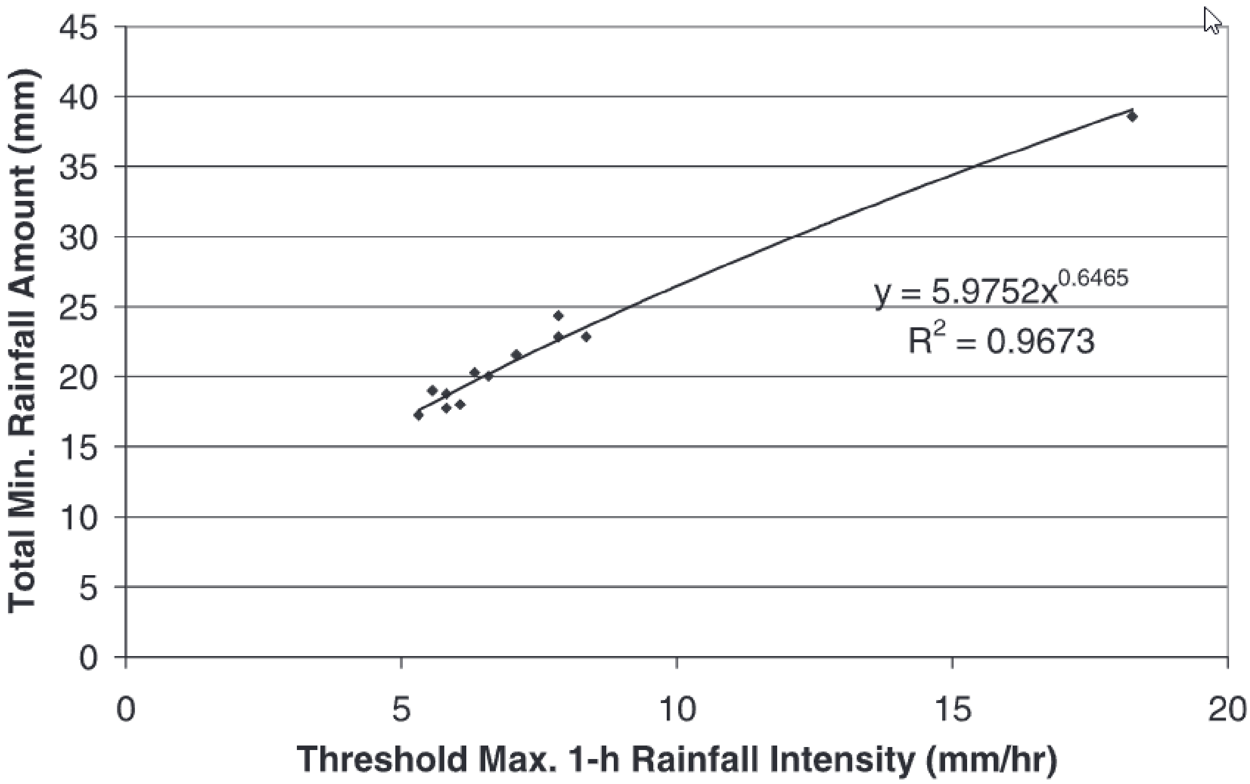
Threshold Maximum 1-hour Rainfall Intensity (TMRI): It's worth noting that not every instance of rainfall can trigger sediment entrainment, as a certain minimum energy level is required to mobilize sediment particles effectively. Consequently, rainfall events underwent a rigorous screening process to isolate those instances where the actual rainfall exceeded the critical threshold necessary for sediment entrainment.

The determination of the TMRI, specifically for the initiation of sediment entrainment, was guided by an initial estimate rooted in the relationship between TMRI and relief ratio, as illustrated in the figure below. The initial TMRI value for each watershed can be obtained from the TMRI-relief ratio graph below. Fine-tuning and calibration occur during the calibration process using outlet flow gage data for increased accuracy and precision.



Total Minimum Rainfall Amount (TMRA): TMRA is directly linked to the sediment transport capacity required to channel sediment toward the concentration point. However, it's crucial to recognize that not all rainfall events possess the capacity to facilitate substantial sediment transport. Once sediment entrainment occurs, an additional level of energy is necessary to transport the sediment effectively to the concentration point. Consequently, another round of screening was conducted to identify rainfall events capable of providing this essential energy.

The critical total rainfall amount, represented as TMRA, was established individually for each watershed by analyzing the interplay between TMRA and TMRI, as visually depicted in the figure below. This pivotal threshold signifies the minimal rainfall accumulation required to facilitate substantial sediment transport within the watershed. The initial TMRA value for each watershed can be extracted directly from the TMRA and TMRI graph below. For further refinement and calibration, precision and accuracy are enhanced through the calibration process, which incorporates outlet flow gage data.



12.1.7 USGS Emergency Assessment Debris Model

The USGS emergency assessment debris model, established by Gartner, Cannon, and Santi in 2014, draws upon a specific subset of data encompassing 92 sediment deposition volumes resulting from debris flows occurring in the two years following a fire event.

This USGS emergency assessment debris model has demonstrated its efficacy in continuously simulating debris yield within a two-year window for relatively modest watersheds, covering areas smaller than 30 km².

Among a multitude of models derived from multiple linear regression analyses, the most robust one provides forecasts for sediment volumes originating from debris flows during the specified post-fire period. These predictions are encapsulated in the following equation:

$$\ln V = 4.22 + 0.39 \times \sqrt{i15} + 0.36 \times \ln \ \mathrm{Bmh} \ + 0.13 \times \sqrt{R}$$

$$\begin{aligned} V &= \text{ Volume of Sediment } \left(\mathrm{m}^3\right) \\ i15 &= \text{ Peak 15-minute Rainfall Intensity (mm/h)} \\ \mathrm{Bmh} &= \text{ Watershed Burned at moderate and high severity } \left(\mathrm{km}^2\right) \\ R &= \text{ Relief (m)} = \mathrm{h}_2 - \mathrm{h}_1 \\ h_2 &= \text{ Highest Elevation in the Watershed (m)} \\ h_1 &= \text{ Lowest Elevation in the Watershed (m)} \end{aligned}$$

12.1.7.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the maximum 15-minute precipitation [inches or millimeters], watershed relief [feet or meters], and watershed area burned at moderate and high severity [mi² or km²] .

A tutorial using the USGS emergency assessment Debris Method in an event simulation can be found here: Applying Debris Yield Methods in HEC-HMS[91].

12.1.7.1.1 A Note on Parameter Estimation

The **Flow Rate Threshold** parameter was introduced as an independent variable to segment storm events for continuous simulation. It establishes the lower boundary for direct runoff flow rate, marking the commencement of a debris flow event when the direct runoff exceeds this threshold. Conversely, the event concludes when the direct runoff drops below the specified threshold. This parameter assumes particular significance in the calibration process, especially for continuous simulations.

12.1.8 USGS Long-Term Debris Model

The USGS long-term debris model, established by Gartner, Cannon, and Santi in 2014, draws upon the complete database of 344 volumes of sediment deposited by debris flows and sediment-laden floods with no time limit since the most recent fire.

The USGS long-term debris model has showcased its effectiveness in continuously simulating debris yield without temporal restrictions, covering areas smaller than 30 km².

The best model developed from the multiple linear regression analyses of this database predicts volumes of sediment based on the following equation:

91 https://www.hec.usace.army.mil/confluence/display/HMSGUIDES/Applying+Debris+Yield+Methods+in+HEC-HMS

$$\ln V = 6.07 + 0.71 \times \ln i60 + 0.22 \times \ln Bt - 0.24 \times \ln T + 0.49 \times \ln A + 0.03 \times \sqrt{R}$$

$$\begin{aligned}
V &= \text{ Volume of Sediment } \left(\mathrm{m}^3\right) \\
i60 &= \text{ Peak 60-minute Rainfall Intensity (mm/h)} \\
\mathrm{Bt} &= \text{ Total Area of Watershed Burned by Most Recent Fire } \left(\mathrm{km}^2\right) \\
T &= \text{ Time since The Most Recent Fire (year)} \\
A &= \text{ Watershed Area } \left(\mathrm{km}^2\right) \\
R &= \text{ Relief (m)} = \mathrm{h}_2 - \mathrm{h}_1 \\
h_2 &= \text{ Highest Elevation in the Watershed (m)} \\
h_1 &= \text{ Lowest Elevation in the Watershed (m)}
\end{aligned}$$

12.1.8.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the maximum 60-minute precipitation [inches or millimeters], watershed relief [feet or meters], and watershed area burned by the most recent wildfire [mi² or km²] .

A tutorial using the USGS Long-Term Debris Method in an event simulation can be found here: Applying Debris Yield Methods in HEC-HMS[92].

12.1.8.1.1 A Note on Parameter Estimation

The **Flow Rate Threshold** parameter was introduced as an independent variable to segment storm events for continuous simulation. It establishes the lower boundary for direct runoff flow rate, marking the commencement of a debris flow event when the direct runoff exceeds this threshold. Conversely, the event concludes when the direct runoff drops below the specified threshold. This parameter assumes particular significance in the calibration process, especially for continuous simulations.

12.2 Sediment Transport Methods

HEC-HMS employs a dual-pronged approach to manage sediment transport within the reach element. First, it calculates the transport capacity for each grain size, determining the sediment available for subsequent routing within each sub-reach. This computation factors in the transport potential and sediment sources on the reach bottom. The subsequent step revolves around computing sediment routing for the available sediment between sub-reaches, employing a range of sediment routing methods.

92 https://www.hec.usace.army.mil/confluence/display/HMSGUIDES/Applying+Debris+Yield+Methods+in+HEC-HMS

12.2.1 Sediment Transport Potential Methods

The Sediment Transport Potential[93] specifies which method to use to calculate the stream flow sediment carrying capacity for non-cohesive sediments. Different methods have been proposed for calculating the transport potential. Each method has been developed for a particular sediment grain-size distribution and environmental condition. The selected transport potential method will be used at all reaches within the Basin Model. The available choices are shown in the table below.

Method	**Type**	**Method**	**Reference**
Ackers and White[94]	NC	SP	Ackers and White, 1973
Engelund-Hansen[95]	NC	SP	Engelund and Hansen, 1967
Laursen-Copeland[96]	NC	ES	Laursen, 1958; Copeland and Thomas, 1989
Meyer-Peter Müller[97]	NC	ES	Meyer-Peter and Müller, 1948
Toffaleti[98]	NC	RE	Toffaleti, 1968
Wilcock-Crowe[99]	NC	ES	Wilcock and Crowe, 2003
Yang[100]	NC	SP	Yang, 1984
Krone and Parthenaides Methods[101]	CO	—	Krone, 1962; Parthenaides, 1962
Sediment Delivery Ratio (only for HEC-HMS)	NC/CO	—	Pak and Lee (2012)

Notes: non-cohesive (NC) or cohesive (CO). Method is excess shear (ES), stream power (SP), or regression (RE).

93 https://www.hec.usace.army.mil/confluence/display/RASSED1D/Sediment+Transport+Potential
94 https://www.hec.usace.army.mil/confluence/display/RAS/Ackers+and+White
95 https://www.hec.usace.army.mil/confluence/display/RAS/Engelund-Hansen
96 https://www.hec.usace.army.mil/confluence/display/RAS/Laursen-Copeland
97 https://www.hec.usace.army.mil/confluence/pages/viewpage.action?pageId=30805747
98 https://www.hec.usace.army.mil/confluence/display/RAS/Toffaleti
99 https://www.hec.usace.army.mil/confluence/display/RAS/Wilcock-Crowe
100 https://www.hec.usace.army.mil/confluence/display/RAS/Yang?src=contextnavpagetreemode
101 https://www.hec.usace.army.mil/confluence/display/RAS/Krone+and+Parthenaides+Methods

A cohesive transport potential method (Krone Parthenaides) can also be selected. When selected, transport of cohesive sediment is computed in addition to the non-cohesive sediment. More information can be founded in HEC-HMS manual (Cohesive Transport[102]).
Transport potential functions for calculating the amount of sediment that can be carried by the stream flow.

For a more comprehensive understanding, including information on algorithms that translate hydrodynamics into transport, please refer to the "HEC-RAS Sediment Manual" by Gibson and Sánchez (2020) (Sediment Manual[103]). This reference is relevant because HEC-HMS shares the same Sediment Transport engine as HEC-RAS.

12.2.1.1 Sediment Delivery Ratio (SDR)

The integration of the SDR Transport Potential method is geared towards managing erosion and deposition functions by relying on fundamental hydrological principles. Its specialized application is tailored specifically for simulating debris flow within the HEC-HMS framework and it's important to note that this particular SDR option is not incorporated within HEC-RAS.

The SDR method itself serves as a valuable technique in hydrology and sediment transport studies. It functions by estimating the proportion of sediment generated in an upstream area, typically a watershed or catchment, that ultimately reaches a downstream location, such as a river or reservoir. This estimation sheds light on the effectiveness of sediment retention and trapping mechanisms operating within the watershed.

The SDR method is conventionally expressed as a fraction or percentage. A high SDR value signifies that a substantial portion of the generated sediment successfully reaches the downstream location, while a low SDR indicates effective sediment trapping, resulting in reduced sediment delivery downstream.

It's important to note that the SDR method necessitates the consideration of ratios for each grain class, such as Clay, Silt, Sand, and Gravel when the Grade Scale is selected as "Clay Silt Sand Gravel." When the Grade Scale is set to "AGU 20," it includes Clay, Silt, Sand, Gravel, Cobble, and Boulder. In this context, a Ratio greater than 1 signifies an erosional scenario, a Ratio less than 1 signifies a depositional scenario, and a Ratio equal to 1 represents an equilibrium situation.

$$O_{sed} = SDR \times I_{sed}$$

$$O_{sed} = \text{ Sediment Outflow}$$
$$SDR = \text{ Sediment Delivery Ratio}$$
$$I_{sed} = \text{ Sediment Inflow}$$

12.2.2 Sediment Routing Methods

HEC-HMS necessitates supplementary sediment transport mechanisms within its unique reach configuration. In contrast to HEC-RAS, which relies on intricate cross-section dividers, HEC-HMS employs inner rough sub-reaches within the reach element. Consequently, these sediment routing options are pivotal in improving sediment transport between these sub-reaches within the reach element.

There are a total of five diverse sediment routing methods at your disposal within HEC-HMS. These methods encompass:

- Fisher's Dispersion
- Linear Reservoir

102 https://www.hec.usace.army.mil/confluence/display/RAS/Cohesive+Transport
103 https://www.hec.usace.army.mil/confluence/display/RAS/Sediment+Manual

- Muskingum
- Uniform Equilibrium
- Volume Ratio

The following sections detail their unique concepts and uses.

12.2.3 Fisher's Dispersion

The Fisher's Dispersion Method is based on an analysis of advection and diffusion of sediment within a reach (Fisher *et al.*, 1979). This is the most detailed of the sediment routing methods and requires more data than the other available methods. Advection and diffusion, represented as Travel and Dispersion parameters, need to be specified for each grain size class. This permits large-grained sediments to move slower than fine-grained sediments. For each time interval, sediment from the upstream elements are added to the sediment already in the reach. After erosion or deposition is calculated, the remaining available sediment is translated in the reach by a Travel Time and attenuated through a diffusion process. The advection and diffusion of sediment are linked to the velocity of water in the reach which is calculated during the flow routing.

$$C(X_2,t) = \int_{-\infty}^{\infty} C(X_1,T) \frac{\exp\left[\frac{-\left\{\overline{u}(\overline{t_2}-\overline{t_1}-t+T)\right\}^2}{4K(\overline{t_2}-\overline{t_1})}\right]}{\sqrt{4\pi K(\overline{t_2}-\overline{t_1})}} \overline{u}dT$$

Note for Jay: Add Parameter descriptions later

12.2.3.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the dispersion coefficient [ft²/s or m²/s], travel time [hour].

12.2.3.1.1 A Note on Parameter Estimation

Dispersion Coefficient must be specified for each grain class (clay, silt, sand, gravel). The Dispersion coefficient indicates the diffusion of the particles during transit through the reach and is dependent on the channel geometry. The dispersion coefficient can vary over several orders of magnitude and often must be adjusted during calibration. Some guidance is available for estimating the dispersion coefficient from Kashefipour and Falconer (2002). The Travel Time must also be specified for each grain class and is often close to the travel time for water in the reach. When the AGU 20 grain size classification is used, the same dispersion and retention values are used for all subclasses of grain classes.

12.2.4 Linear Reservoir

The Linear Reservoir Sediment Routing Method employs a straightforward linear reservoir concept to guide the movement of sediment of various grain sizes within a given reach. During each time interval, the sediment load is determined based on the incoming sediment from upstream and local erosion or deposition processes. Importantly, sediment within each grain size category is individually routed through a linear reservoir, independently of the hydrological routing of water flow. This design allows sediment particles of different sizes to travel at distinct rates through the reach.

For precise parameter assignment, it is advisable to calibrate the model using observed data. This ensures that the parameter values accurately reflect the real-world conditions. The Component Editor, which is illustrated in the accompanying figure, provides a tool for making these adjustments.

The "Retention" parameter, analogous to the storage coefficient, plays a critical role in sediment routing within the reach. Sediment routing occurs separately for clay, silt, sand, and gravel. When employing the AGU 20 grain size classification, the same retention value is applied to all subclasses within each class. This retention value is essentially a representation of the median time it takes for sediment particles of a specific size class to traverse the reach. While this value can vary for each sediment size class, it often approximates the travel time for water within the reach.

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

$$S_t = RO_t$$

$$\begin{aligned} &dS/dt = \text{Change in Storage at time t} \\ &I_t = \text{Average Inflow to Storage at time t} \\ &O_t = \text{Outflow from Storage at time t} \\ &R = \text{Linear Reservoir Coefficient} \end{aligned}$$

12.2.4.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the retention parameters for each grain size.

12.2.4.1.1 A Note on Parameter Estimation

Estimating the retention parameters for each grain size typically involves a calibration process based on observed hydrological data.

12.2.5 Muskingum

The Muskingum Method employs a straightforward mass conservation approach to manage sediment and debris routing within a stream reach. During each time interval, the method calculates the available sediment, considering both upstream sediment inputs and local erosion or deposition within the reach. This available sediment, segmented by grain size, is then routed using Muskingum routing parameters: the Attenuation Coefficient (X) and Travel Time (K). These parameters facilitate the movement of sediment with varying grain sizes at different speeds and approximate its attenuation as it traverses the reach.

The Muskingum sediment routing method shares similarities with the Muskingum flow routing method (Muskingum Model (see page 207)) in that both employ a mass conservation approach for routing. In the context of sediment routing, this method effectively controls time lag with the travel time parameter (K) and regulates attenuation using the dimensionless weight factor (X, typically ranging from 0 to 0.5).

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

HEC-HMS solves this equation recursively to compute O_t given inflow (I_t and $I_{t\text{-}1}$), an initial condition ($O_{t=0}$), K, and X.

12.2.5.1 Required Parameters

Parameters that are required to utilize this method within HEC-HMS include the attenuation coefficients and travel times for each grain size.

A tutorial using the Muskingum sediment routing method can be found here: Task 4: Debris Flow Modeling using Debris Channel Routing Method[104]

12.2.5.1.1 A Note on Parameter Estimation

Estimating the Muskingum routing method's K and X parameters typically involves a calibration process based on observed hydrological data.

12.2.6 Uniform Equilibrium

The Uniform Equilibrium Method operates on the assumption that sediment is instantaneously transported through the reach without any temporal lag. It represents the simplest approach because it does not account for any delay in the sediment's passage through the reach.

Here's how this method generally works:

1. Sediment Inflow: Sediment enters the reach from upstream elements, such as tributaries or other contributing sources.
2. Transport Capacity Assessment: The method calculates the transport capacity for each grain size to determine whether the stream is experiencing sediment deposition or erosion.
3. Sediment Constraints: It also considers any constraints on sediment deposition and erosion within the reach.
4. Immediate Routing: The remaining sediment, after considering all factors, is routed instantly during the same time interval, regardless of the flow velocity.

104 https://www.hec.usace.army.mil/confluence/display/HMSGUIDES/Task+4%3A+Debris+Flow+Modeling+using+Debris+Channel+Routing+Method

In essence, the Uniform Equilibrium Method simplifies sediment transport calculations by assuming that sediment moves through the reach without delay, making it a straightforward approach within sediment routing modeling.

12.2.7 Volume Ratio

The Volume Ratio sediment routing method is a technique used in hydrology and hydraulic engineering to estimate the proportion of sediment transported from one location to another within a river or stream network. Specifically, it calculates the sediment volume ratio between two adjacent reaches or sub-reaches. The Volume Ratio Method directly pairs the sediment transport to the streamflow. For each time interval, sediment from the upstream elements is added to the sediment already in the reach. Deposition or erosion is calculated for each grain size to determine the available sediment for routing. The proportion of available sediment that leaves the reach in each time interval is assumed equal to the proportion of stream flow that leaves the reach during that same interval. This means that the all grain sizes are transported through the reach at the same rate, even though erosion and deposition are determined separately for each grain size.

$$\text{Sed}_{\text{out}} = \frac{\text{Volume}_{\text{out}}}{\text{Volume}_{\text{channel}}} \text{Sed}_{\text{channel}}$$

$$\text{Sed}_{\text{out}} = \text{Sediment Outflow}$$
$$\text{Volume}_{\text{out}} = \text{Water Volume Outflow}$$
$$\text{Volume}_{\text{channel}} = \text{Water Volume in Channel}$$
$$\text{Sed}_{\text{channel}} = \text{Sediment in Channel}$$

12.3 Reservoir Sediment Methods

A reservoir offers an ideal setting for the gradual removal of suspended sediment from water. In this tranquil water body, where there is minimal horizontal water movement, sediment particles gradually lose their buoyancy and settle to the reservoir's bed. The settling process, however, is not instantaneous due to the minute frictional interactions between each sediment grain and the surrounding water molecules. This friction generates turbulence, which retards the descent of individual sediment particles. The settling velocity of a particular particle depends primarily on its size, with density and shape playing secondary roles. Larger grains descend rapidly, while smaller ones settle more slowly. Extremely fine-grained sediments, such as silt and clay, may require several years to fully settle from the water column.

The duration of sediment residence in the reservoir significantly impacts the proportion of incoming sediment that ultimately settles at the bottom. Nevertheless, the tiniest clay particles may never settle due to electrical charges inherent in the mineral composition of these particles and the electrical charges within water molecules, arising from hydrogen bonding.

The influx of sediment into a reservoir, leading to sediment settling, reduces the available storage capacity of the reservoir. Over time, the cumulative loss of storage space due to sediment accumulation can become substantial. If you choose the Reservoir Capacity Method, the program will calculate the sediment balance within the reservoir across time intervals. It will update the Elevation-Area and Elevation-Storage curves for each time step using efficient trapping mechanisms based on the sediment and debris inflow from the upstream watersheds.

12.3.1 Brune Sediment Trap

The Brune Trap Efficiency method is a hydrological modeling approach used to estimate the efficiency with which a reservoir or basin traps sediment or debris flowing into it from upstream areas. It is named after its developer, Albert P. Brune.

In the context of sediment transport and reservoir siltation modeling, the Brune Trap Efficiency method assesses how effectively a reservoir or basin retains sediments of varying sizes. This method considers factors such as the size distribution of incoming sediment, the hydraulic conditions within the reservoir or basin, and the sediment deposition patterns.

The Brune Trap Efficiency method is a useful tool for assessing the sedimentation rates in reservoirs, which is crucial for managing water resources, maintaining reservoir capacity, and planning for sediment removal activities. It helps in understanding how efficiently a reservoir traps sediment, which can have implications for water quality, flood control, and hydropower generation.

An additional trap efficiency method is needed to account for reservoir volume reduction based on the sediment siltation volume. One candidate method is the Brune's trap efficiency method (Brune, 1953) utilized by Kansas City District. By adding Brune's trap efficiency method in HEC-HMS, USACE local district offices can easily replace their manual excel spreadsheet to estimate the long-term reservoir siltation with HEC-HMS. Brune (1953) presents a family of curves (Figure 1) that estimates the trapping efficiency based on the ratio of reservoir capacity to mean annual discharge.

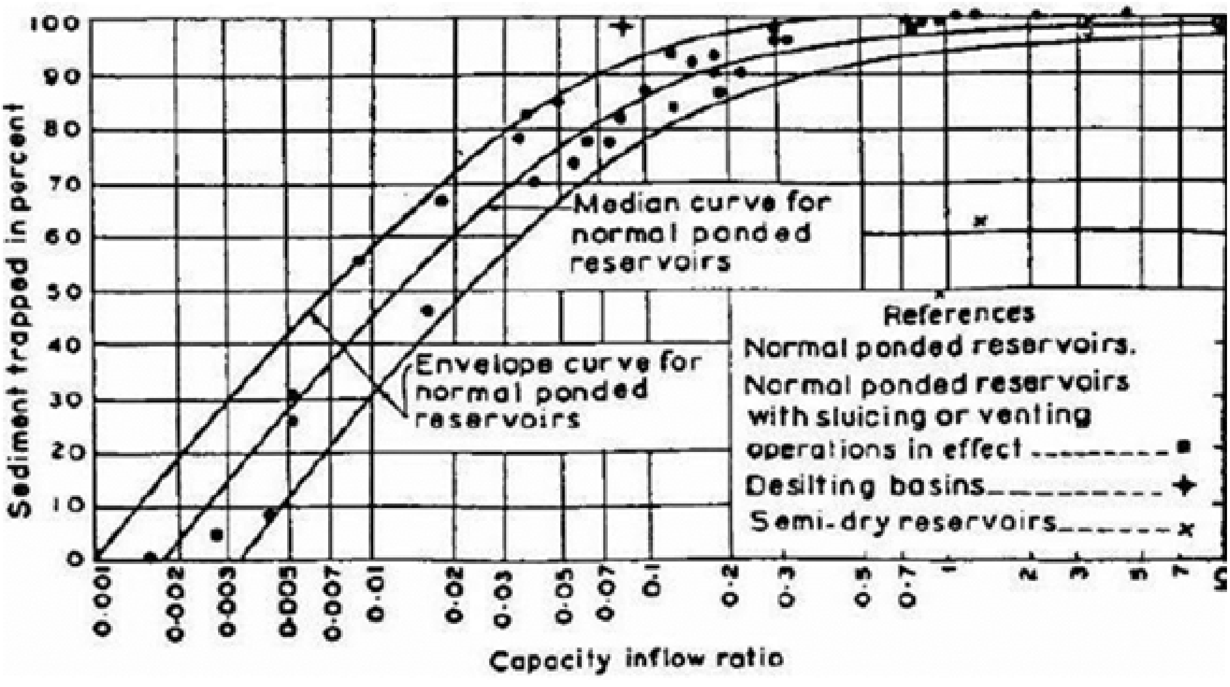


Figure 1. Trap efficiency as related to capacity-inflow ratio, type of reservoir, and method of operation (Brune, 1953)

The following equations can be used to estimate these curves:

$$TE = a[1 - 2e^{-bV_*^{0.35}}]$$

$$V_* = \frac{V_{res}}{V_{inflow}}$$

$$V_* = Capacity/Annual\ Inflow\ Ratio$$
$$V_{res} = Reservoir\ Capacity$$
$V_{inflow} = Annual\ Inflow$ (A user specified initial value is required)

a = Constant (recommended range: Minimum :95, Medium: 97, Maximum: 100)

b = Constant (recommended range: Minimum: 5.37, Medium: 6.42, Maximum: 7.71)

12.3.2 Chen Sediment Trap

The trap efficiency (TE) of a reservoir is a measure of how effectively it retains sediment, expressed as the ratio of the sediment load retained in the reservoir to the total sediment load entering the reservoir. To estimate trap efficiency, one can compare the settling velocity of sediment to a critical settling velocity, as suggested by Camp in 1945. The settling velocity is determined based on the chosen method for the Basin Model. The critical settling velocity is calculated as the discharge rate from the reservoir divided by its surface area. These calculations are performed separately for different grain size classes or subclasses.

Camp (1945) conducted a study on sedimentation in an ideal rectangular continuous flow basin or settling tank, as depicted in Figure 1. This model assumes conditions of quiescent and steady flow, complete mixing of water and sediment, and no resuspension. When sediment flows into the tank, discrete particles settle with a settling velocity (vs), which varies with particle size. In an ideal setting, one can define a critical settling velocity (vc) for the pond. Particles with vs equal to vc settle in the pond just before they exit. This critical settling velocity depends on the water depth (d) and the time required for water to flow through the pond (T):

$$v_c = \frac{d}{T} = \frac{d}{1/v} = \frac{dv}{1} = \frac{dvb}{1b} = \frac{Q}{A} = \text{overflow rate}$$

where:
l and *b* are the length and width of the settling zone, respectively;
v is the water velocity through the pond;
A is the surface area of the pond; and
Q is the in- or outflowing discharge.

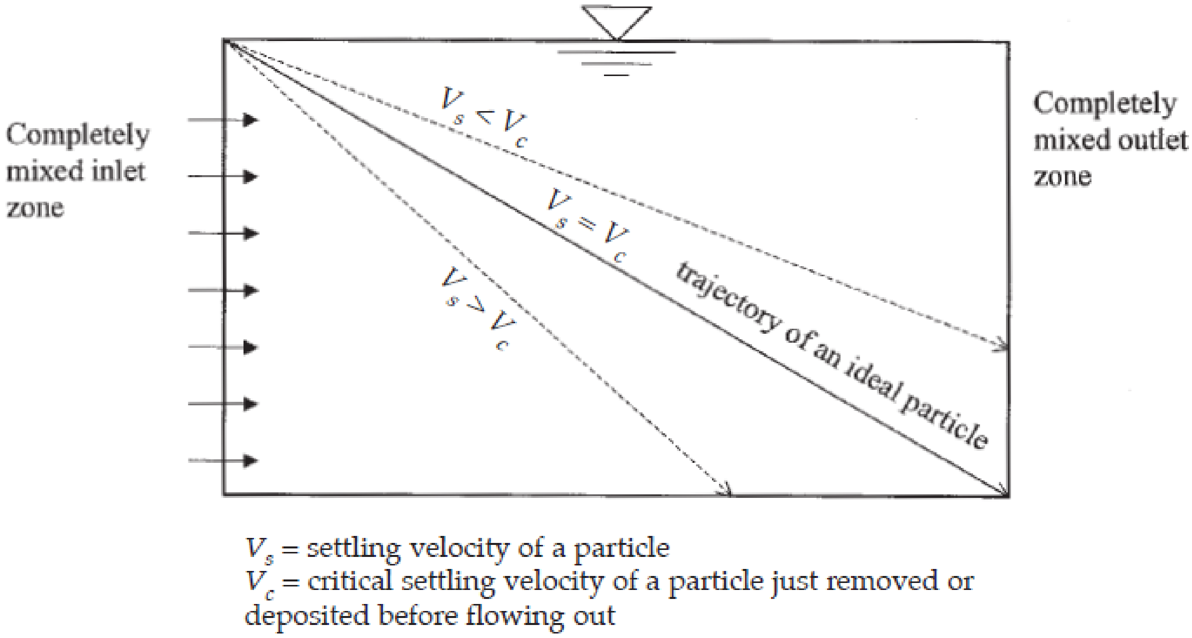


Figure 1. Settling conditions in an ideal rectangular settling basin (Camp, T.R. 1945)

The critical settling velocity is therefore equal to the overflow rate of the pond. For an ideal rectangular pond, the fraction of particles trapped with vs less than v_c is given by the TE:

$$TE = 100\frac{v_s}{v_c} = 100\frac{A}{Q}v_s$$

Chen (1975) modified above equation for turbulent flow conditions:

$$TE = 100\left[1 - e^{-\frac{v_s}{v_c}}\right]$$

HEC-HMS employs the equation corresponding to Chen's TE method for its calculations.

12.3.3 Reservoir Volume Reduction

Reservoir siltation plays a pivotal role in determining the effective operation and maintenance of reservoirs, serving as a fundamental gauge. The sedimentation of reservoirs carries significant implications for various aspects, including hydropower generation, flood control, debris flow management, water supply, irrigation, and recreational activities. This impact arises primarily from the gradual loss of reservoir storage capacity. Correspondingly, the accumulation of sediment in small debris basins has consequences for the control of debris yield and flow, owing to the diminishing storage capacity of these basins.

The primary objective of this methodology is to devise approaches for modeling the reduction in volume within reservoirs and debris basins by employing efficient trapping methods and updating Elevation-Storage and Elevation-Area relation curves, which are contingent on the inflow of sediment and debris from upstream watersheds.

The processes for reducing reservoir volume, as illustrated in Figure 1 along with their respective trap efficiency methods, will significantly elevate the precision of siltation simulations for reservoir components integrated into the HEC-HMS framework.

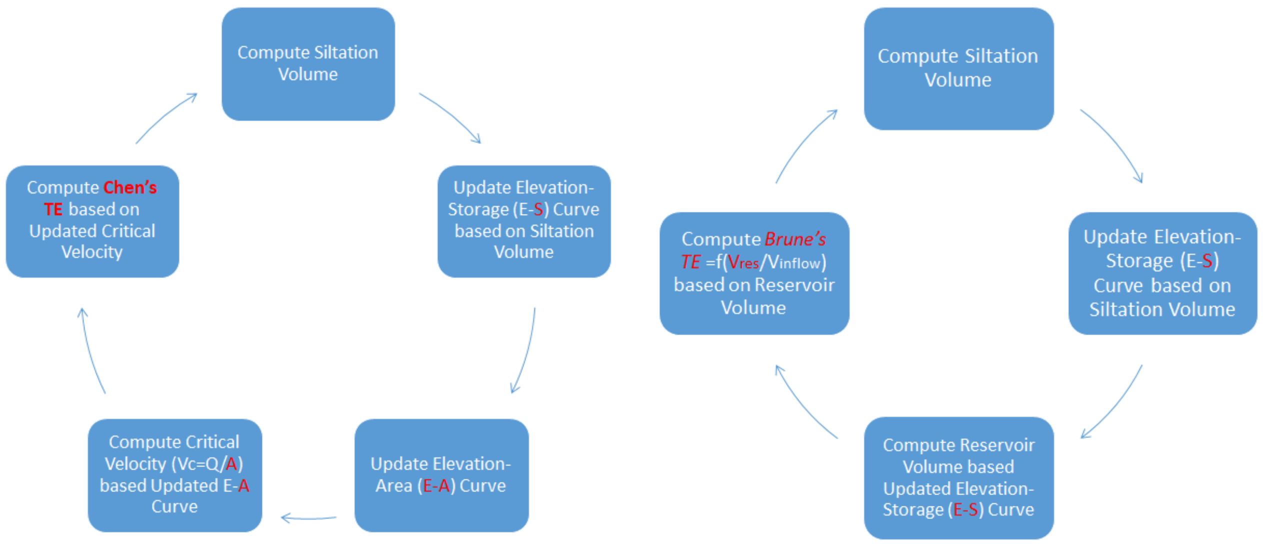


Figure 1. Reservoir Volume Reduction Process: (a) Chen's Trap Efficiency Method (b) Brune's Trap Efficiency Method

When users choose the "Reservoir Capacity Method" within the HEC-HMS user interface, they are offered a selection between two deposition shape options: "V-Shape" and "Elongated Taper." This choice of deposition shape for each grain size initiates modifications to the Elevation-Storage and Elevation-Area relation curves, and these alterations are visually depicted in Figure 2.

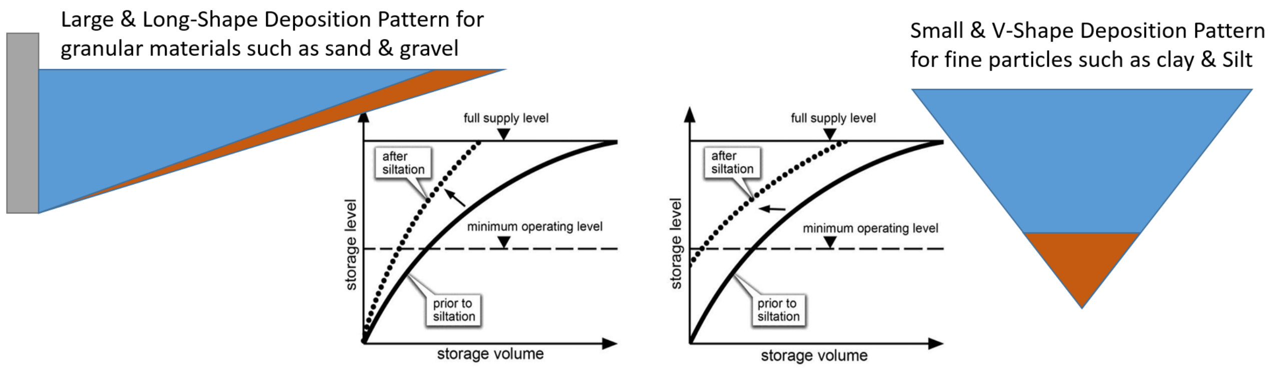
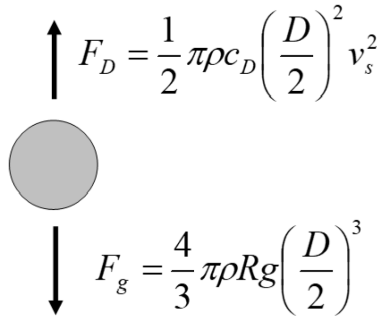


Figure 2. Elevation-Storage Curve after Siltation (Left Figure: Delta Deposition (coarse material (Sand & Gravel)/large and long reservoir) Right Figure: Deposited Muddy Lake Deposition at Dead Storage Zone (fine material (Clay & Silt / Small and V-shaped Reservoir))

12.4 Fall Velocity and Settling

Fall Velocity[105] is also used in several of the Sediment Settling Methods for the reservoir element. All reaches and reservoirs that require the calculation of a fall velocity will use the method selected in the sediment properties. The available methods for calculating fall velocity include Report 12 (Interagency Committee, 1957), Rubey (1933), Toffaleti (1968), and Van Rijn (1993). The default selection is Rubey.

Most fall velocity derivations start with balancing the gravitational force and the drag force on a particle falling through the water column. The free body diagram is included in the figure below.



$$F_D = \frac{1}{2}\pi\rho c_D\left(\frac{D}{2}\right)^2 v_s^2$$

$$F_g = \frac{4}{3}\pi\rho R g\left(\frac{D}{2}\right)^3$$

However, the resulting equation is circular because fall velocity is function of the drag coefficient C_D, which is a function of the Reynolds number, which is itself a function of fall velocity. This self-referential quality of the force balance requires either an approximation of the drag coefficient/Reynolds number or an iterative solution. The fall velocity options in HEC-RAS are detailed in Chapter 12, pages 12-30 to 12-32, but a few brief comments on how each of these methods attempts to solve this equation (fall velocity dependence on fall velocity) are given below.

Rubey assumes a Reynolds number to derive a simple, analytical function for fall velocity. Toffaleti developed empirical, fall velocity curves that, based on experimental data, which HEC-RAS reads and interpolates directly. Van Rijn uses Rubey as an initial guess and then computes a new fall velocity from experimental curves based on the Reynolds number computed from the initial guess. Finally, Report 12 is an iterative solution that uses the same curves as Van Rijn but uses the computed fall velocity to compute a

105 https://www.hec.usace.army.mil/confluence/display/RASSED1D/Fall+Velocity

new Reynolds number and continues to iterate until the assumed fall velocity matches the computed within an acceptable tolerance.

Fall velocity is also dependent upon particle shape. The aspect ratio of a particle can cause both the driving and resisting forces in Figure 2 9 to diverge from their simple spherical derivation. All of the equations assume a shape factor or build one into their experimental curve. Only Report 12 is flexible enough to compute fall velocity as a function of shape factor. Therefore, HEC-RAS exposes shape factor as a user input variable but only uses it if the Report 12 method is selected.

For a more comprehensive understanding, including information on algorithms that translate hydrodynamics into transport, please refer to the "Sediment Transport Capacity[106]". This reference is relevant because HEC-HMS shares the same Sediment Transport engine as HEC-RAS.

Comprehensive information on fall velocity methods is provided in the HEC-RAS Hydraulic Reference Manual, as illustrated below.

The suspension of a sediment particle is initiated once the bed-level shear velocity approaches the same magnitude as the fall velocity of that particle. The particle will remain in suspension as long as the vertical components of the bed-level turbulence exceed that of the fall velocity. Therefore, the determination of suspended sediment transport relies heavily on the particle fall velocity.

Within HEC-RAS, the method for computing fall velocity can be selected by the user. Three methods are available and they include Toffaleti (1968), Van Rijn (1993), and Rubey (1933). Additionally, the default can be chosen in which case the fall velocity used in the development of the respective sediment transport function will be used in RAS. Typically, the default fall velocity method should be used, to remain consistent with the development of the sediment transport function, however, if the user has specific information regarding the validity of one method over the other for a particular combination of sediment and hydraulic properties, computing with that method is valid. The shape factor (sf) is more important for medium sands and larger. Toffaleti used a sf of 0.9, while Van Rijn developed his equations for a sf of 0.7. Natural sand typically has a sf of about 0.7. The user is encouraged to research the specific fall velocity method prior to selection.

$$sf = \frac{c}{\sqrt{ab}}$$

Symbol	Description	Unit
a	Length of particle along the longest axis perpendicular to the other two axes.	
b	Length of particle along the intermediate axis perpendicular to other two axes.	
c	Length of particle along the short axis perpendicular to other two axes.	

106 https://www.hec.usace.army.mil/confluence/display/RAS1DTechRef/Sediment+Transport+Capacity

12.4.1 Particle Settling Velocity

12.4.1.1 Toffaleti

Toffaleti (1968) presents a table of fall velocities with a shape factor of 0.9 and specific gravity of 2.65. Different fall velocities are given for a range of temperatures and grain sizes, broken up into American Geophysical Union standard grain size classes from Very Fine Sand (VFS) to Medium Gravel (MG). Toffaleti's fall velocities are presented in Table below.

Fall Velocity (Toffaleti, 1968)

Sand Grain Settling Velocity Versus Temperature, SP.G. 2.65, Shape Factor 0.9

TEMP °F	SETTLING VELOCITY IN FT/SEC							
	VFS	FS	MS	CS	VCS	VFG	FG	MG
35	.013	.045	.130	.305	.590	1.00	1.41	1.95
36	.013	.045	.131	.307	.592	1.00	1.41	1.95
37	.013	.046	.132	.310	.594	1.00	1.41	1.95
38	.014	.047	.133	.312	.596	1.00	1.41	1.95
39	.014	.047	.135	.314	.598	1.00	1.41	1.95
40	.014	.048	.136	.316	.600	1.00	1.41	1.95
41	.015	.049	.137	.318	.602	1.00	1.41	1.95
42	.015	.050	.138	.320	.604	1.00	1.41	1.95
43	.015	.051	.140	.321	.606	1.00	1.41	1.95
44	.016	.051	.141	.322	.608	1.00	1.41	1.95
45	.016	.052	.142	.323	.609	1.00	1.41	1.95
46	.016	.053	.143	.325	.610	1.00	1.41	1.95
47	.016	.053	.144	.326	.612	1.00	1.41	1.95
48	.017	.054	.145	.328	.614	1.00	1.41	1.95
49	.017	.055	.146	.330	.616	1.00	1.41	1.95
50	.017	.055	.147	.331	.618	1.00	1.41	1.95
51	.018	.056	.148	.333	.620	1.00	1.41	1.95
52	.018	.057	.150	.334	.621	1.00	1.41	1.95
53	.018	.057	.151	.336	.623	1.00	1.41	1.95
54	.018	.058	.152	.338	.624	1.00	1.41	1.95
55	.018	.059	.153	.340	.626	1.00	1.41	1.95
56	.019	.059	.154	.341	.627	1.00	1.41	1.95
57	.019	.060	.155	.343	.629	1.00	1.41	1.95
58	.019	.061	.156	.344	.630	1.00	1.41	1.95
59	.019	.061	.157	.346	.632	1.00	1.41	1.95
60	.020	.062	.158	.347	.633	1.00	1.41	1.95
61	.020	.063	.160	.349	.635	1.00	1.41	1.95
62	.020	.063	.161	.350	.636	1.00	1.41	1.95
63	.020	.064	.162	.351	.638	1.00	1.41	1.95
64	.021	.065	.163	.353	.639	1.00	1.41	1.95

TEMP °F	SETTLING VELOCITY IN FT/SEC							
	VFS	FS	MS	CS	VCS	VFG	FG	MG
65	.021	.065	.165	.354	.640	1.00	1.41	1.95
66	.021	.066	.166	.356	.641	1.00	1.41	1.95
67	.021	.067	.167	.357	.643	1.00	1.41	1.95
68	.022	.067	.168	.359	.644	1.00	1.41	1.95
69	.022	.068	.170	.360	.646	1.00	1.41	1.95
70	.022	.069	.171	.361	.647	1.00	1.41	1.95
71	.022	.070	.172	.362	.649	1.00	1.41	1.95
72	.023	.071	.173	.363	.650	1.00	1.41	1.95
73	.023	.071	.175	.364	.652	1.00	1.41	1.95
74	.023	.072	.176	.365	.653	1.00	1.41	1.95
75	.024	.072	.177	.366	.655	1.00	1.41	1.95
76	.024	.073	.178	.367	.656	1.00	1.41	1.95
77	.024	.073	.180	.368	.657	1.00	1.41	1.95
78	.024	.074	.181	.370	.658	1.00	1.41	1.95
79	.025	.074	.182	.371	.659	1.00	1.41	1.95
80	.025	.075	.183	.373	.660	1.00	1.41	1.95
81	.025	.075	.184	.375	.661	1.00	1.41	1.95
82	.025	.076	.185	.376	.662	1.00	1.41	1.95
83	.025	.077	.186	.378	.663	1.00	1.41	1.95
84	.026	.077	.187	.380	.664	1.00	1.41	1.95
85	.026	.078	.188	.381	.665	1.00	1.41	1.95
86	.026	.078	.190	.383	.666	1.00	1.41	1.95
87	.026	.079	.192	.385	.667	1.00	1.41	1.95
88	.027	.079	.194	.386	.668	1.00	1.41	1.95
89	.027	.080	.195	.388	.669	1.00	1.41	1.95
90	.027	.080	.196	.390	.670	1.00	1.41	1.95
91	.028	.081	.197	.391	.671	1.00	1.41	1.95
92	.028	.081	.198	.392	.672	1.00	1.41	1.95
93	.028	.082	.199	.393	.673	1.00	1.41	1.95
94	.028	.082	.200	.394	.674	1.00	1.41	1.95

12.4.1.2 Van Rijn

Van Rijn (1993) approximated the US Inter-agency Committee on Water Resources' (IACWR) curves for fall velocity using non-spherical particles with a shape factor of 0.7 in water with a temperature of 20°C. Three equations are used, depending on the particle size:

$$\omega_{sd} = \begin{cases} l\frac{Rgd^2}{18\nu} & \text{for } 0.065\,\text{mm} < d \leq 0.1\,\text{mm} \\ \frac{10\nu}{d}\left(\sqrt{1+0.01\frac{Rgd^3}{\nu^2}}-1\right) & \text{for } 0.1\,\text{mm} < d \leq 1\,\text{mm} \\ 1.1\sqrt{Rgd} & \text{for } 1\,\text{mm} \geq d \end{cases}$$

where

ν = kinematic viscosity [L^2/T]

d = grain size [L]

d∗ = d(Rg)1/3ν−2/3 = dimensionless grain size [-]

R = $\rho_s/\rho_w - 1$ = submerged specific gravity [-]

ρ_s = particle density [M/L^3]

ρ_w = water density [M/L^3]

g = gravitational constant (~9.81 m/s^2) [L/T^2]

12.4.1.3 Rubey

Rubey (1933) developed an analytical relationship between the fluid, sediment properties, and the fall velocity based on the combination of Stoke's law (for fine particles subject only to viscous resistance) and an impact formula (for large particles outside the Stoke's region). This equation has been shown to be adequate for silt, sand, and gravel grains. Rubey suggested that particles of the shape of crushed quartz grains, with a specific gravity of around 2.65, are best applicable to the equation. Some of the more cubic, or uniformly shaped particles tested, tended to fall faster than the equation predicted. Tests were conducted in water with a temperature of 16° Celsius.

$$\omega = F_1 \sqrt{(s-1)gd_s}$$

in which

$$F_1 = \sqrt{\frac{2}{3} + \frac{36\nu^2}{gd^3(s-1)}} - \sqrt{\frac{36\nu^2}{gd^3(s-1)}}$$

where

ν = kinematic viscosity [L^2/T]

g = gravitational constant (~9.81 m/s^2) [L/T^2]

d = grain size diameter [L]

R = ρs/ρw−1 = submerged specific gravity [-]

ρ_s = particle density [M/L^3]

ρ_w= water density [M/L^3]

The parameter F_1 has an upper limit of 0.79 which corresponds to particles larger than 1 mm settling in water with temperatures between 10 and 25°C. The Rubey formula is one of the earliest sediment fall velocity formulas developed. However, it tends to significantly under-predict the fall velocity for sediments courser than fine sand.

12.4.2 Hindered Settling

Hindered settling is the condition in which the settling velocity of particles or flocs is reduced due to a high concentration of particles. Hindered settling is primarily produced by particle collisions and the upward water flow equal to the downward sediment volume flux. Hindered settling occurs to both cohesive and noncohesive particles. However, the hindered settling correction described here only applies to noncohesive particles. When the sediment concentration is high (approximately larger than 3,000 mg/l), the settling of particles is reduced due to return flow, particle collisions, increased mixture viscosity, increased buoyancy, and wake formation. This process is referred to as hindered settling.

12.4.2.1 Richardson and Zaki

When using other particle settling velocities hindered settling is considered using a modified form of Richardson and Zaki (1952) (Hindered Settling[107]).

$$\frac{w_{sd}}{w_{sd0}} = (1 - C_{tV})^n$$

where

w_{sd} = sediment particle settling velocity for turbid water [L/T]

w_{sd0} = sediment particle settling velocity for clear water [L/T]

C_{tV} = total sediment concentration by volume [-]

n = empirical exponent [-]

The empirical coefficient n ranges between 3.75 to 4.45 for medium to fine sands (approximately 4.0 for normal flow conditions and particles in the range of 0.05 to 0.5mm) The above formulation differs from Richardson and Zaki (1952) in the inclusion of the maximum suspended sediment concentration which may be set to the bed dry density. This is physically correct since the particle velocity should become zero when the concentration is equal to the bed dry density. The empirical exponent that varies from is a function of the particle Reynolds number but is set to user-defined constant here for simplicity.

12.4.2.2 Kumbhakar

When using other particle settling velocities hindered settling is considered using a modified form of Kumbhakar (2017).

$$\omega_n = \omega_0 (1 - c)^{2.41 - 2\ln[1 - 0.83\exp\{-3.84(\frac{R}{c\Delta_p})^{0.45}\}]}$$

where

ω_n = Hindered settling velocity in mixture

107 https://www.hec.usace.army.mil/confluence/display/RAS2DSEDTR/Hindered+Settling

ω_0 = Settling velocities of particles in clear fluid

c = Volumetric concentration of suspended sediment particle

R = Particle Reynolds number

Δ_p = Submerged specific weight

12.5 Erosion and Sediment Transport References

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13 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.

13.1 Diversion Modeling

13.1.1 Basic Concepts and Equations

Figure 45 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.

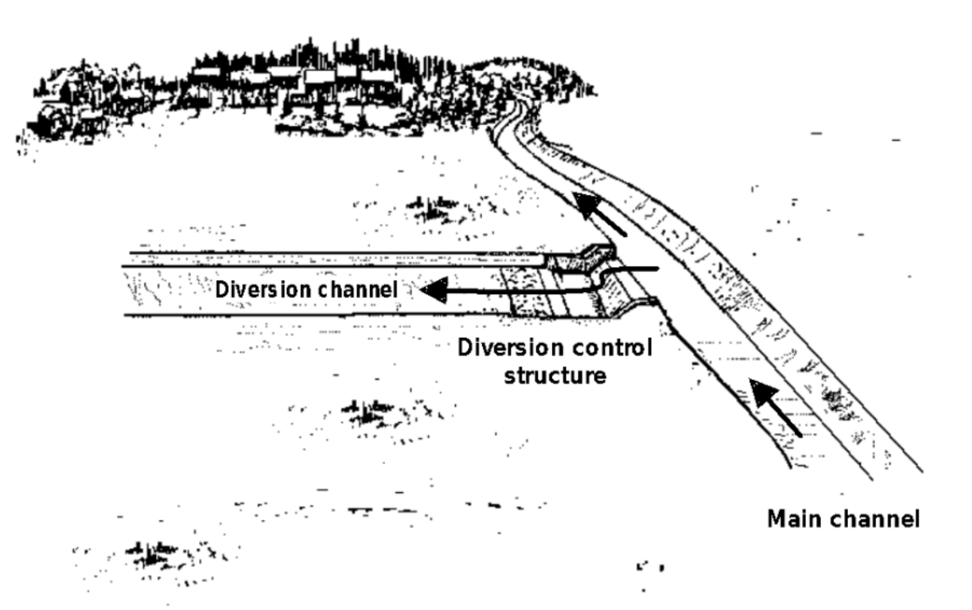

Figure 45.Illustration of a diversion structure.

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:

$$O_t^{main} = I_t - O_t^{bypass}$$

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.

13.1.2 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:

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

in which f(It) = 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 **Error! Reference source not found.** can be computed with the weir equation:

$$O = CLH^{1.5}$$

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.

13.1.3 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 **Error! Reference source not found.**. 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 **Error! Reference source not found.**. Chapter 8 provides more information about modeling a confluence.

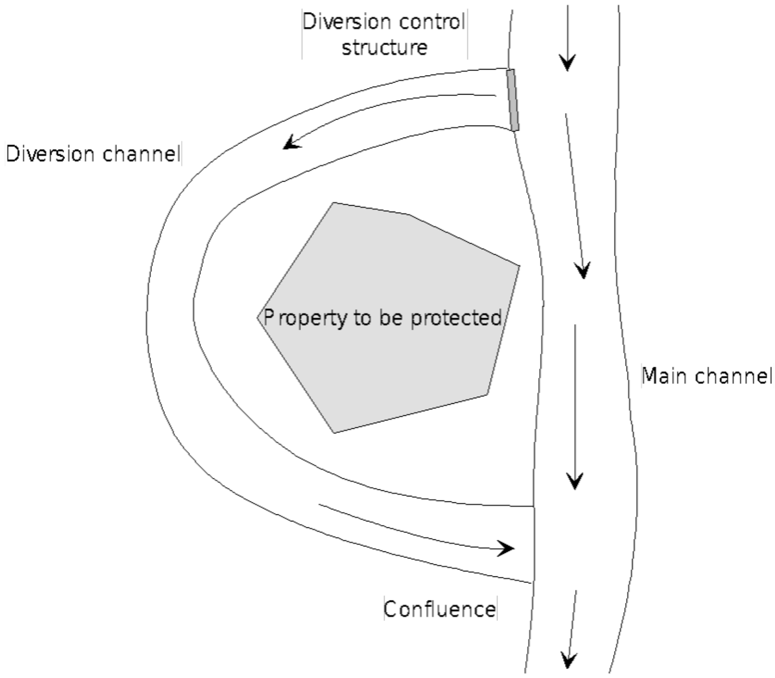

Figure 46.Illustration of a diversion return flow.

13.1.4 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.

13.2 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 **Error! Reference source not found.**. 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.
Error! Reference source not found. 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.)

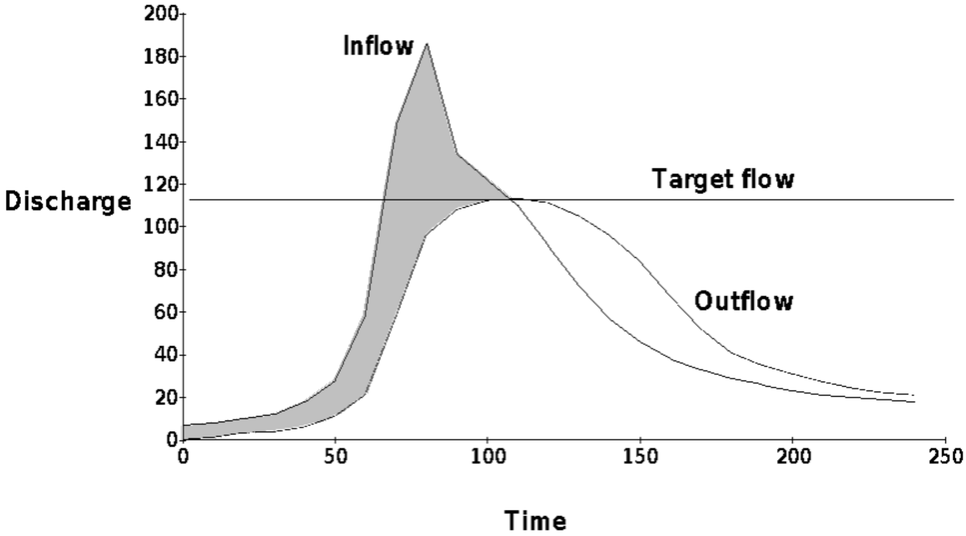

Figure 47.Illustration of the impact of detention.

The reservoir outlet may consist of a single culvert, as shown in **Error! Reference source not found.**. 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.

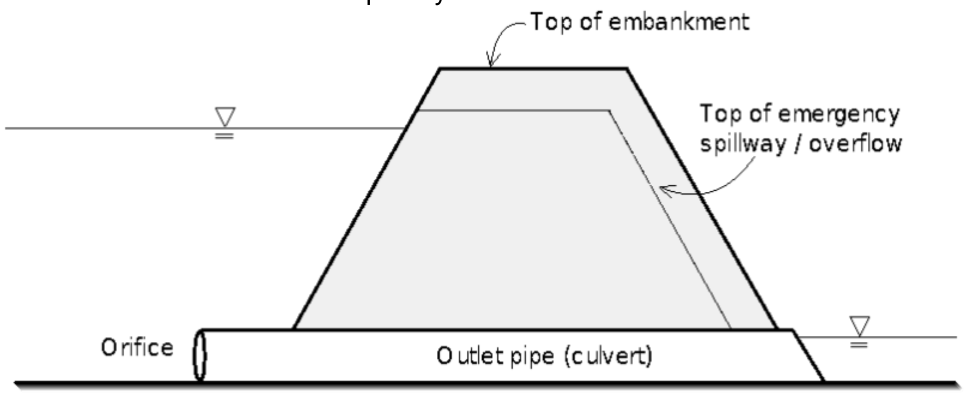

Figure 48.Simple detention structure.

13.2.1 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 t. It then solves recursively the following one-dimensional approximation of the continuity equation:

$$I_{avg} - O_{avg} = \frac{\Delta S}{\Delta t}$$

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:

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

in which t = index of time interval; It 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:

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

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 $(\frac{2S_{t+1}}{\Delta t} + O_{t+1})$ 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.

13.2.2 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. **Error! Reference source not found.** 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.

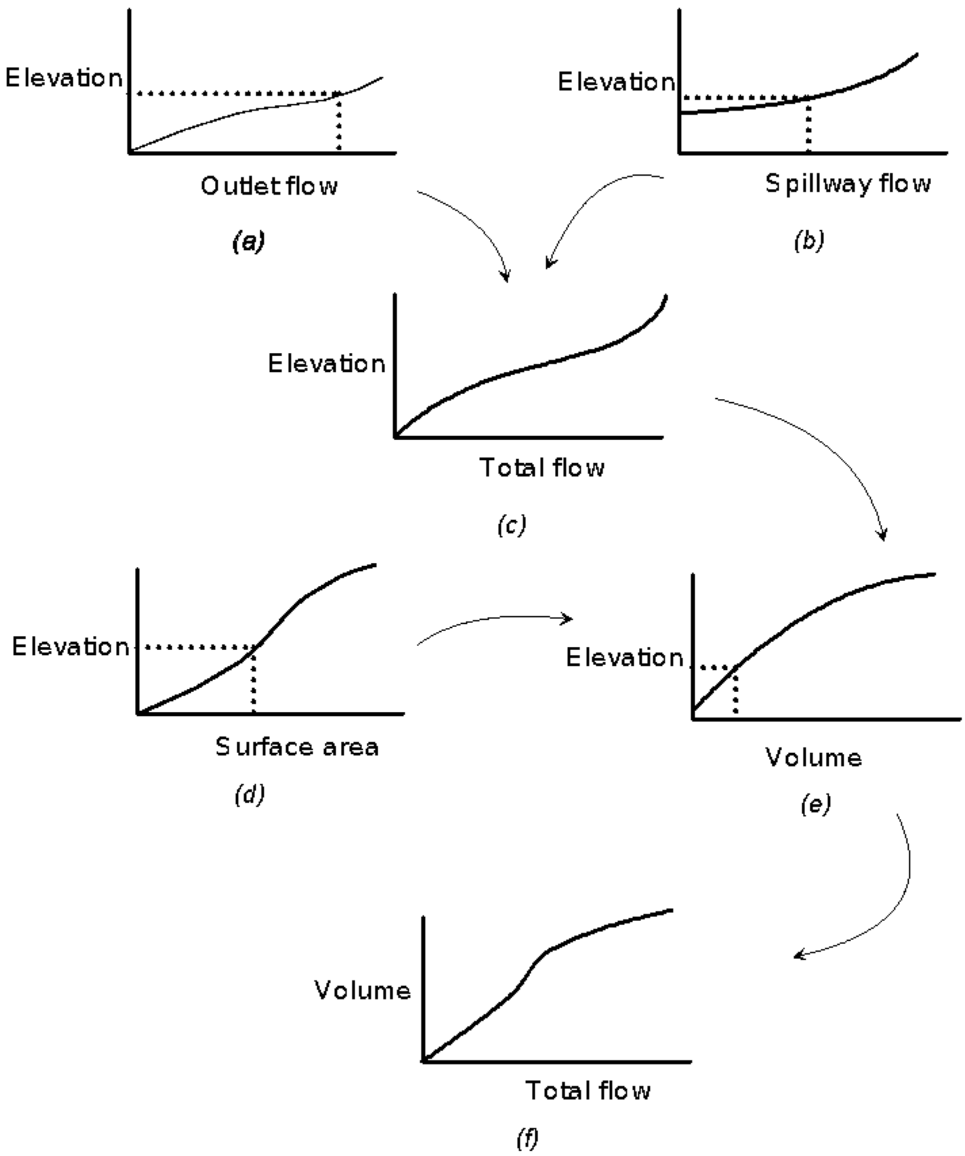

Figure 49.Illustration of the procedure for defining storage-outflow a relationship.

Figure 49 (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 **Error! Reference source not found.**, 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:

$$O = KA\sqrt{2gH}$$

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.

Error! Reference source not found.(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 111021603

(1965), to publications of the Soil Conservation Service (1985), and to publications of the Bureau of Reclamation (1977) for appropriate rating procedures.
Error! Reference source not found.(a) and (b) are combined to yield (c), which represents the total outflow when the reservoir reaches a selected elevation.
Error! Reference source not found.(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. **Error! Reference source not found.**(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.

13.2.3 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.

13.3 References9

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USACE (1998b). HEC-5 simulation of flood control and conservation systems user's manual. Hydrologic Engineering Center, Davis, CA.
CHAPTER 12

14 Modeling Reservoirs

This chapter describes how the reservoir element in HEC-HMS is used for modeling reservoirs or other types of water storage features, such as detention or retention ponds or natural lakes. Reservoirs have many uses, including flood mitigation, water supply, hydropower, and recreation. Reservoirs functionally change the hydrograph on a stream, allowing for inflowing water to be stored and then released at altered times and rates. Some reservoirs are operable and releases can be controlled, while others, such as detention ponds, may use an uncontrolled culvert or weir as a release structure. The primary objective of modeling reservoirs in HEC-HMS is to include how the reservoir changes the hydrologic response in a watershed.

A reservoir can mitigate downstream flooding and other 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 1. 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, the reservoir provides storage for the water. 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 2 is a sketch of a simple detention structure. The structure stores water temporarily and releases it, either through controlled or uncontrolled outlets 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. (Structure operators also check that this change in timing of the peak does not adversely coincide with flows from other parts of the basin that are downstream of the detention structure.)

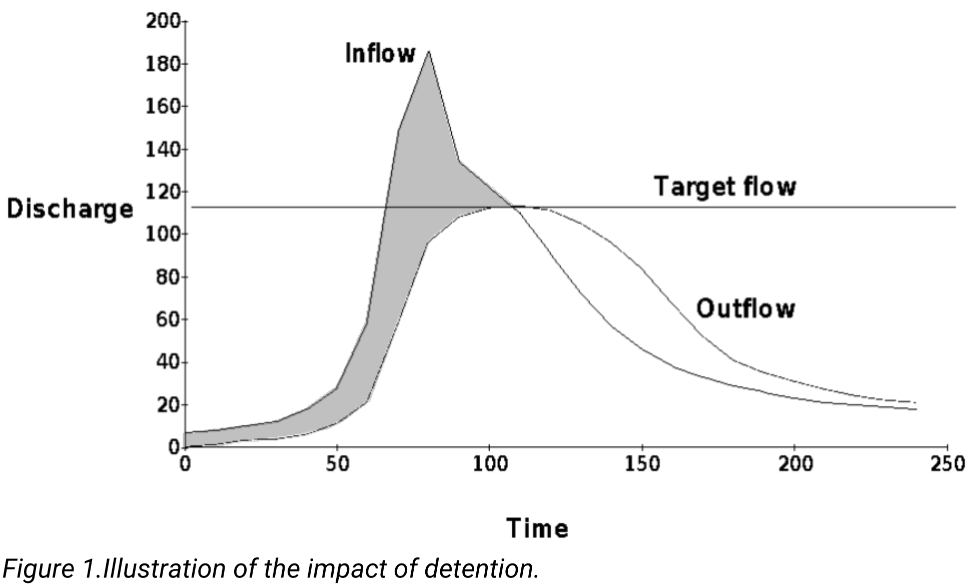


Figure 1.Illustration of the impact of detention.

The reservoir outlet may consist of a single culvert, as shown in Figure 2. 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, the characteristics of the spillway, and the tailwater condition. The reservoir can also have an auxiliary spillway that releases to a different stream.

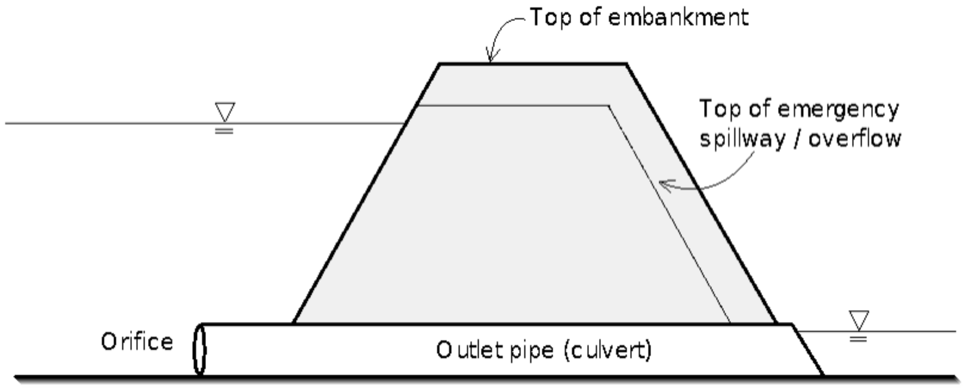


Figure 2.Simple detention structure.

14.1 Basic Concepts and Equations

While a reservoir element in HEC-HMS conceptually represents a natural lake or a lake behind a dam, the actual storage simulation calculations are performed by a routing method in conjunction with a storage method, contained within the reservoir.

14.1.1 Defining Routing

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

$$I_{avg} - O_{avg} = \frac{\Delta S}{\Delta t}$$

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

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

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

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

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 $(\frac{2S_{t+1}}{\Delta t} + O_{t+1})$ can be calculated with the equation above. 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.

14.1.2 Defining Detention (Storage)

To model storage within the reservoir element, the relationship between outflow and storage for the reservoir must be defined by the user. HEC-HMS can accept this relationship described in terms of storage-outflow, elevation-storage-outflow, or a combination of elevation-area-outflow with the elevation-storage relationship. The relationship that is developed and provided will depend on the available data, characteristics of the pond or reservoir, the outlet, and the spillway. Figure 3 illustrates how the relationship in a simple case might be developed. Hand calculations or a hydraulic model, like HEC-RAS, can be used develop storage-outflow relationships for complex structures.

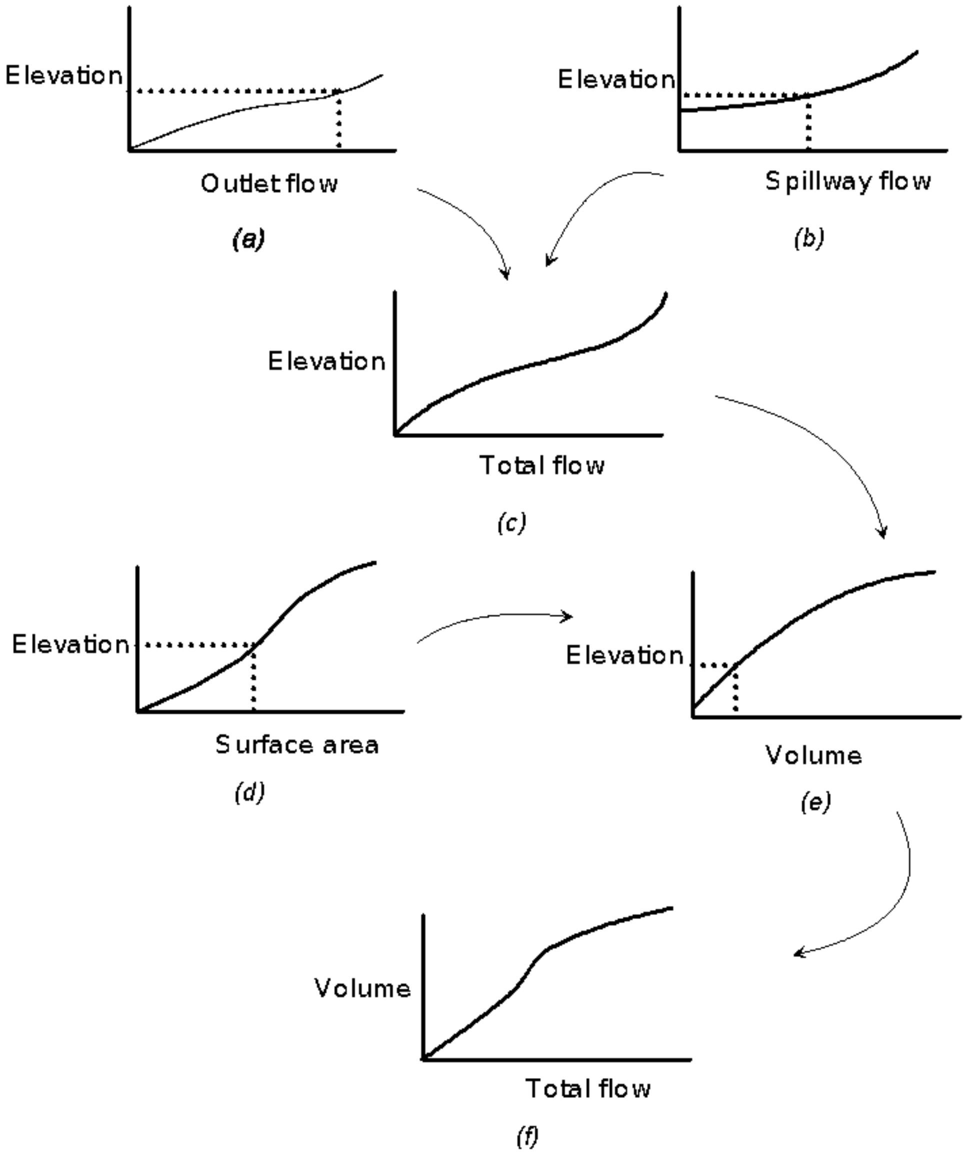


Figure 3.Illustration of the procedure for defining a storage-outflow relationship.

Figure 3(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 2, 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:

$$O = KA\sqrt{2gH}$$

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; and *g* is the gravitational constant. This head is the difference in the downstream water-surface elevation and the upstream (pond) water-surface elevation.

Figure 3(b) is the spillway rating function. In the simplest case, this function can be developed with the weir equation. 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 3(a) and (b) are combined to yield Figure 3(c), which represents the total outflow when the reservoir reaches a selected elevation.

Figure 3(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 3(e) is developed from this with solid-geometry principles.
For an arbitrarily-selected elevation, the storage volume can be found in Figure 3(e), the total flow found in Figure 3(c), and the two plotted to yield the desired relationship, as shown in Figure 3(f). With this relationship, Equation x can be solved recursively to find the outflow hydrograph ordinates, given the inflow.

If storage is not known but the relationship for elevation-area is, the conic formula is used to estimate the volume in slices of the reservoir using elevation and area. The conic formula, as used to approximate storage, is expressed by the following equation:

$$\Delta S = \frac{(elev_1 - elev_2)}{3}(A_1 + A_2 + \sqrt{A_1 A_2})$$

where ΔS is the incremental storage between two reservoir elevations ($elev_1$ and $elev_2$) and their respective sectional areas are A_1 and A_2. A limitation to this approach is that it does not work well for very large reservoirs, where the level pool assumption is not realistic.

The reservoir can have one or more inflows and computed outflows through one or more outlets. Assumptions include a level pool.

14.2 Routing and Storage Methods

While a reservoir element in HEC-HMS conceptually represents a natural lake or a lake behind a dam, the actual storage simulation calculations are performed by a routing method in conjunction with a storage method, contained within the reservoir. The available options are shown in Table 1.

Table 1.Storage Method and Initial Condition Options for each Routing Method

Routing Method	**Storage Method**	**Initial Condition Options**
Outflow Curve	Elevation-Area-Discharge	Discharge, Elevation, Inflow = Outflow
	Elevation-Storage-Discharge	Discharge, Elevation, Inflow = Outflow, Storage
	Storage-Discharge	Discharge, Inflow = Outflow, Storage
Specified Release	Elevation-Area	Elevation, Inflow = Outflow

	Elevation-Storage	Elevation, Inflow = Outflow, Storage
Outflow Structures	Elevation-Area	Elevation, Inflow = Outflow
	Elevation-Storage	Elevation, Inflow = Outflow, Storage

14.2.1 Selecting a Routing Method

Three different routing methods are available: Outflow Curve, Specified Release, and Outflow Structures. A fourth option is to choose a routing method of None, which assumes no storage and passes inflow, making the model element only nominally representative of a reservoir.

14.2.1.1 Outflow Curve Routing method

The **Outflow Curve Routing method** is designed to represent a reservoir with a known storage-outflow relationship. This method does not allow for individual representation of the components of the outlet works, rather, it lumps all outflow structures into one storage-discharge relationship. This routing method is best used for simple reservoirs.

For the **Outflow Curve Routing method**, regardless of which storage method option is selected, the routing is always performed using only the storage-discharge curve. After the routing is complete using the storage-discharge curve, the program will compute the elevation and surface area for each time step, depending on the selected storage method.

14.2.1.2 Specified Release Routing method

The **Specified Release Routing method** is designed to model reservoirs where the total discharge is known for each time interval of a simulation. Usually this method is used when the discharge is either observed or completely specified by an external decision process. The method can then be used to preserve the specified release and track the storage using the inflow, outflow, and conservation of mass. The Specified Release method uses a specified release and computes the storage that would result.
With this method, it is possible to set a maximum capacity limit on the reservoir. The storage at the end of the previous time interval, and the inflow volume and specified outflow volume for the current time interval, are used to calculate the storage at the end of the current time interval (equation below). The calculated storage will not be changed when it exceeds the user setting; however, a warning message will occur when the calculated storage exceeds the setting value.

$$S_{t+1} = S_t + I_t - O_t$$

14.2.1.3 Outflow Structures Routing method

The **Outflow Structures Routing method** is designed to model reservoirs with a number of uncontrolled outlet structures. For example, a reservoir may have a spillway and several low-level outlet pipes. While there is an option to include gates on spillways, the ability to control the gates is extremely limited at this time. There are currently no gates on outlet pipes. However, there is an ability to include a time-series of releases in addition to the uncontrolled releases from the various structures. An external analysis may be used to

develop the additional releases based on an operations plan for the reservoir.
Additional features in the reservoir for culverts and pumps allow the simulation of interior ponds. This class of reservoir often appears in urban flood protection systems. A small urban creek drains to a collection pond adjacent to a levee where flood waters collect. When the main channel stage is low, water in the collection pond can drain through culverts into the main channel. Water must be pumped over the levee when the main channel stage is high.

14.2.2 Selecting a Storage Method

Once a routing method has been selected, an associated Storage Method must be selected. The Storage Method defines the relationship between detention and discharge. There are five storage methods to choose from: Elevation-Area-Discharge, Elevation-Storage-Discharge, Storage-Discharge, Elevation-Area, and Elevation-Storage. The user must select the functions to be used and which is to be primary if there is more than one curve.

Different sets of storage methods are available, depending on the routing method selected. Each of these combinations are shown in Table XXX. For a simple reservoir, the combination of the Outflow Curve routing method and the Storage-Discharge storage method is preferred. The storage-discharge relationship can be defined with some dead pool storage associated with zero outflow. For the Specified Release Routing Method, a time series of releases is specified. Storage is post-processed using the elevation-area or the elevation-storage relationship. For the Elevation-Area-Discharge and Elevation-Area Storage Methods, the program automatically transforms the elevation-area curve into an elevation-storage curve using the conic formula. The Elevation-Area-Discharge Storage Method option is rarely used.

Because HMS doesn't use triplet curves, if Elevation-Area-Discharge or Elevation-Storage-Discharge is selected, ultimately the storage-discharge relationship is still used to calculate the routing. Elevation is post-processed. The user will select the primary field when entering data (i.e., which curve to key on), and interpolation is used to get the other column.

14.2.2.1 Setting Initial Conditions

The initial condition sets the amount of storage in the reservoir at the beginning of a simulation. The initial condition can simply be defined using the storage value, or there are other options depending on the method selected for specifying the storage characteristics of the reservoir. Initial conditions can be set using Discharge, Elevation, Inflow-Outflow, or Storage.

When the Elevation-Storage method is selected, you may choose to specify the initial elevation or the initial storage. An initial elevation will be converted to storage using the elevation-storage relationship. When the Elevation-Area method is selected, you must specify an initial elevation. Again, the conic formula is used to develop an Elevation-Storage relationship, and thus the initial elevation is converted to a storage. The initial condition can also be set to Inflow=Outflow, which takes the reservoir inflow at the beginning of the simulation, and uses the storage-discharge curve to determine the storage required to produce that same flowrate as the outflow from the reservoir. If only the elevation-discharge curve is available, it uses the elevation-storage curve to find the matching storage. Typically, the Inflow=Outflow option should not be used. The initial storage or elevation should be defined at the beginning of a simulation.

Some storage methods permit the specification of Elevation as the initial condition. In such a case, the elevation provided by the user is used to interpolate a storage value from the elevation-storage curve. Other storage methods permit the specification of Discharge as the initial condition. In such a case, the storage is interpolated from the storage-discharge curve. The Pool Elevation method can also be selected for the initial condition. In those cases, the elevation provided by the user is used to interpolate a storage value from the elevation-storage curve.

14.3 Outflow Curve Routing Method

For the **Outflow Curve Routing Method**, regardless of which storage method option is selected, the routing is always performed using only the storage-discharge curve, because HEC-HMS does not use triplet curves. After the routing is complete using the storage-discharge curve, the program will compute the elevation and surface area for each time step, depending on the selected storage method.

Interpolation is used when the **Elevation-Storage-Discharge** or **Elevation-Area-Discharge** storage methods are used. This means that it is not necessary for the storage-discharge and elevation-storage curves used in the Elevation-Storage-Discharge method to contain matching independent variables. The two curves do not need to have the same storage values in each curve, or even have the same number of rows. At compute time, the two curves selected by the user are combined into a single routing table with three rows: elevation, storage, and discharge. The table is initially configured using the curve selected by the user as the **Primary** curve. The remaining column is interpolated from the curve *not selected* as the primary curve. Finally the storage routing is completed from the combined table using the storage and outflow columns, and then elevation and area is calculated from the computed storages where possible. A similar procedure is also used with the **Elevation-Area-Discharge** storage method. Elevation is post-processed output.

The approach for calculating the outflow is basically the same as using Modified Puls routing with a single step (maximum attenuation) and assumes a level pool for the whole element. For the determination of routing, the Storage-Discharge relationship is not used directly. Instead a Storage Indication Table (which is also used for Modified Puls routing) is used:

$$SI_i = \frac{Stor_i}{\Delta t} + \frac{1}{2}Q_i^{out}$$

$$SI_{i+1} = SI_i - Q_i^{out} + \frac{1}{2}(Q_i^{in} + Q_i^{out})$$

Storage Indication Table

SI	*Qout*

14.3.1 Applicability and Limitations of the Outflow Curve Method

The Outflow Curve Routing Method of modeling a reservoir is appropriate for simulating performance of any configuration of outlets. 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 outflow curve method should not be used. Instead, an unsteady-flow network model 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-ResSim (USACE, 2013), should be used.

14.4 Specified Release Routing Method

For the **Specified Release Routing Method**, there are two different options for specifying the storage relationship. The first option is the **Elevation-Storage** choice. The user must select an elevation-storage curve from the available curves in the Paired Data Manager. After the routing is complete, the program will compute the elevation and storage for each time interval. The second option is the **Elevation-Area** choice, which requires the selection of an elevation-area curve from the available curves in the Paired Data Manager. With this choice, the program automatically transforms the elevation-area curve into an elevation-storage curve using the conic formula. After the routing is complete, the program will compute the elevation, surface area, and storage for each time interval.

You must select a discharge time-series gage as the outflow from the reservoir. The gage should record the discharge to use for each time interval of the simulation. If there is missing data in the record and the basin model options are set to replace missing data, a zero flow rate will be substituted for each missing data value. If the basin model is not set to replace missing data, any missing data will cause the simulation to stop and an error message will be displayed.

The maximum release setting is optional. It will cause a warning message during a simulation if the specified release exceeds the setting value. The specified release from the time-series gage record will always be discharged from the reservoir. However, the warning will occur when the specified release exceeds the optional maximum release value.

The maximum capacity setting is optional. It will cause a warning message during a simulation if the calculated storage exceeds the setting value. The storage is calculated for each time interval **using conservation of mass**. The storage at the end of the previous time interval, and the inflow volume and specified outflow volume for the current time interval, are used to calculate the storage at the end of the current time interval. The calculated storage will not be changed when it exceeds the setting value. However, the warning message will occur when the calculated storage exceeds the setting value.
Start with the conservation of mass.

$$\frac{S_{i+1} - S_i}{dt} = \frac{dS}{dt} = \sum Q_{i+1}^{in} - \sum Q_{i+1}^{out}$$

The change in storage in a timestep is the difference between the inflows and the outflows. Adding the inflows is straightforward, but there can be many outflows. We assume the outflows can be calculated given the pool and tailwater. Typically, the determination of each individual outflow object is straightforward. Each outlet has its own piece in the equation which is calculated based on given pool and tailwater. The object that represents each outlet is captured and stored at the end of the time step.

This becomes a root-finding problem and can be solved for storage using standard numerical methods. The root-finding algorithm Brent's Method is used because it is a very dynamic solution that combines speed and reliability (Numerical Methods reference).

14.4.1 Applicability and Limitations of the Specified Release Method

The Specified Release Routing Method is best used for replicating events, forensic hydrology, or calibrating the channel network. Discharge is calculated based on the outlet elements that are specified, which can include outlets, spillways, gates, and dam top, but it cannot include evaporation. Maximum release and capacity settings offer warnings but do not stop the compute. After calibration with the Specified Release Routing Method, the model may be used with a different routing approach.

14.5 Outflow Structures Routing Method

For the **Outflow Structures Routing Method**, the routing is computed by evaluating all outlets defined by the user. After the routing is complete, the program will compute the elevation, surface area, and storage for each time interval.

An initial condition is given to set the amount of storage in the reservoir at the beginning of a simulation. The storage can be simply set as a reservoir elevation or volume of water. If an elevation is set, the elevation-storage curve is used to interpolate a storage volume. Some storage methods permit the specification of **Elevation** as the initial condition. The initial condition can also be set using discharge; in this case, the storage is interpolated from the storage-discharge curve. The initial condition options depend on the selected storage method.

14.5.1 Applicability and Limitations of the Outflow Structures Method

The Outflow Structures Routing Method is ideal for XXX. 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-ResSim (USACE, 2013), should be used.

14.6 Outflow Structures

Outlets and other discharge structures can only be modeled individually when using the **Outflow Structures** routing method (see page 270). The table below lists available outflow structure types and subtypes, which include outlets, spillways, gates, pumps, and dam tops, breaks, and seepage. Evaporation is also a specific type of release that requires the outflow structures routing method.

Table 2.Outflow Structures available in HEC-HMS

Structure Type	Description
Outlet	Lower elevation pipes, etc.
Culvert	Allows for partially full and submerged, pressure flow

Structure Type	Description
Orifice	Assumes a large, submerged outlet
Spillway - ungated	Structures on dam top, typically uncontrolled
Broad-Crested	Uncontrolled flow using weir assumptions
Ogee	Uncontrolled outlet with an approach channel, designed for a specific head. Weir flow assumptions, adjusts based on upstream head
Specified	Allows for an elevation-discharge relationship
Spillway - gated	Allows for controlled spillways
Sluice	Gate moves on a vertical plane
Radial	Gate rotates on a horizontal axis
Dam Top	Allows for uncontrolled overtopping
Level	Constant elevation dam top
Non-Level	Represents several different segments of a cross section. Can be used for separate sections of dam, separated by other outlets.
Pump	All pumps use the head-discharge relationship to calculate flow
Dam Break	Dam failures. Allows for defined trigger time and development
Overtop	Dam is overtopped and experiences erosion
Piping	Piping occurs through dam and eventually cuts away the dam top
Dam Seepage	Tabular
Additional Release	User specified tabular release

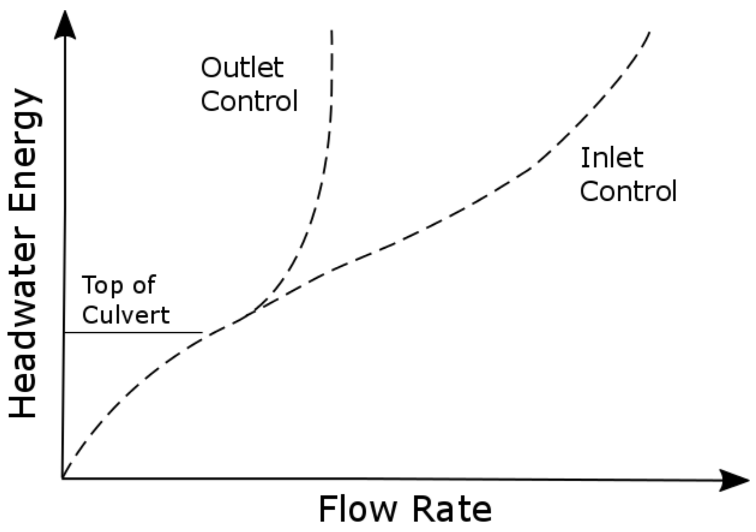
Structure Type	Description
Evaporation	Modeled as its own outflow structure based on user-defined relationship

14.7 Outlets

Outlets typically represent structures near the bottom of the dam that allow water to exit in a controlled manner. They are often called gravity outlets because they can only move water when the head in the reservoir is greater than the head in the tailwater. Up to ten independent outlets can be included in the reservoir.

14.7.1 Culvert Outlet

The culvert outlet can handle may flow types including pressure flow, so it allows for partially full or submerged flow through a culvert with a variety of cross-sectional shapes. Culvert flow calculations can be complicated, but the approach taken by HEC-HMS is shared with RAS – it simplifies the analysis by considering the flow either Inlet Control or Outlet Control. As described in Chapter 6 of the HEC-RAS Technical Reference Manual, "Inlet control flow occurs when the flow capacity of the culvert entrance is less than the flow capacity of the culvert barrel. The control section of a culvert operating under inlet control is located just inside the entrance of the culvert. The water surface passes through critical depth at or near this location, and the flow regime immediately downstream is supercritical. For inlet control, the required upstream energy is computed by assuming that the culvert inlet acts as a sluice gate or as a weir. Therefore, the inlet control capacity depends primarily on the geometry of the culvert entrance. Outlet control flow occurs when the culvert flow capacity is limited by downstream conditions (high tailwater) or by the flow carrying capacity of the culvert barrel. The HEC RAS culvert routines compute the upstream energy required to produce a given flow rate through the culvert for inlet control conditions and for outlet control conditions. In general, the higher upstream energy "controls" and determines the type of flow in the culvert for a given flow rate and tailwater condition. For outlet control, the required upstream energy is computed by performing an energy balance from the downstream section to the upstream section. The HEC RAS culvert routines consider entrance losses, friction losses in the culvert barrel, and exit losses at the outlet in computing the outlet control headwater of the culvert."
HEC-HMS shares the RAS culvert routines, and thus requires the same input data. RAS, however, assumes a roadway crest, whereas HEC-HMS does not. Figure XXX depicts the relationships between upstream energy and flow rate for outlet control and inlet control flows.



Thus, there are three different solution methods for culvert outlets, allowing for **Inlet Control**, **Outlet Control**, or **Automatic**. Automatic allows the program to determine whether the flow is inlet or outlet controlled at each timestep. This approach is highly recommended, unless the user is modeling a special case.
Table 2.Culvert Outlet Settings

Settings	**Options**
Direction	
Number of Barrels	Select up to 10 identical barrels
Solution Method	Automatic, Inlet Control, Outlet control
Shape	Circular, Semi-Circular, Elliptical, Arch, High Profile Arch, Low Profile Arch, Pipe Arch, Box, Con Span
	Depending on which shape you select, you have a subset of Charts and Scales to choose from. (big list)
Settings	Length, Rise, {Span}, Inlet Elevation, Entrance Coefficient, Outlet Elevation, Exit Coefficient, Manning's n.

The limitations to the culvert calculations are that no upstream or downstream cross sections are used. So the energy gradeline is calculated assuming a quiescent, level pool above the inlet and a quiescent stilling basin at the outlet.

Inlet Controlled
Use this method when the culvert outflow is controlled by a high pool elevation in the reservoir. See equations 6-2 and 6-3 in the RAS Tech Ref Man for the calculations used. HEC-HMS departs from the RAS approach at the Outlet Controlled Headwater. Equation 6-4 is modified for HEC-HMS so that the velocities are assumed zero. The same is true for equation 6-7.

Outlet Controlled

Outlet controlled calculation can handle zero, partial, pressurized flow. It changes based on the shape of the culvert. Culvert outflow that is controlled by a high tailwater condition uses this

Automatic
In order to determine which type of flow rules, energy minimization is performed. Flow is computed using inlet or outlet control and the lower flow is chosen.

Outlet Shape

You must specify the outlet shape and related parameters so that the HEC-HMS can use the appropriate equations. Table XXX shows a list of the outlet cross section shapes and the associated parameters used in the calculations.

Table 3.Listing of which parameters are required for each cross section shape.

Cross Section Shape	Diameter	Rise	Span
Circular	X		
Semi Circular	X		
Elliptical		X	X
Arch		X	X
High-Profile Arch			X
Low-Profile Arch			X
Pipe Arch		X	X
Box		X	X
Con Span		X	X

14.7.2 Orifice Outlet

The orifice outlet assumes a large outlet with sufficient submergence for orifice flow conditions to dominate. It should not be used to represent an outlet that may flow only partially full. The necessary submergence is typically present for low level reservoir outlets, but it may not be assured for small reservoirs, such as those on farms. If there is any uncertainty about whether or not the large orifice approach is appropriate, it is best to use the Culvert approach instead. The downside is that it may take ten to twenty times longer to calculate.

In order to ensure that the outlet is experiencing pressure flow conditions, the inlet of the structure should be submerged at all times by a depth at least 0.2 times the height of the orifice outlet. The approximate height is estimated to be the square root of the area. The water elevation is calculated and compared accordingly. HEC-HMS will check this condition and return and error messages with the number of times the condition was not met.

Table 4.Orifice Outlet Settings

Settings	Options
Direction	Main or auxiliary
Number of Barrels	You must select up to 10 identical barrels
Center Elevation	center of the cross-sectional flow area
Area	cross-sectional flow area
Coefficient	dimensionless discharge coefficient

The center elevation specifies the center of the cross-sectional flow area. It is used to compute the head on the outlet, so no flow will be released until the reservoir pool elevation is above this specified elevation. The cross-sectional flow area of the outlet must be specified. The orifice assumptions are independent of the shape of the flow area. The dimensionless discharge coefficient must be entered. This parameter describes the energy loss as water exits the reservoir through the outlet.

Section 6 of the HEC-RAS Technical Reference Manual includes detailed descriptions and equations used in culvert hydraulics and flow analysis. Since much of the approach is shared with HEC-HMS, these descriptions are not repeated here. Refer to the HEC-RAS manual for details on the algorithms. Note that the difference between the HEC-RAS and the HEC-HMS outlet flow calculations is that HEC-HMS does not consider flow in upstream or downstream cross sections, thus the equations are simplified by assuming zero velocity.

14.8 Spillways

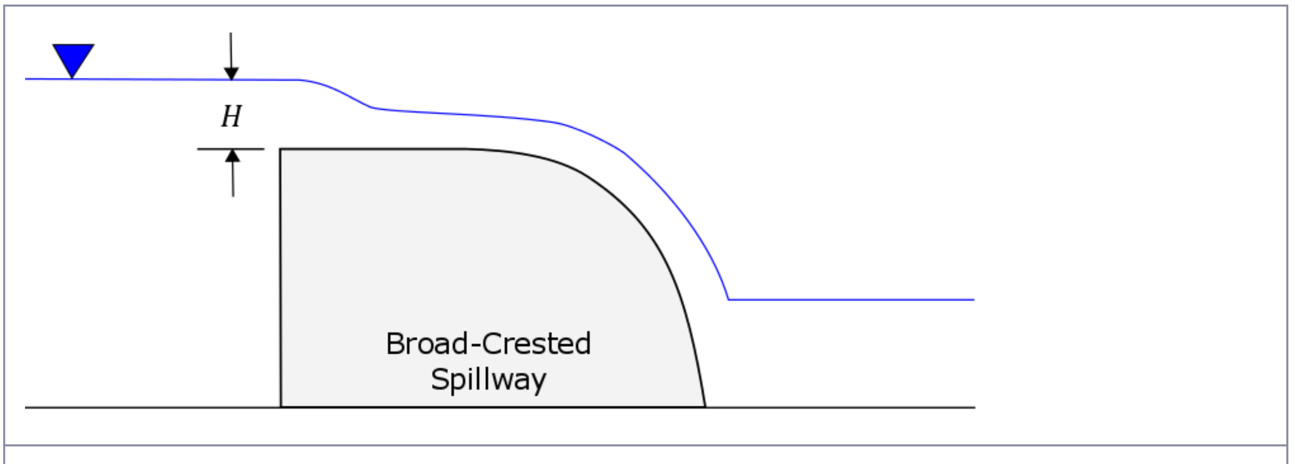
Spillways typically represent structures at the top of the dam that allow water to go over the dam top in an uncontrolled manner. Up to ten independent spillways can be included in the reservoir. There are three different methods for computing outflow through a spillway: Broad-Crested, Ogee, and User Specified. The broad-crested and ogee methods may optionally include gates. If no gates are selected, then flow over the spillway is unrestricted. When gates are included, the flow over the spillway will be controlled by the gates. Up to ten independent gates may be included on a spillway. The spillway may release to the main channel or an auxiliary location.

The calculations for spillways begin with the standard weir equation:

$$Q = CLH^{\frac{3}{2}}$$

where:
Q = Discharge over the weir or spillway crest.
C = Discharge coefficient, accounts for energy losses as water enters the spillway, flows through the spillway, and eventually exits the spillway. Typical values will range from 2.6 to 4.0 depending upon the shape of the spillway crest.
L = Length of the spillway crest.
H = Upstream energy head above the spillway crest.

HEC-HMS includes a secondary calculation to determine whether tailwater conditions alter the computed discharge, and the program adjusts accordingly. Details on the application of this equation can be found in the HEC-RAS Technical Reference Manual. HEC-HMS uses the same approach, with some simplifications.
Broad-Crested Spillway
The broad-crested spillway allows for uncontrolled flow over the top of the reservoir according to the weir flow assumptions.
The discharge coefficient *C* accounts for energy losses as water enters the spillway, flows through the spillway, and eventually exits the spillway. Depending on the exact shape of the spillway, typical values range from 1.10 to 1.66 in System International units (2.0 to 3.0 US Customary units) (Note: RAS manual says 2.6-3.1).
For each time step within the simulation, the head is estimated using the user-specified crest elevation of the spillway and the reservoir pool elevation.



14.8.1 Ogee Spillway

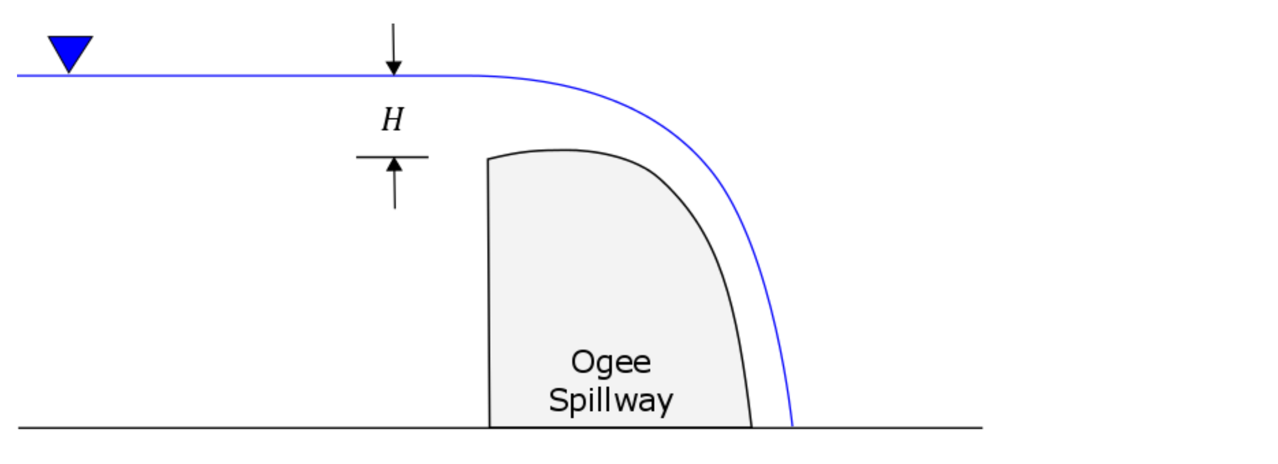
The ogee spillway allows for uncontrolled flow over the top of the reservoir according to the weir flow assumptions. However, the discharge coefficient in the weir flow equation is automatically adjusted when the upstream energy head is above or below the design head.

The ogee spillway may be specified with concrete or earthen abutments. These abutments should be the dominant material at the sides of the spillway above the crest. The selected material is used to adjust energy loss as water passes through the spillway. The spillway can be conceptually represented using one, two, or no abutments.

The ogee spillway is assumed to have an approach channel that moves water from the main reservoir to the spillway. If there is such an approach channel, you must specify the depth of the channel, and the energy loss that occurs between the main reservoir and the spillway. If there is no approach channel, the depth should be the difference between the spillway crest and the bottom of the reservoir, and the loss should be zero.

The crest elevation and length of the spillway are needed, as are the apron elevation and width.

The design head is the total energy head for which the spillway is designed. The discharge coefficient will be automatically calculated when the head on the spillway departs from the design head.

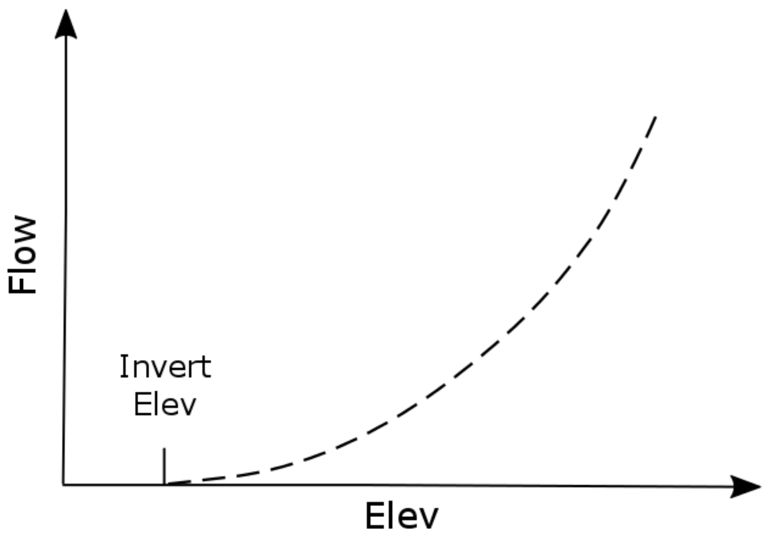


Don't know what equations are used here. Linear interpolation
Calculate the head ratio. If less than design ratio, then X, if greater than or equal to design ratio, then …
Check RAS. It's complicated.
Gate flow equation – either radial or sluice.

- **gate coefficient**
- **gate width**
- **trunnion^trunnion exponent**
- **gate opening^opening exponent**
- **head^head exponent**

14.8.2 Specified Spillway

The user-specified spillway can be used to represent spillways with flow characteristics that cannot be represented by the broad-crested or ogee weir assumptions. The user must create an elevation-discharge curve (Figure X) that represents the spillway discharge as a function of reservoir pool elevation, and HEC-HMS determines discharge though a basic table lookup approach. At this time there is no ability to include submergence effects on the specified spillway discharge. Therefore the user-specified spillway method should only be used for reservoirs where the downstream tailwater stage cannot affect the discharge over the spillway.



14.8.3 Gated Spillways

Spillway gates are an optional part of specifying the configuration of a spillway. They may be included on either broad-crested or ogee spillways.
An important part of defining gates on a spillway is the specification of how each gate will operate. It is rare that a gate is simply opened a certain amount and then never changed. Usually gates are changed on a regular basis in order to maintain the storage in the reservoir pool at targets; usually seasonal targets will be defined in the reservoir regulation manual. Under some circumstances, the gate operation may be changed to prevent flooding or accommodate other special concerns. At this time there is only one method for controlling spillway gates but additional methods will be added in the future.
The Fixed opening control method only accommodates a single setting for the gate. The distance between the spillway and the bottom of the gate is specified. The same setting is used for the entire simulation time window.

14.8.3.1 Sluice Gate

A sluice gate moves up and down in a vertical plane above the spillway in order to control flow. The water passes under the gate as it moves over the spillway. For this reason it is also called a vertical gate or underflow gate.
The width of the sluice gate must be specified. It should be specified as the total width of an individual gate.
The gate coefficient describes the energy losses as water passes under the gate. Typical values are between 0.5 and 0.7 depending on the exact geometry and configuration of the gate.
The orifice coefficient describes the energy losses as water passes under the gate and the tailwater of the gate is sufficiently submerged. A typical value for the coefficient is 0.8.

The HEC-RAS Hydraulic Reference Manual describes sluice gate flow calculations as follows:
An example sluice gate with a broad crest is shown in the figure below.

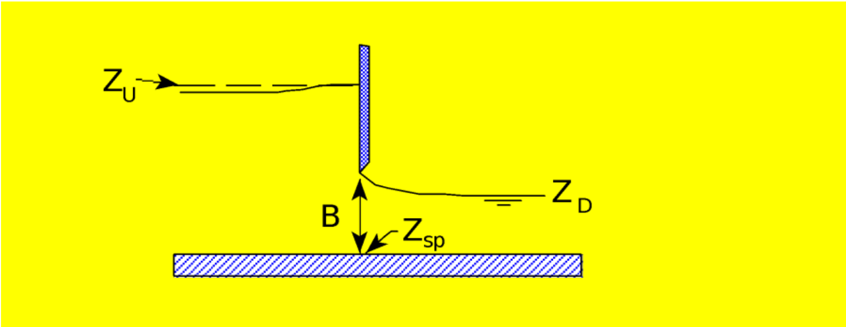


Figure X Example Sluice Gate with Broad Crested Spillway. (source: HEC-RAS Hydraulic Reference Manual)

The equation for a free flowing sluice gate is as follows:

$$Q = CWB\sqrt{2gH}$$

where: H = Upstream energy head above the spillway crest (Z_U - Z_{sp})
C= Coefficient of discharge, typically 0.5 to 0.7

When the downstream tailwater increases to the point at which the gate is no longer flowing freely (downstream submergence is causing a greater upstream headwater for a given flow), the program switches to the following form of the equation:

$$Q = CWB\sqrt{2g3H}$$

Where: H= Z_U - Z_D

Submergence begins to occur when the tailwater depth above the spillway divided by the headwater energy above the spillway is greater than 0.67. Equation 8-5 is used to transition between free flow and fully submerged flow. This transition is set up so the program will gradually change to the fully submerged Orifice equation (Equation x) when the gates reach a submergence of 0.80.

14.8.3.2 Radial Gate

A radial gate rotates above the spillway with water passing under the gate as it moves over the spillway. This type of gate is also known as a tainter gate.
The width of the radial gate must be specified. It should be specified as the total width of an individual gate. The gate coefficient describes the energy losses as water passes under the gate. Typical values are between 0.5 and 0.7 depending on the exact geometry and configuration of the gate.
The orifice coefficient describes the energy losses as water passes under the gate and the tailwater of the gate is sufficiently submerged. A typical value for the coefficient is 0.8. The pivot point for the radial gate is known as the trunnion. The height of the trunnion above the spillway must be entered. The trunnion exponent is part of the specification of the geometry of the radial gate. A typical value is 0.16. The gate opening exponent is used in the calculation of flow under the gate. A typical value is 0.72. The head exponent is used in computing the total head on the radial gate. A typical value is 0.62.
The HEC-RAS Hydraulic Reference Manual describes radial gate calculations as follows:
An example radial gate with an ogee spillway crest is shown in Figure X. (copied out of RAS)

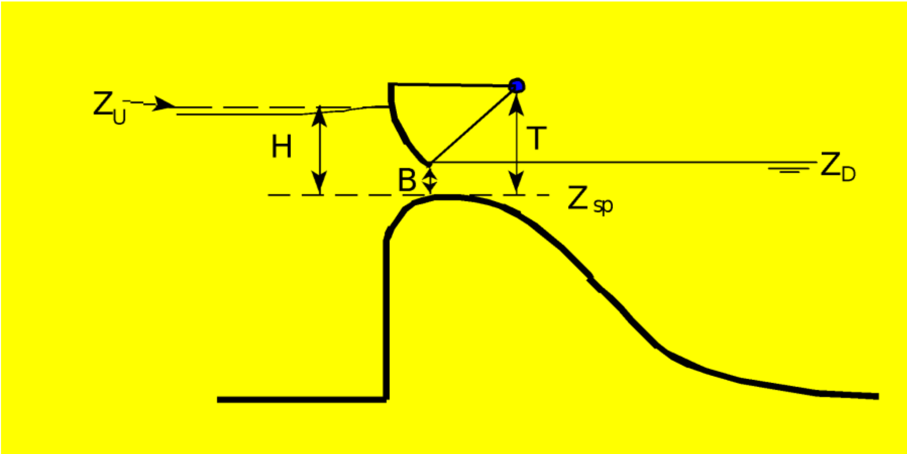


Figure X Example Radial Gate with an Ogee Spillway Crest

The flow through the gate is considered to be "Free Flow" when the downstream tailwater elevation (Z_D) is not high enough to cause an increase in the upstream headwater elevation for a given flow rate. The equation used for a Radial gate under free flow conditions is as follows:

$$Q = C\sqrt{2g}WT^{TE}B^{BE}H^{HE}$$

where:

Q= Flow rate in cfs
C= Discharge coefficient (typically ranges from 0.6 - 0.8)
W= Width of the gated spillway in feet
T= Trunnion height (from spillway crest to trunnion pivot point)
TE= Trunnion height exponent, typically about 0.16 (default 0.0)
B= Height of gate opening in feet
BE= Gate opening exponent, typically about 0.72 (default 1.0)
H= Upstream energy head above the spillway crest Z_U - Z_{sp}
HE= Head exponent, typically about 0.62 (default 0.5)
Z_U= Elevation of the upstream energy grade line
Z_D= Elevation of the downstream water surface
Z_{sp}= Elevation of the spillway crest through the gate

When the downstream tailwater increases to the point at which the gate is no longer flowing freely (downstream submergence is causing a greater upstream headwater for a given flow), the program switches to the following form of the equation:

$$Q = C\sqrt{2g}WT^{TE}B^{BE}(3H)^{HE}$$

where: $H=Z_U - Z_D$

Submergence begins to occur when the tailwater depth divided by the headwater energy depth above the spillway, is greater than 0.67. Equation 8-2 is used to transition between free flow and fully submerged flow. This transition is set up so the program will gradually change to the fully submerged Orifice equation when the gates reach a submergence of 0.80. The fully submerged Orifice equation is shown below:

$$Q = CA\sqrt{2gH}$$

where:

A= Area of the gate opening.
H= Z_U - Z_D
C= Discharge coefficient (typically 0.8)

Sluice gate always uses 0.5 head exponent. It appears the same for radial gate.
They are currently rebuilding to allow the user to specify the opening.
New outlet that allows for gates (no orifice or culvert gate)

14.9 Dam Tops

The top of the dam is important when modeling conditions that may cause water to flow over the top of the dam. Dam tops can be added to reservoirs that use the outflow structure routing method. These represent the top of the dam, above any spillways, where water goes over the dam top in an uncontrolled manner. In some cases a dam top can be used to represent an emergency spillway. Up to 10 independent dam tops can be included in the reservoir. There are two different methods for computing outflow through a dam top: level or non-level.

14.9.1 Level Dam Top

The level dam top assumes flow over the dam can be represented as a broad-crested weir. The calculations are essentially the same as for a broad-crested spillway, but they are included separately for conceptual representation of the reservoir structures.

$$Q = CLH^{3/2} \quad (8\text{-}6)$$

where:
Q = Discharge over the dam top.
C = Discharge coefficient; accounts for energy losses as water flows over the dam. Typical values will range from 2.6 to 3.3, depending upon the shape of the dam.
L = Length of the dam top.
H = Upstream energy head above the dam top.

The crest elevation of the dam top must be specified in order to allow for the determination of head.

The length of the dam top should represent the total width through which water passes, excluding any amount occupied by spillways.
The discharge coefficient accounts for energy losses as water approaches the dam top and flows over the dam. Depending on the exact shape of the dam top, typical values range from 1.45 – 1.84 in SI units (2.63 – 3.33 in US Customary units). ~~1.10 to 1.66 in System International units (2.0 to 3.0 US Customary units)~~. Civil Engineering Reference Manual says 1.45 – 1.84 2.63 – 3.33

14.9.2 Non-Level Dam Top

In order to model a non-level dam top, the structure can be represented by a cross section with eight station-elevation pairs. A separate flow calculation is carried out for each segment of the cross section, and the modeler can specify different coefficients. The broad-crested weir assumptions are made for each segment. An eight point cross section should be developed to represent the dam from one abutment to the other but excluding any spillways. Multiple dam tops can be used to represent the different sections of the dam top between spillways. The cross section should extend from the dam top up to the maximum water surface elevation that will be encountered during a simulation.

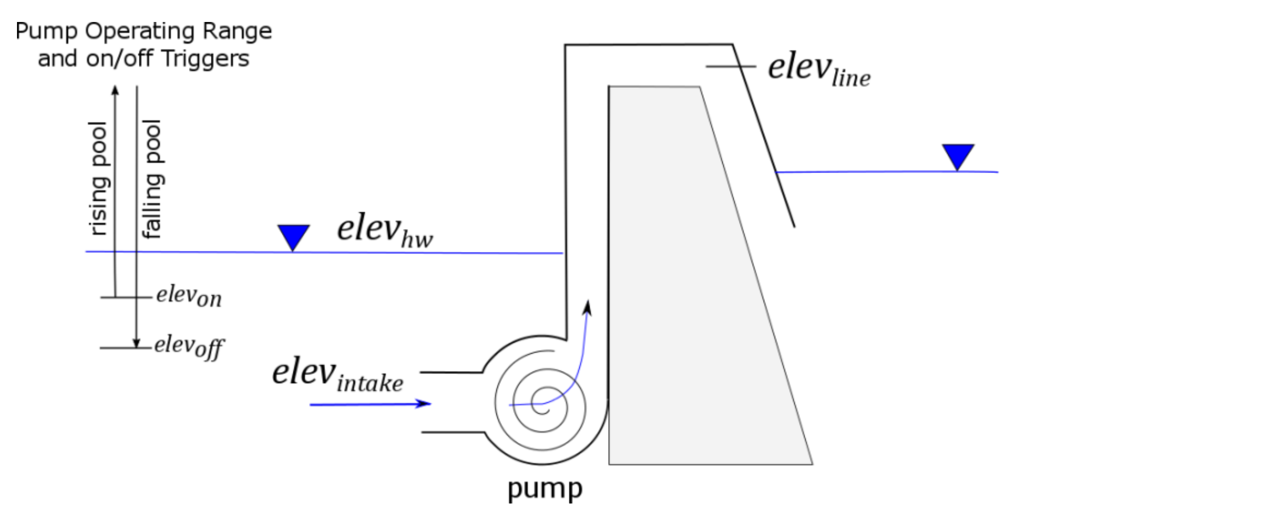
The same discharge coefficient value is used for all segments of the dam top. Typical values range from 2.6 to 4.0 depending on the exact shape of the dam top. Users enter the coefficients. Check the RAS manual for recommendations.
The tailwater is calculated.
Pool – dam elev
Then tailwater and recalculate pool-tailwater and if the head is smaller, you will go with the smaller flow. No reverse flow is allowed.

14.10 Pumps

Some smaller reservoirs such as interior detention ponds or pump stations may use pumps to move water out of the reservoir and into the tailwater when gravity outlets alone are insufficient. Pumps can only be included in reservoirs using the **Outflow Structures** routing method. Up to 10 independent pumps can be included in the reservoir.

14.10.1 Head-Discharge Pump

The head-discharge pump is designed to represent pumps that are applied in low-head, high-flow situations, such as the centrifugal type. These pumps are designed for high flow rates against a relatively small head. There are options for setting a reservoir pool elevation range for pumping and minimum times for the on or off condition. Figure X depicts the representation of this type of pump in HEC-HMS.



The intake elevation, $elev_{intake}$, defines the elevation at which the pump takes in water within the reservoir pool.
The line elevation, $elev_{line}$, defines the highest elevation in the pressure line from the pump to the discharge point.
The *equipment head loss* includes all energy losses between the intake and discharge points, including pipe losses related to expansion, contraction, and bends, and losses due to the pump itself. These losses are currently modeled as static losses, not as a function of flow. This total head loss is added to the head

difference due to reservoir pool elevation and tailwater elevation to determine the total energy head, h_{total} , against which the pump must operate.

$$h_{total} = elev_{line} - elev_{hw} + h_{loss}$$

A head-discharge curve is used to describe the capacity of the pump as a function of the total head. Total head is the head difference due to reservoir pool elevation and tailwater elevation, plus equipment loss. The head-discharge curve must be defined as an elevation-discharge function, although it actually represents head rather than elevation. If the pump is determined to be active, the pump discharge is determined by looking up and interpolating the flow value based on the total head value.

The pump is set to turn on at a specified *on-trigger elevation,* $elev_{on}$, and remain on until the pool has dropped below a specified *off-trigger elevation,* $elev_{off}$, at which point it turns off. In addition, it is possible to constrain the pump such that when it is triggered to turn on, it must stay on for a minimum run time.

The pump operation can also be constrained to stay on or off for a minimum length of time by setting a minimum run time or minimum rest time. If it is used, once the pump turns on it must remain on the specified minimum run time even if the reservoir pool elevation drops below the trigger elevation to turn the pump off. The only exception is if the pool elevation drops below the intake elevation, then the pump will shut off even though the minimum run time is not satisfied.

So HEC-HMS will check to see if the tailwater or headwater is above the intake.

14.11 Dam Break

Sometimes it is of interest to model a scenario in which there is a dam failure. Two types of dam failure can be modeled in HEC-HMS, overtop and piping. For both types of breach methods, a trigger method, development time, and progression method are used to define when the failure initiates, how long it takes to attain maximum breach opening, and how the breach develops during the development time. Typically dam breaks would be modeled in HEC-HMS only for periodic assessments. At a certain size, RAS would be more meaningful, since it includes the ability to manage a nonlevel pool.

In order to model a dam break, the **Outflow Structures** routing method must be used. Only one dam break can be included in the reservoir.

14.11.1 Trigger Method

There are three methods for triggering the initiation of the failure: elevation, duration at elevation, and specific time. For the **Elevation** method, the breach will begin forming as soon as the reservoir reaches a specified elevation. For the **Duration at Elevation** method, the reservoir elevation must remain at or above a specified elevation for a specified length of time in order to initiate the breach. For the **Specific Time** method, the breach will begin opening at the specified time regardless of the reservoir pool elevation.

Once the breach has been triggered, the development of the breach is determined using a selected progression method over the development time. The **development time** defines the total time (in hours) for the breach to form, from initiation to reaching the maximum breach size.

14.11.2 Progression Method

The progression method determines how the breach grows from initiation to maximum size during the user-specified development time. Current progression method options are: linear, sine wave, and user curve. The **Linear** method causes the breach to grow in equal increments of depth and width from initiation to the

end of the development time.
The **Sine Wave** method causes the breach to grow quickly in the early part of breach development and more slowly as it reaches maximum size. The speed varies over the development time according to the first quarter cycle of a sine wave.
The **User Curve** method allows the modeler to specify a pattern for the breach growth by defining the relationship between percent of the development time versus percent of the maximum breach size.

14.11.3 Overtop Dam Break Method

The overtop dam break is designed to represent failures caused by overtopping of the dam. These failures are most common in earthen dams but may also occur in concrete arch, concrete gravity, or roller compacted dams as well. The failure begins when appreciable amounts of water begin flowing over or around the dam face. The flowing water will begin to erode the face of the dam.
The method begins the failure at a point on the top of the dam (or below the top) and expands it in a trapezoidal shape until it reaches the maximum size. The maximum breach size is defined using the top and bottom elevation, bottom width, and side slopes.
The bottom elevation defines the elevation of the bottom of the trapezoidal opening in the dam face when the breach is fully developed. The bottom width defines the width of the bottom of the trapezoidal opening in the dam face when the breach is fully developed.
Flow through the expanding breach is modeled using the weir flow equations:

Q=CLH3/2
(X)
where:
Q = Discharge over dam breach
C = Discharge coefficient = 1.7 or 1.35
L = bottom width of the dam breach = average side slope x head
H = Upstream energy head above dam breach
The size of the dam breach at each timestep is determined using the trigger, progression, and definition of the maximum dam breach size. The invert elevation is changing, along with the head and the length.

14.11.4 Piping Dam Break

The piping dam break is designed to represent failures caused by piping inside the dam. These failures typically occur only in earthen dams. The failure begins when water naturally seeping through the dam core increases in velocity and quantity enough to begin eroding fine sediments out of the soil matrix. If enough material erodes, a direct piping connection may be established from the reservoir water to the dam face. Once such a piping connection is formed it is almost impossible to stop the dam from failing.
The method begins the failure at a point in the dam face and expands it as a circular opening. When the opening reaches the top of the dam, it continues expanding as a trapezoidal shape. Flow through the circular opening is modeled as orifice flow while in the second stage it is modeled as weir flow.

The piping elevation indicates the point in the dam where the piping failure first begins to form.
The piping coefficient is used to model flow through the piping opening as orifice flow. As such, the coefficient represents energy losses as water moves through the opening.

Q=KA2gH
(4)
in which *Q* = flow rate; *K* = user-specified orifice coefficient; *A* = the cross-sectional area of the culvert, normal to the direction of flow; *H* = total energy head on outlet; and *g* is the gravitational constant.

Once the piping opening reaches the top of dam elevation, it transitions from pressurized piping flow to an open, overtopping breach. Then the outflow is modeled as weir flow, using the top elevation, bottom elevation, bottom width, left slope, and right slope to describe the trapezoidal breach opening that will be the maximum opening in the dam.

14.12 Dam Seepage

Most dams have some water seeping through the face of the dam. The amount of seepage depends on the elevation of water in the dam, the elevation of water in the tailwater, the integrity of the dam itself, and other factors. In some situations, seepage from the pool through the dam and into the tailwater can be a significant source of discharge that must be modeled. Less commonly, water in the main channel downstream may seep through the levee or dam face and enter the pool. Both of these situations can be represented using the dam seepage structure. Only one seepage structure can be added to any reservoir, so all sources and sinks of seepage must be represented collectively.

It is assumed that all reservoir seepage ends up in the river downstream of the reservoir. Seepage into the reservoir is taken from a global source and only will occur when tailwater is lower than the reservoir elevation. When water seeps out of the reservoir, the seepage is automatically taken from the reservoir storage and added to the main tailwater discharge location. This is the mode of seepage when the pool elevation is greater than the tailwater elevation. Seepage into the reservoir happens when the tailwater elevation is higher than the pool elevation. In this mode the appropriate amount of seepage is added to reservoir storage, but it is not subtracted from the tailwater. Dam seepage is also commonly used with pump stations outside levees. One side must be declared to be tailwater.

Currently the only dam seepage method available is Tabular Seepage. This is similar to modeling an "Unknown Spillway". The user gives two elevation-discharge curves.

14.12.1 Tabular Seepage

The tabular seepage method uses an elevation-discharge curve to represent seepage. Usually the elevation-discharge data will be developed through a geotechnical investigation separate from the hydrologic study. A curve may be specified for inflow seepage from the tailwater toward the pool, and a separate curve can be specified for outflow seepage from the pool to the tailwater. The same curve may be selected for both directions if appropriate. If a curve is not selected for one of the seepage directions, then no seepage will be calculated in that direction.

In order to determine which table to use, the pool elevation $elev_{pool}$ is compared to the tailwater elevation $elev_{tw}$. Positive values of the seepage head h_s between the pool and tailwater indicate seepage out of the dam, as long as the pool elevation is above a defined minimum. Negative values of h_s indicate seepage from the tailwater into the pool. Once the direction of seepage is determined, the seepage value is simply a matter of looking up the seepage from the seepage table.

$h_s = elev_{pool} - elev_{tw}$

14.13 Additional Release

In most situations a dam can be properly configured by defining outlet structures such as spillways, uncontrolled outlets, etc. The total outflow from the reservoir can be calculated automatically using the physical properties entered for each of the included structures. However, some reservoirs may have an additional release beyond what is represented by the various physical structures. In many cases this additional release is a schedule of managed releases achieved by operating spillway gates.

Currently the only method for making an additional release is the Gage Release method. The modeler can specify the additional releases based on a gage reading (time series of discharge). This release is subtracted from the reservoir pool during the iterative calculation, in order to include the controlled releases the releases from the other outlet structures are determined.

14.14 Evaporation

Water losses due to evaporation may be an important part of the water balance for a reservoir, especially in dry or desert environments. If evaporation should be captured, the model must use the Outflow Structures routing method (see page 270) with the **elevation-area** storage method. An error message will appear if you attempt to model evaporation without using these routing and storage methods. This provides a reservoir surface area with which evaporation can be determined. An evaporation depth is computed for each time interval and then multiplied by the current surface area.

Currently the only evaporation option is the **Monthly** evaporation method. It can be used to specify a separate evaporation rate for each month of the year, entered as a total depth for the month.

The evaporation depth at timestep t, $evapDepth_t$, is a function of the current month's evaporation depth $evapDepth_{month}$ and the number of timesteps in the current month, $timesteps_{month}$:

$$evapDepth_t = \frac{evapDepth_{month}}{timesteps_{month}}$$

For each timestep, the evaporation volume, $evapVol_t$, is then calculated as the depth of evaporation $evapDepth_t$, multiplied by the current surface area, A_t:

$$evapVol_t = evapDepth_t A_t$$

14.15 References91

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15 Calibration

15.1 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.

15.2 Summary of the Calibration Procedure

In HEC-HMS, the systematic search for the best (optimal) parameter values follows the procedure illustrated in Figure 45. 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 23 and Table 24 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.

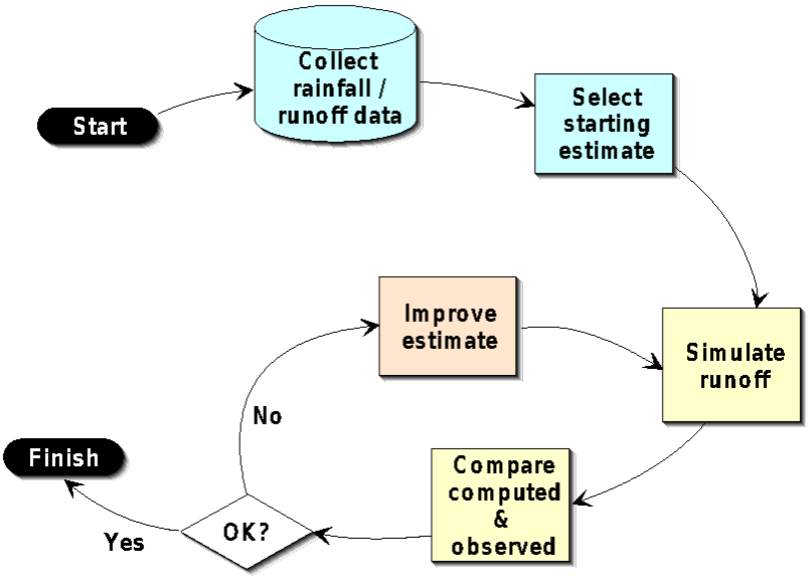


Figure 50.Schematic of calibration procedure.

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 46 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 25.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 26.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.

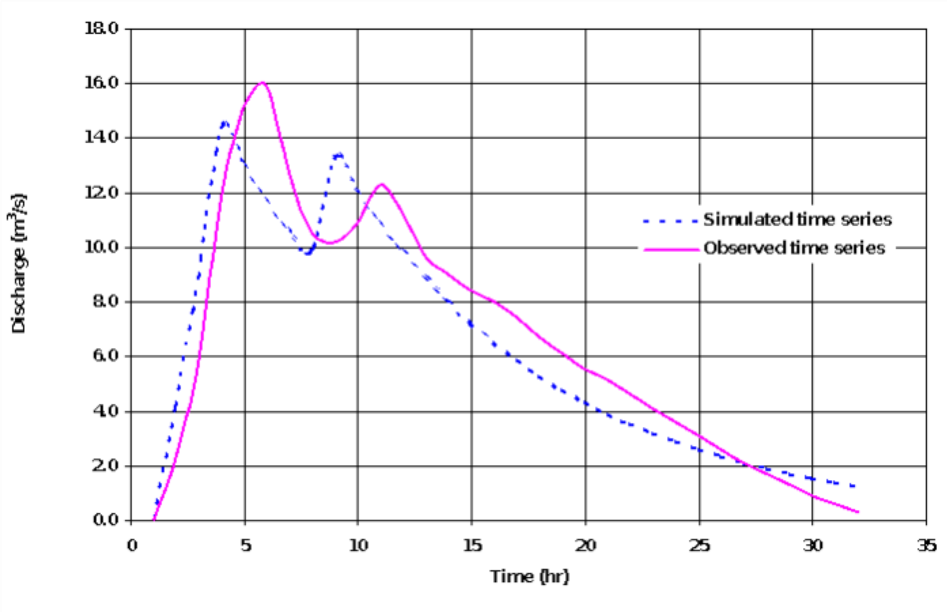


Figure 51.How well does the computed hydrograph "fit"?

15.3 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 25) 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 27.Objective functions for calibration.

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

Note: 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 46. In addition, the program displays a scatter plot, as shown in Figure 47. 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.

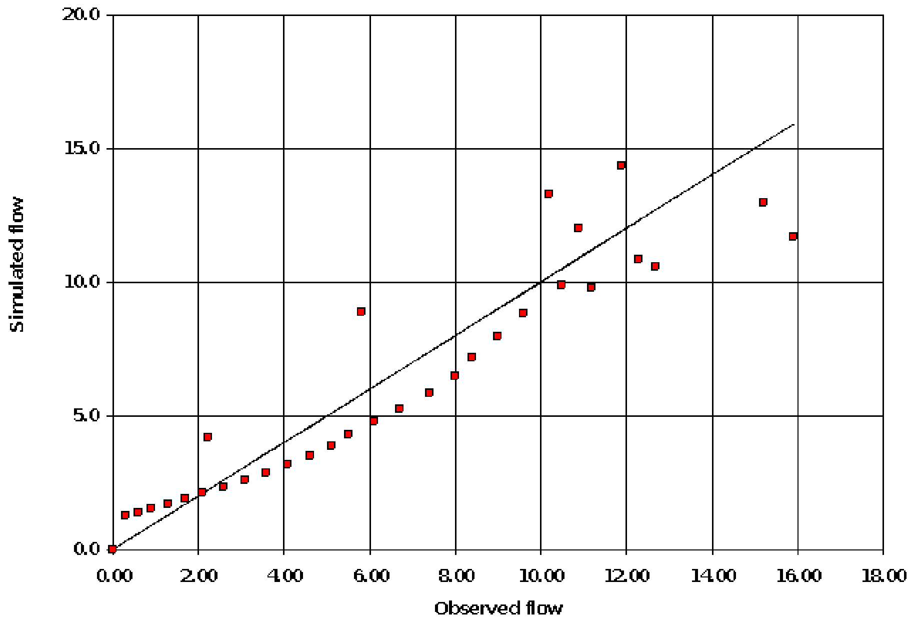


Figure 52.Scatter plot of optimization results.

The program also computes and plots a time series of residuals—differences between computed and observed flows. Figure 48 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.

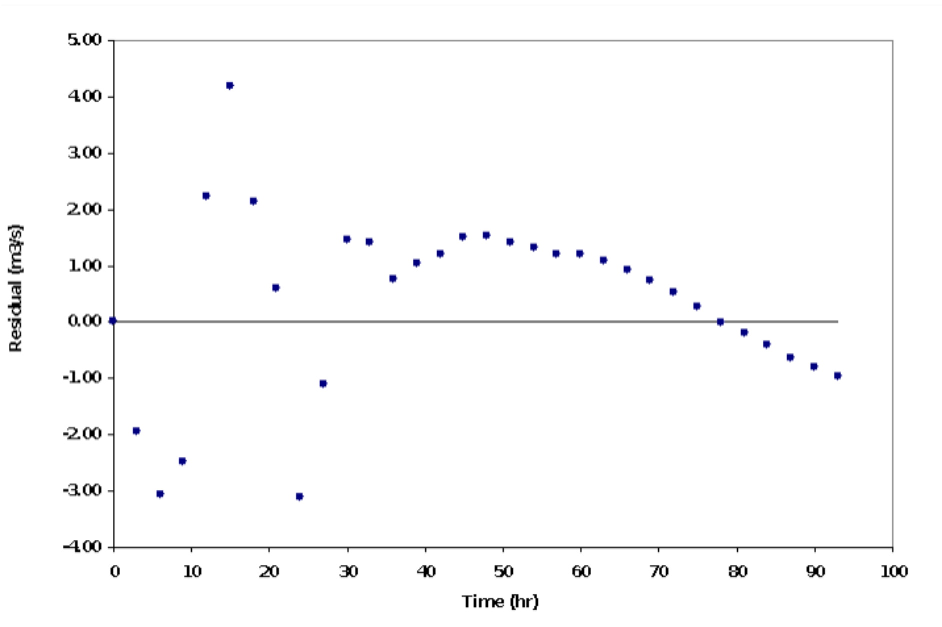


Figure 53.Residual plot of optimization results.

15.4 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 45, 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.

15.4.1 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:

$$x^{k+1} = x^k + \Delta x^k$$

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:

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

in which $f(x^{k+1})$ = the objective function at iteration k; and $\frac{df(x^k)}{dx}$ and $\frac{d^2f(x^k)}{ds^2}$ = 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

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

This equation is rearranged and combined with Equation 106, yielding

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

The program uses a numerical approximation of the derivatives $\frac{df(x^k)}{dx}$ and $\frac{d^2f(x^k)}{dx^2}$ 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} = $0.98x^{k_2}$, and the objective function value is computed for each.
- Differences are computed, yielding Δ_1 = $f(x^{k_1})$ − $f(x^k)$ and Δ_2 = $f(x^{k_2})$ − $f(x^{k_1})$
- The derivative $\frac{df(x^k)}{dx}$ is approximated as Δ_1 , and $\frac{d^2f(x^k)}{dx^2}$ 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:
$\Delta x^k = 0.01Cx^k$
in which C is as shown in Table 26.
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

$$x^{k+2} = 0.7x^k + 0.3x^{k+1}$$

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

Table 28.Coefficients for correction in the univariant gradient search.

$\Delta_2 - \Delta_1$	Δ_1	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 Cp and tp are sought, Cp, is adjusted while holding tp at the initial estimate. Then, the algorithm will adjust tp, holding Cp 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%.

15.4.2 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 49.

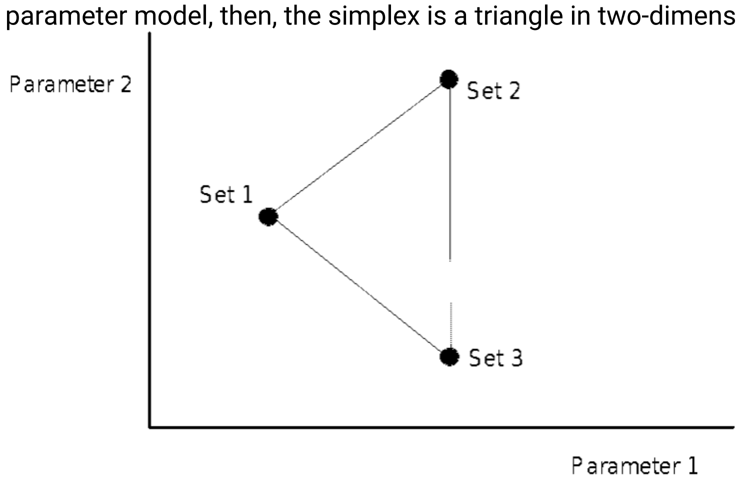
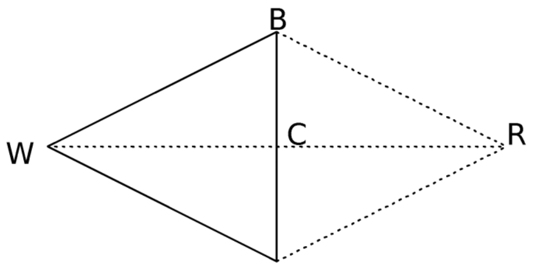


Figure 54.Initial simplex for a 2-parameter model.

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:

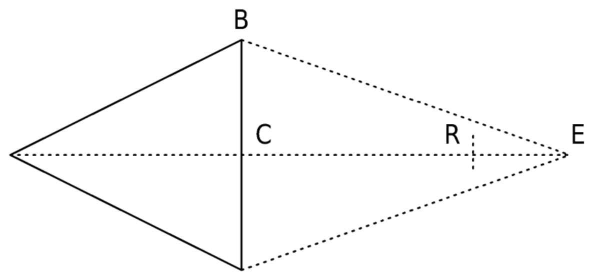
- Comparison. The first step in the evolution is to find the vertex of the simplex that yields the worst (greatest) value of the objective function and the vertex that yields the best (least) value of the objective function. In Figure 50, these are labeled W and B, respectively.
- Reflection. The next step is to find the centroid of all vertices, excluding vertex W; this centroid is labeled C in Figure 50. 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 50.



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

Figure 55.Reflection of a simplex.

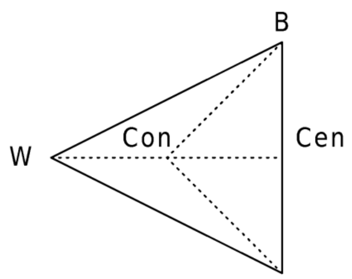
- Expansion. If the parameter set represented by vertex R is better than, or as good as, the best vertex, the algorithm further expands the simplex in the same direction, as illustrated in Figure 51. This defines an expanded vertex, labeled E in the figure. If the expanded vertex is better than the best, 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 56.Expansion of a simplex.

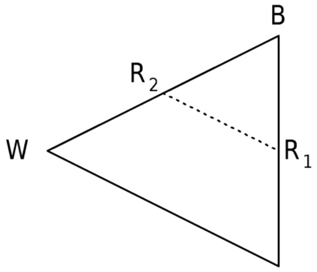
- Contraction. If the reflected vertex is worse than the best vertex, but better than some other vertex (excluding the worst), the simplex is contracted by replacing the worst vertex with the reflected vertex. If the reflected vertex is not better than any other, excluding the worst, the simplex is contracted. This is illustrated in Figure 52. 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 57.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 53.



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

Figure 58.Reduction of a simplex.

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

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

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.

15.5 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 27, 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:

$$Penalty\$ = 2\prod_{i=1}^{n} (x_i - c_i \ | +1)\,\$$$

in which xi = estimate of parameter i; ci = 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 29.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
	Cp	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

15.6 Calibration References

Diskin, M.H. and Simon, E. (1977). "A procedure for the selection of objective functions for hydrologic simulation models." Journal of Hydrology, 34, 129-149.
Stephenson, D. (1979). "Direct optimization of Muskingum routing coefficients." Journal of Hydrology, 41, 161-165.
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.

16 Optimization

17 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					

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	**Average percent impervious area**[2]	**A**	**B**	**C**	**D**
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

Cover description		**Curve numbers for hydrologic soil group**			
Cover type and hydrologic condition	**Average percent impervious area**[2]	**A**	**B**	**C**	**D**
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	83
	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.
Poor: Factors impair infiltration and tend to increase runoff.

Good: 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	Poor		68	79		86		89		
forage for graving.[2]	Fair		49	69		79		84		
	Good		39	61		74		80		
Meadow – continuous grass, protected from			30	58		71		78		

grazing and generally mowed for hay.										
Brush – brush-weed mixture with brush	Poor		48	67		77		83		
the major element.[3]	Fair		35	56		70		77		
	Good		30[4]	48		65		73		
Woods – grass combination (orchard	Poor		57	73		82		86		
or tree farm).[5]	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,			59	74		82		86		
and surrounding lots.										

[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,	Poor	63	77	85	88
greasewood, creosotebush, blackbrush, bursage,	Fair	55	72	81	86
palo verde, mesquite, and cactus.	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.

18 Glossary

This glossary is a collection of definitions from throughout the Technical Reference Manual plus definitions of other pertinent hydrology terms. Many of the definitions herein are from the electronic glossaries available from U.S. Geological Survey[112] and the Bureau of Reclamation[113].

Additional terms commonly used within USACE Flood Risk Management studies can be found here: Key USACE Flood Risk Management Terms[114].

18.1 A

A14: NOAA Atlas 14. A multi-volume document produced by the NWS Hydrometeorological Design Studies Center that contains precipitation-frequency estimates across the United States. Not all areas are covered by a volume of A14, most notably the Northwest.

ACE: annual chance exceedance

AEP: annual exceedance probability

AMS: annual maximum series. A sample containing the largest observation of some variable from each year.

Annual Flood: The maximum peak discharge in a water year.

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.

AORC: Analysis of Record for Calibration, a high-resolution meteorological dataset created by assimilation of multiple input datasources including NEXRAD radar, satellite precipitation estimation, and gage data.

ARF: area reduction function/factor. A means for generalizing the behavior of area-averaged precipitation based on the point maximum intensity.

ARI: average return interval

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.

18.2 B

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

112 https://www.usgs.gov/special-topics/water-science-school/science/water-science-glossary
113 https://www.usbr.gov/library/glossary/
114 https://www.hec.usace.army.mil/publications/TrainingDocuments/TD-40.pdf

flooding of the stream below. The difference between the observed stage and that indicated by the stage-discharge relation, is reported as backwater.

Balanced Hyetograph: a storm temporal pattern in which any nested duration has the same frequency; e.g. the 1-hour, 3-hour, 6-hour, 12-hour, and 24-hour totals all have 1% AEP.

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.

18.3 C

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.

18.4 D

DAD: Depth-area-duration, an idealized way of representing the spatial-temporal pattern of a precipitation event

DDF: Depth-duration-frequency, a generalization of precipitation-frequency analysis results

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.

18.5 E

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.

ERL: equivalent record length. A measure of information content in a regional analysis based on counting the number of independent storms in the regionally-pooled observations. Also referred to as "equivalent independent record length (EIRL)".

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.

18.6 F

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.

Fluvial Flooding: inundation caused by riverine flooding instead of overland flow or infiltration excess (to contrast *pluvial* flooding).

18.7 G

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.

GEV: the generalized extreme value distribution. It is the probability distribution of IID block maxima, and generalizes the three extreme value distributions (Gumbel, Fréchet, Weibull).

GPA: the generalized Pareto distribution. It is the probability distribution of IID excesses of a sufficiently high threshold. **GPD** is also sometimes used.

GPD: see **GPA**

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.

18.8 H

Heterogeneous/heterogeneity: having different properties (or the degree to which the properties are different). May also be called "inhomogeneous/inhomogeneity."

Homogeneous/homogeneity: having the same properties (or the degree to which the properties are similar)

HUC: hydrologic unit code, a unique numeric identifier of watersheds in the United States[115]

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.

18.9 I

IID: independent and identically distributed

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.

Interflow: Rapid subsurface flow through pipes, macropores, and seepage zones in the soil

115 https://nas.er.usgs.gov/hucs.aspx

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.

Isopluvial: Greek for "same rainfall", in the precipitation frequency context, a map showing the precipitation depth for the same AEP and duration everywhere

18.10 K

Kriging: an interpolation method that relies on Gaussian processes to describe the relationship between variables across dimensions. Typical application is in 2-dimensional spatial statistics. Kriging is a complicated topic and this definition does not do it justice.

18.11 L

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.

L-moment: a descriptor of the shape of a sample or population of data using linear combinations of the values in the dataset

LMRD: L-moment ratio diagram. A plot of L-skewness vs. L-kurtosis that can be used for characterizing sample data and probability distributions.

18.12 M

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.

18.13 N

NARR: North American Regional Reanalysis; an NCEP reanalysis product for North America

NCEP: National Centers for Environmental Prediction

Non-stationarity: a sample that has properties that are not constant across a dimension; e.g. a time series with a trend

NWS: National Weather Service

18.14 O

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.

18.15 P

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.

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.

PDS: partial duration series; also called "peaks over threshold." A sample containing all independent observations of some variable greater than a chosen value.

PF: precipitation-frequency

PFDS: Precipitation Frequency Data Server. NOAA/National Weather Service/Hydrometeorological Design Studies Center source for information related to precipitation frequency analysis. https://hdsc.nws.noaa.gov/hdsc/pfds/

Pluvial flooding: inundation caused by precipitation instead of flowing from a river (in contrast to *fluvial* flooding). Can be caused by overland flow or infiltration excess.

PMF: probable maximum flood

PMP: probable maximum precipitation

Point-to-area reduction: accounting for the difference between the maximum intensity of rainfall at a point, and the average intensity over a larger area. Synonyms: depth-area-reduction, area reduction factor

POR: period of record

POT: peaks over threshold (see **PDS**)

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.

18.16 Q

Quasi-Continuous: a hydrologic modeling technique that mimics continuous modeling in an "event mode" by randomly selecting the event date and dependent initial conditions, to capture the full range of variability in hydrologic conditions for design storm modeling. The initial conditions are drawn from a POR continuous hydrologic simulation.

18.17 R

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.

Region/regionalization: a collection of sites grouped together based on similarity that is used to improve estimates for the properties of extremes in that area. In the SWT method, this is typically one of the NWS climate divisions, which is viewed as meteorologically but not necessarily statistically homogeneous

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.

ROI: region of influence. A regionalization method that treats each station as the nucleus of a number of other, homogeneous stations, that form a region. The process is repeated for every station in the study area.

RRFA: regional rainfall-frequency analysis

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.

18.18 S

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.

Site: a location where observations of the hydrometeorological variable of interest are taken

Space-for-time substitution: using collections of similar observations of extremes across a geographic extent to increase the effective number of observations of those extremes

Spatial regression: a regression analysis where the predictor(s) and predictand are linked by being co-located in space

Snow: A form of precipitation composed of ice crystals.

Snow Water Equivalent: the height of water if a snow cover is completely melted, on a corresponding horizontal surface area

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.

SST: stochastic storm transposition

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.

Stationarity: in typical usage, constant with respect to time. More generally, a process does not display trends in any parameter across any dimension. The mean and variance are constant with respect to time (or space, in the case of spatial statistics.)

Station-year: total number of years of record when aggregated over a number of stations. For example, there are 60 station-years in an area where there are two stations, one with 35 years of record, and one with 25.

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.

Storm typing: identifying the causal mechanism behind all rainfall events from a meteorological basis

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.

Sub-region: a subset of sites within a region that offers some refinement of region characteristics, typically used to improve estimates of spatial variability. In the SWT method these are statistically homogeneous collections of sites within a climate division

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.

18.19 T

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.

Tobler's First Law of Geography: “Everything is related to everything else, but near things are more related than distant things.”

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.

18.20 U

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.

18.21 V

Vadose Zone: The portion of the soil profile above the saturation zone.

Validation: The calibrated model, without any further parameter modifications, is used to compute outputs which are compared against observed data for independent events that were not considered during model calibration.

18.22 W

Water Year: A 12-month period during which hydrologic quantities are measured. In the United States, a water year is defined as October 1 through September 30 and is is designated by the calendar year in which it ends and which includes 9 of the 12 months. Thus, the water year ending on 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.