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of Engineers**  
Hydrologic Engineering Center

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# Hydrologic Modeling System HEC-HMS



## Applications Guide

June 2017

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## Applications Guide

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## **Hydrologic Modeling System HEC-HMS, Applications Guide**

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# Table of Contents

List of Figures .....	v
List of Tables .....	vii
Abbreviations .....	ix
Preface .....	xiii
Executive Summary .....	xv

## Chapters

### 1 Introduction

What Studies Does USACE Undertake that Require Watershed and Channel Information? .....	1-1
Study Classification .....	1-1
Study Process Overview .....	1-2
What is the Source of the Required Information? .....	1-3
Analysis of Historical Records .....	1-3
Modeling .....	1-4
What is HEC-HMS and what is its Role? .....	1-4
How Should HEC-HMS be used? .....	1-6
Using the Software .....	1-6
Using the Model .....	1-6
What is in the Rest of this Document? .....	1-10
Are Other Methods Required? .....	1-11
References .....	1-12

### 2 Urban Flooding Studies

Background .....	2-1
Objectives .....	2-1
Authority and Procedural Guidance .....	2-1
Study Procedures .....	2-2
Case Study: Estimating Impacts of Urbanization in the CRS/SRS Watershed .....	2-3
Watershed Description .....	2-3
Decisions Required .....	2-4
Information Required .....	2-5
Spatial and Temporal Extent .....	2-5
Model Selection .....	2-7
Temporal Resolution .....	2-8
Model Calibration and Verification .....	2-9
Application .....	2-15
Sensitivity Analysis .....	2-18
Processing Of Results .....	2-20
Summary .....	2-20
References .....	2-20

### 3 Flood Frequency Studies

Background .....	3-1
Objectives .....	3-1
Authority and Procedural Guidance .....	3-1
Study Procedures .....	3-2

# Table of Contents

## Chapters

<b>3 Flood Frequency Studies (continued)</b>	
Case Study: Estimating Flood Frequency in the CRS/SRS Watershed .....	3-4
Watershed Description .....	3-4
Decisions and Information Required.....	3-4
Model Selection, Temporal Resolution, and Spatial and Temporal Extent.....	3-5
Model Calibration and Verification .....	3-6
Application .....	3-6
Adopting a Frequency Curve.....	3-11
Computing Future Development Frequency Functions.....	3-14
Using Frequency Curves in Project Analysis .....	3-18
Summary .....	3-19
References .....	3-19
<b>4 Flood-Loss Reduction Studies</b>	
Background.....	4-1
Authority and Procedural Guidance.....	4-1
Study Objectives .....	4-2
Study Procedure .....	4-4
Case Study: Evaluation of Inundation-Reduction Benefits in the CRS Watershed .....	4-4
Watershed Description .....	4-4
Required Decisions and Necessary Information .....	4-5
Model Selection, Fitting, and Boundary and Initial Conditions .....	4-6
Application .....	4-7
Processing of Results .....	4-16
Summary .....	4-18
References .....	4-18
<b>5 Flood Warning System Planning Studies</b>	
Background.....	5-1
Overview of Flood Warning Systems.....	5-1
Authority and Procedural Guidance.....	5-3
Study Objectives .....	5-4
Study Procedures.....	5-5
Case Study: Estimating Benefit of a Proposed FWS for an Urban Watershed .....	5-6
Objective of Hydrologic Engineering Study.....	5-6
Decision Required.....	5-7
Watershed Description .....	5-7
Procedure for Estimating Warning Time .....	5-8
Model Selection and Fitting .....	5-10
Application .....	5-10
Results Processing .....	5-16
Summary .....	5-16
References .....	5-16

# Table of Contents

## Chapters

<b>6 Reservoir Spillway Capacity Studies</b>	
Background.....	6-1
Objectives .....	6-1
Extreme Events.....	6-2
Procedural Guidance .....	6-2
Study Procedures.....	6-3
Case Study: PMF Evaluation of Spillway Adequacy for Bonanza	
Reservoir .....	6-3
Watershed and Reservoir Description .....	6-3
Decisions and Information Required.....	6-4
Model Selection and Parameter Estimation.....	6-5
Boundary Condition: PMP Development .....	6-6
Reservoir Model .....	6-8
Initial Conditions.....	6-10
Application .....	6-10
Summary .....	6-12
References .....	6-12
<b>7 Stream Restoration Studies</b>	
Background.....	7-1
Goals of Stream Restoration .....	7-1
Hydrologic Engineering Study Objectives and Outputs.....	7-2
Authority and Procedural Guidance.....	7-3
Case Study: Channel Maintenance along Stirling Branch .....	7-4
Watershed Description.....	7-4
Decisions Required and Information Necessary for Decision Making .....	7-5
Model Selection.....	7-5
Model Fitting and Verification .....	7-6
Boundary Conditions and Initial Conditions .....	7-8
Application .....	7-8
Additional Analysis .....	7-10
Summary .....	7-10
References .....	7-10
<b>8 Surface Erosion and Sediment Routing Studies</b>	
Background.....	8-1
Goals of Erosion and Sediment Control .....	8-1
Surface Erosion and Sediment Routing Study Objectives and	
Outputs .....	8-2
Authority and Procedural Guidance.....	8-3
Study Procedures.....	8-4
Case Study: Estimating Sediment Yield in the Upper North Bosque River	
Watershed .....	8-5
Watershed Description .....	8-5
Decisions Required .....	8-6
Information Required.....	8-7

# Table of Contents

## Chapters

<b>8 Surface Erosion and Sediment Routing Studies (continued)</b>	
Model Selection and Fitting .....	8-8
Boundary Conditions and Initial Conditions .....	8-15
Surface Runoff Model Calibration and Verification .....	8-15
Surface Erosion and Sediment Routing Model Initialization.....	8-17
Surface Erosion and Sediment Routing Model Calibration and Verification .....	8-17
Results and Discussion .....	8-19
Additional Analysis .....	8-22
Summary .....	8-22
References .....	8-23
<b>9 Flood Forecasting Application</b>	
Background.....	9-1
General Process .....	9-1
Authority and Procedure.....	9-5
Forecasting Procedure .....	9-6
Case Study: Flood Forecasting in the Kaskaskia .....	9-7
Watershed Description .....	9-4
Decisions Required .....	9-8
Information Required.....	9-9
Spatial and Temporal Extent .....	9-10
Model Selection.....	9-11
Model Calibration and Verification to Historic Flood Events.....	9-11
Application to Flood Forecasting .....	9-15
Summary .....	9-24
References .....	9-24

# List of Figures

## Figure Number

1	CRS/SRS Watershed.....	2-3
2	Creating a Discharge Gage with Historical Data .....	2-11
3	Adding Observed Flow for Calibration.....	2-11
4	Adding a Parameter to the Calibration Event .....	2-12
5	Calibration Results (10 January 1995) .....	2-13
6	Calibration Results (22 January 1997) .....	2-13
7	Verification of Estimated Parameters (26 January 1997) .....	2-14
8	Routing Reach Parameters.....	2-16
9	CRS Basin Schematic.....	2-17
10	Meteorologic Model for the 0.01 AEP Event.....	2-18
11	Connected Meteorologic Model to Basin Model .....	3-8
12	Selected Appropriate Precipitation Method .....	3-8
13	Input for 0.500 AEP Precipitation Event .....	3-9
14	Watershed Explorer .....	3-10
15	Control Specifications for Hypothetical Events.....	3-11
16	Method 1 Steps to Calibrate Models to Flow-Frequency Function.....	3-15
17	Comparison of Without-Project Flow-Frequency Curves.....	3-16
18	Method 2 Steps to Calibrate Models to Flow-Frequency Function.....	3-17
19	Example of using Adopted Frequency Curves to Assign AEP.....	3-18
20	Complete Basin Model of CRS/SRS Watershed .....	4-6
21	Basin Model for Alternative 1 .....	4-9
22	Component Editor for Specifying Properties of Detention Pond .....	4-9
23	Alter Parameter Values using Global Parameter Editor.....	4-11
24	Conceptual Effect of Alternative 1 on the Flow-Frequency Curve .....	4-12
25	Basin Model for Alternative 2 .....	4-12
26	Inflow-Diversion Function for Diversion Element .....	4-13
27	Sketch of Proposed Diversion Control Structure for Alternative 3 .....	4-14
28	Basin Model for Alternative 3 .....	4-14
29	Copying a Basin Model .....	4-17
30	Flow-Frequency Function with Uncertainty Distribution.....	4-18
31	Components of Flood Response and Energy Preparedness System (©2001 David Ford Consulting, Inc., used with permission) .....	5-1
32	Day Curve.....	5-5
33	Urban Flooding in the Watershed for which FWS is Proposed.....	5-6
34	Warning Time Components .....	5-9
35	Flood Hydrographs at the Critical Location using Entire Precipitation Event .	5-11
36	Precipitation Gage Manager .....	5-13
37	Component Editor for a Precipitation Gage.....	5-13
38	Select Pathname From HEC-DSS File Editor .....	5-14
39	Specified Hyetograph Precipitation Method is Selected .....	5-15
40	TOF versus Peak Flow at the Critical Location.....	5-15
41	Photograph of Bonanza Dam and Spillway (courtesy of California Department of Water Resources).....	6-4
42	PMP Depth-Duration Curve for Bonanza Dam.....	6-7
43	PMP Rainfall Input.....	6-8

# List of Figures

**Figure  
Number**

44	Basin Model for PMP Evaluation.....	6-9
45	Plan View of Proposed Design to Increase Spillway Capacity.....	6-11
46	Summary Results for Bonanza Reservoir .....	6-12
47	Before and After Photos of "Restored" Stream (Fischenich, 1999).....	7-4
48	Illustration of Velocity Profiles with Submerged and Unsubmerged Vegetation (Fischenich, 2000) .....	7-6
49	Component Editor for Reach Element Using the Muskingum Cunge Method..	7-7
50	Basin Model Schematic of Stirling Branch.....	7-9
51	Project Location Map of UNBRW near Hico, TX .....	8-6
52	Percentage of Surface Soil Components in UNBRW (from the SSURGO database).....	8-9
53	Basin Average Erodibility Factors for UNBRW (computed using SSURGO data) .....	8-10
54	Land Uses Types in UNBRW .....	8-11
55	Sediment Model Setup in HEC-HMS .....	8-12
56	Comparison between Measured and Computed Load during Calibration Period.....	8-19
57	Comparison between Measured and Computed Sediment Load during Validation Period.....	8-19
58	Comparison between Measured and Computed Reservoir Trap Efficiency Rate (percent).....	8-20
59	Calibration Results for Subbasin Surface Erosion Rates (ton/mi <sup>2</sup> /yr), Reservoir Trap Efficiency Rates (percent), Reach Erosion Trend (Aggradation/Degradation), and Soil Gradation (percent) at Stream Gage Locations during Calibration Period (1 January 1995 – 31 December 1996).	8-21
60	Forecast Deficit and Constant Loss Parameter Adjustment Editor .....	9-4
61	Kaskaskia Watershed .....	9-8
62	2002 Wet/Typical Calibrated Hydrographs at the Venedy Gage .....	9-13
63	2005 Wet/Typical Calibrated Hydrographs at the Venedy Gage .....	9-14
64	2011 Wet/Typical Calibrated Hydrographs at the Venedy Gage .....	9-14
65	2012 Dry Calibrated Hydrographs at the Venedy Gage .....	9-15
66	Zone Configurations .....	9-16
67	Forecast Initial Baseflow Editor for the Venedy Station Zone .....	9-18
68	Initial Baseflow at the Venedy Station Gage .....	9-19
69	Forecast Reservoirs Editor at the Start Time of the Forecast.....	9-19
70	Carlyle Lake Reservoir Initialization .....	9-20
71	Forecast Parameter Adjustments Editor for Deficit and Constant Loss Parameters.....	9-21
72	Forecast ModClark Transform Slider Bar Adjustment Editor .....	9-21
73	Venedy Station Gage Results Assuming no Forecasted Precipitation.....	9-23
74	Venedy station Gage Results Assuming Forecasted Precipitation .....	9-23

# List of Tables

Table Number		
1	Description of Project Phases .....	1-3
2	Summary of Simulation Methods Included in HEC-HMS .....	1-5
3	Document Contents .....	1-10
4	Other HEC Software that can be used Along with HEC-HMS to Perform a Hydrologic Analysis .....	1-11
5	Subbasin Physical Properties .....	2-6
6	Infiltration Rates by Hydrologic Soil-Cover Groups (inches/hour) .....	2-7
7	Unit Hydrograph Lag and Percent Impervious Estimates .....	2-7
8	Basin $n$ Values for Equation 1 .....	2-8
9	Parameter Estimates from Calibration for the Headwater Basin.....	2-14
10	Subbasin Parameter Estimates.....	2-15
11	Initial Loss Values for Sacramento Area .....	2-16
12	Loss Parameters for the 0.01 AEP Event.....	2-16
13	HEC-HMS Results at Arden Way for 0.01 AEP Event.....	2-17
14	Subbasin Outlets, Location of Flow-Frequency Points .....	3-5
15	Depth-Duration-Frequency (DDF) Functions (inches) .....	3-7
16	Without-Project, Current Development Peak-Flow at Arden Way.....	3-12
17	Alternative Methods for Deriving Flow-Frequency Functions.....	3-12
18	Comparison of Results to Fitted Flow-Frequency Functions and USGS Regression Equations.....	3-13
19	Without-Project, Future Development Peak-Flow at Arden Way .....	3-16
20	Damage Reduction Measures and their Impact .....	4-3
21	Without-Project, Flow Frequency Curve at Arden Way .....	4-8
22	Alternative 1 Flow-Frequency Functions at Arden Way for Current and Future Land Development Conditions.....	4-11
23	Alternative 2 Flow-Frequency Functions at Arden Way for Current and Future Land Development Conditions.....	4-13
24	Flow Diversion at Fulton Avenue .....	4-15
25	Alternative 3 Flow-Frequency Functions at Arden Way for Current and Future Land Development Conditions.....	4-15
26	Alternative 4 Flow-Frequency Functions at Arden Way for Current and Future Land Development Conditions.....	4-16
27	EAD and Inundation Reduction Benefit of Alternatives.....	4-17
28	Sample Calculation of Warning Time .....	5-12
29	Summary of PMP Depth Duration Data Provided by Project Meteorologist.....	6-6
30	Extended PMP Depth-Duration Data for HEC-HMS Input .....	6-8
31	Elevation-Storage Data for Bonanza Reservoir.....	6-9
32	Existing and Proposed Spillway Characteristics for Bonanza Dam .....	6-12
33	Peak Flows of Restoration Alternatives (CFS) .....	7-9
34	Initial Parameters and Methods for Sediment Model.....	8-13
35	Calibrated Parameters and Methods for Hydrology Model.....	8-16
36	Statistics (Mean, SD, ME) of Daily Flow at Five Sampling Sites during Calibration and Validation Periods .....	8-17
37	Calibrated Parameters and Methods for Sediment Model.....	8-18
38	Calibration Events for Various Watershed Conditions.....	9-12



# Abbreviations

A	area
AEP	annual exceedance probability
ALERT	Automated Local Elevation in Real Time
BMP	Best Management Practices
cfs	cubic feet per second
CN	SCS curve number
$C_p$	Snyder peaking coefficient
CRS	Chicken Ranch Slough
DDF	depth-duration-frequency
DEM	Digital Elevation Map
DSS	Hydrologic Engineering Center Data Storage System
DSSUTL	DSS Utility software
du/acre	dwelling units per acre
EAD	expected annual damage
ELV	estimated limiting value
EM	USACE Engineer Manual
EMSI	Denver Lag Equation
EPA	Environmental Protection Agency
ER	USACE Engineer Regulation
FWS	flood warning system
GIS	Geographic Information System
GUI	graphical user interface
HEC	Hydrologic Engineering Center
HEC-DSSVue	HEC Data Storage System Visual Utility Engine
HEC-FDA	Flood Damage Reduction Analysis software
HEC-FIA	Flood Impact Analysis software
HEC-GeoHMS	Geospatial Hydrologic Modeling extension
HEC-HMS	Hydrologic Modeling System software
HEC-RAS	River Analysis System software

# Abbreviations

HEC-ResSim	Reservoir System Simulation software
HEC-SSP	Statistical Software Package
HEMP	hydrologic engineering management plan
HMR	Hydrometeorological Report
IDF	intensity-duration-frequency
L	flow path length
LAR	Lower American River
$L_c$	length to the centroid
MAP	mean areal precipitation
ME	mean error
MUSLE	Modified Universal Soil Loss Equation
NB	net benefit
NED	national economic development
NOAA	National Oceanic and Atmospheric Administration
NPS	non-point source
NRC	Natural Research Council
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
PED	pre-construction engineering and design
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RA	risk analysis
S	slope
SCS	Soil Conservation Service
SD	standard deviation
SMA	soil moisture accounting
SPS	standard project storm
SRS	Strong Ranch Slough
SSURGO	Soil Survey Geographic database

# Abbreviations

TIAER	Texas Institute for Applied Environmental Research
TMDL	total maximum daily loads
TNC	The Nature Conservancy
TOF	time of forecast
TSS	total suspended solids
UH	unit hydrograph
UNBRW	Upper North Bosque River Watershed
U.S.	United States
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USGS	U.S. Geological Survey
WMO	World Meteorological Organization
WRDA	Water Resources Development Act



# Preface

This Applications Guide illustrates application of computer program HEC-HMS (Hydrologic Engineering Center's (HEC) Hydrologic Modeling System) in studies typical of those undertaken by hydrologic engineers of the U.S. Army Corps of Engineers (USACE), including (1) urban flooding studies; (2) flood-frequency studies; (3) flood-loss reduction studies; (4) flood-warning system planning studies; (5) reservoir design studies; (6) environmental studies; (7) surface erosion and sediment routing studies; and (8) flood forecasting applications.

Data for the examples presented herein were adapted from actual studies. However, the data have been modified extensively to illustrate key points. Consequently, no conclusions regarding decisions made in the actual studies should be drawn from the results presented.

Many engineers, computer specialists, student interns, and contractors have contributed to the writing and production of previous editions. Each one has made valuable contributions that enhance the overall success of this Applications Guide. Nevertheless, the completion of this edition was overseen by Matthew J. Fleming (Chief, Hydraulics and Hydrology Technical Division, HEC) while Christopher N. Dunn was Director of HEC.

This Applications Guide was updated using Version 4.2 of HEC-HMS.



# Executive Summary

Hydrologic engineers in USACE offices nationwide support USACE planning, designing, operating, permitting, and regulating activities by providing information about current and future runoff from watersheds, with and without water control features. Computer program HEC-HMS can provide much of that information, including estimates of runoff volumes, peak flow rates, timing of peak flows, and sediment yields. The program provides this information by simulating the behavior of the watershed, its channels, and water-control facilities in the hydrologic system.

The document illustrates application of program HEC-HMS to studies typical of those undertaken by USACE offices, including:

- Urban flooding studies.
- Flood-frequency studies.
- Flood-loss reduction studies.
- Flood-warning system planning studies.
- Reservoir design studies.
- Environmental studies.
- Surface erosion and sediment routing studies
- Flood forecasting applications.

For each category, this document presents an example and illustrates how the following steps can be taken to develop the required information using computer program HEC-HMS:

1. Identify the decisions required.
2. Determine what information is required to make a decision.
3. Determine the appropriate spatial and temporal extent of information required.
4. Identify methods that can provide the information, identify criteria for selecting one of the methods, and select a method.
5. Fit model and verify the fit.
6. Collect/develop boundary conditions and initial conditions appropriate for the application.
7. Apply the model.
8. Do a reality check and analyze sensitivity.
9. Process results to derive required information.



# CHAPTER 1

## Introduction

The mission of USACE is broad, and within the scope of that broad mission, information about watershed and channel behavior must be available for decision making for planning, designing, operating, permitting, and regulating. This chapter identifies studies for which such information is required, it describes conceptually the role that computer program HEC-HMS can play in providing that information, and it shows conceptually how HEC-HMS would be used to provide the information.

### What Studies Does USACE Undertake that Require Watershed and Channel Information?

#### Study Classification

Hydrologic engineers in USACE are called upon to provide information for decision making for:

- **Planning and designing new flood-damage reduction facilities.** These planning studies are commonly undertaken in response to floods that damage property and threaten public safety. The studies seek solutions, both structural and nonstructural, that will reduce the damage and the threat. Hydrologic and hydraulic information forms the basis for design and provides an index for evaluation of candidate damage-reduction plans.
- **Operating and/or evaluating existing hydraulic-conveyance and water-control facilities.** USACE has responsibility for operation of hundreds of reservoirs nationwide for flood control, water supply, hydropower generation, navigation, and fish and wildlife protection. Watershed runoff forecasts provide the information for release decision making at these reservoirs.
- **Preparing for and responding to floods.** Beyond controlling flood waters to reduce damage and protect the public, USACE activities include flood emergency preparedness planning and emergency response. In the first case, a thorough evaluation of flood depths, velocities, and timing is necessary, so that evacuation routes can be identified, temporary housing locations can be found, and other plans can be made. In the second case, forecasts of stage

a few hours or a few days in advance are necessary so that the response plans can be implemented properly.

- **Regulating floodplain activities.** As part of USACE's goal to promote wise use of the nation's floodplains, hydrologic engineers commonly delineate these floodplains to provide information for use regulation. This delineation requires information about watershed runoff, creek and stream stages, and velocities.
- **Restoring or enhancing the environment.** The USACE environmental mission includes ecosystem restoration, environmental stewardship, and radioactive site cleanup. Each of these activities requires information about the hydrology and hydraulics of sensitive sites so that well-informed decisions can be made.

In addition, since passage of the Rivers & Harbors Act of 1899 USACE has been involved in regulating activities in navigable waterways through the granting of permits. Information about flow depths, velocities, and the temporal distribution of water is vital to the decision making for this permitting.

## Study Process Overview

For any of the studies listed above, one of the initial steps is to develop a "blue print" of the study process. EP 1110-2-9 (USACE, 1994b) describes the steps needed in a detailed hydrologic engineering management plan (HEMP) prior to study initiation. A HEMP defines the hydrologic and hydraulic information required to evaluate the national economic development (NED) contribution and to ascertain satisfaction of the environmental-protection and performance standards. It also defines the methods to be used to provide the information, and identifies the institutions responsible for developing and/or employing the methods. From this detailed technical study plan, the time and cost estimates, which are included in the HEMP, can be developed. The HEMP maximizes the likelihood that the study is well planned, provides the information required for proper decision making, and is completed on time and within budget.

The USACE approach to flood studies is to follow a process that involves planning, design, construction, and operation. The sequential phases are described in [Table 1](#). The initial HEMP is prepared at the end of the reconnaissance phase; this defines procedures and estimates resources required for the feasibility phase. At the end of the feasibility phase, a HEMP is prepared to define procedures and estimate resources for the design phase. At the beginning of the feasibility and design phases, a HEMP may also be prepared to define in detail the technical analyses.

**Table 1.** Description of Project Phases

<b>Project Phases</b>	<b>Description</b>
Reconnaissance	This is this first phase. In this phase, alternative plans are formulated and evaluated in a preliminary manner. The goal is to determine if at least one plan exists that has positive net benefit, is likely to satisfy the environmental-protection and performance standards, and is acceptable to local interests. In this phase, the goal is to perform detailed hydrologic engineering and flood damage analyses for the existing without-project condition if possible. If a solution can be identified, and if a local sponsor is willing to share the cost, the search for the recommended plan continues to the second phase.
Feasibility	In this second phase, the set of feasible alternatives is refined and the search narrowed. The plans are nominated with specific locations and sizes of measures and operating policies. Detailed hydrologic and hydraulic studies for all conditions are completed as necessary "... to establish channel capacities, structure configurations, levels of protection, interior flood-control requirements, residual or induced flooding, etc." (ER 1110-2-1150 - USACE, 1999). Then, the economic objective function is evaluated, and satisfaction of the performance and environmental standards tested. Feasible solutions are retained, inferior solutions are abandoned, and the cycle continues. The NED and locally preferred plans are identified from the final array. The process concludes with a recommended plan for design and implementation.
Design	In this phase (also known as the preconstruction engineering and design (PED) stage), necessary design documents, plans, and specifications for implementation of the proposed plan are prepared. These further refine the solution to the point that construction can begin. Engineering during construction permits further refinement of the proposed plan and allows for design of those elements of the plan not initially implemented or constructed. Likewise, the engineering during operations stage permits fine-tuning of operation, maintenance, replacement, and repair decisions.

The contents of a HEMP vary slightly depending on the study phase, but all contain the best estimate of the work to be performed, the methods for doing so, and the associated resources required.

## What is the Source of the Required Information?

### Analysis of Historical Records

In some cases, a record of historical flow or stage can provide all the information needed for the decision making. For example, suppose that the 0.01 annual exceedance probability (AEP) stage at a floodplain location is required for regulating floodplain activities. If a long continuous record of measured stage is available, fitting a statistical distribution to the record (following procedures described in EM 1110-2-1415 - USACE, 1993) and using this fitted distribution to find the stage will provide the information required for the decision making.

## Modeling

Historical records are not often available or are not appropriate for the decision making. The record length may be too short for reliable statistical analysis, the gage may be at a location other than the location of interest, or the data of interest may be something that cannot be measured.

For example, to compute expected annual damage (EAD) with which to compare proposed flood-damage measures in a watershed, runoff peaks are required. But until the measures are implemented and floods occur, no record of peaks can be available. Implementing the measures and waiting to see what impact the changes will actually have is unacceptable, as the benefits of the measures must be determined before decisions can be taken to expend funds to implement the measures.

Similarly, a record of inflow is needed to determine appropriate reservoir releases should a tropical storm alter its course and move over the contributing watershed. But until the rain actually falls and runs off, no record of such inflow will be available. Waiting to observe the inflow is not acceptable, because actions must be taken beforehand to protect the public and property.

In these cases, flow, stage, velocity, and timing must be predicted to provide the required information. This can be achieved with a mathematical model of watershed and channel behavior – a set of equations that relate something unknown and of interest (the model's output) to something known (the model's input). In hydrologic engineering studies, the known input is precipitation or upstream flow and the unknown output is stage, flow, and velocity at a point of interest in the watershed.

## What is HEC-HMS and what is its Role?

HEC-HMS is a numerical model (computer program) that includes a large set of methods to simulate watershed, channel, and water-control structure behavior, thus predicting flow, stage, and timing. The HEC-HMS simulation methods, which are summarized in [Table 2](#), represent:

- **Watershed precipitation and evaporation.** These describe the spatial and temporal distribution of rainfall on and evaporation from a watershed.
- **Runoff volume.** These 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?

**Table 2.** Summary of Simulation Methods Included in HEC-HMS

<b>Category</b>	<b>Method</b>
Precipitation	User-specified hyetograph User-specified gage weighting Inverse-distance-squared gage weighting Gridded precipitation Frequency-based hypothetical storms Standard Project Storm (SPS) for Eastern United States Soil Conservation Service (SCS) hypothetical storm
Evapotranspiration	Monthly Average Priestly-Taylor Gridded Priestly-Taylor
Snowmelt	Temperature Index Gridded Temperature Index
Runoff-volume	Initial and constant SCS curve number (CN) Gridded SCS CN Green and Ampt Exponential Smith Parlange Deficit and constant Gridded deficit and constant rate Soil moisture accounting (SMA) Gridded SMA
Direct-runoff	User-specified unit hydrograph (UH) Clark's UH Snyder's UH SCS UH ModClark Kinematic wave User-specified s-graph
Baseflow	Constant monthly Exponential recession Linear reservoir Nonlinear Boussinesq
Routing	Kinematic wave Lag Modified Puls Muskingum Muskingum-Cunge
Water control structures	Diversion Reservoir/detention pond

- **Direct runoff, including overland flow and interflow.** 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.
- **Baseflow.** These simulate the slow subsurface drainage of water from a hydrologic system into the watershed's channels.
- **Channel flow.** These so-called routing methods simulate one-dimensional open channel flow, thus predicting time series of downstream flow, stage, or velocity, given upstream hydrographs.

The HEC-HMS methods are described in greater detail in the HEC-HMS Technical Reference Manual (HEC, 2000). That manual presents the concepts of each method and the relevant equations that are included. It discusses solution of the equations, and it addresses configuration and calibration of each method.

## How Should HEC-HMS be used?

### Using the Software

The HEC-HMS User's Manual (HEC, 2013b) provides instructions for developing a hydrologic model using computer program HEC-HMS. That manual describes how to install the program on a computer. It also describes how to use the HEC-HMS graphical user interface (GUI) to create and manage analysis projects; create and manage basin models; create and manage meteorologic models; create and manage HEC-HMS control specifications; create and manage simulation runs; calibrate the models; and review the results. However, using HEC-HMS to gain information required for decision making goes far beyond the mouse-clicking and entering data described in that manual.

### Using the Model

To use HEC-HMS to develop information required for planning, designing, operating, permitting, and regulating decision making, the following steps should be taken:

1. **Identify the decisions required.** This is perhaps the most difficult step in a modeling study: deciding exactly what decisions are to be taken as a consequence of a study. In some cases, this may be obvious. For example, in a flood-damage reduction planning study, the decision to be taken is what measures, if any, to implement to reduce damage in a watershed. In other cases, the decision is not as obvious. However, it is seldom the case that the objective of the study is simply to model the watershed or its channels. Instead, the modeling is a source of information that is to be considered in the decision making.
2. **Determine what information is required to make a decision.** After the decision that is to be made has been identified, the information required to make that decision must be determined. This subsequently will guide selection and application of the methods used. For example, in a flood-damage reduction study, the hydrologic engineering information required is an annual maximum flow or stage frequency function at an index location. While infiltration plays some role in estimating this frequency

function, infiltration information itself is not required for the decision making. Thus the emphasis should be on development of a model that provides peak flow and stage information, rather than on development of a model that represents in detail the spatial distribution of infiltration.

3. **Determine the appropriate spatial and temporal extent of information required.** HEC-HMS simulation methods are data driven; that is, they are sufficiently flexible to permit application to watersheds of all sizes for analysis of events long and short, solving the model equations with time steps appropriate for the analysis. The user must select and specify the extent and the resolution for the analysis. For example, a watershed that is thousands of square miles can be analyzed by dividing it into sub-watersheds that are hundreds of square miles, by computing runoff from the individual sub-watersheds, and by combining the resulting hydrographs. A time step of six hours might be appropriate for such an application. However, the methods in HEC-HMS can also be used to compute runoff from a two or three square mile urban watershed, using a five-minute time step. Decisions about the watershed extent, about subdividing the watershed, and about the appropriate time step must be made at the onset of a modeling study to ensure that appropriate methods are selected, data gathered, and parameters estimated, given the level of detail required for decision making.
4. **Identify methods that can provide the information, identify criteria for selecting one of the methods, and select a method.** In some cases, more than one of the alternative methods included in HEC-HMS will provide the information required at the spatial and temporal resolution necessary for wise decision making. For example, to estimate runoff peaks for an urban flooding study, any of the direct runoff methods shown in [Table 2](#) will provide the information required. However, the degree of complexity of those methods varies, as does the amount of data required to estimate method parameters. This should be considered when selecting a method. If the necessary data or other resources are not available to calibrate or apply the method, then it should not be selected, regardless of its academic appeal or reported use elsewhere. Furthermore, the assumptions inherent in a method may preclude its usage. For example, backwater conditions eliminate all routing methods in HEC-HMS except Modified Puls, and may even eliminate that method if significant enough.

Finally, as Loague (1985) point out "... *Predictive hydrologic modeling is normally carried out on a given catchment using a specific model under the supervision of an individual hydrologist. The usefulness of the results depends in large measure on the*

*talents and experience of the hydrologist ...* " This must be weighed when selecting a method from amongst the alternatives. For example, if engineers in a USACE district office have significant experience using Snyder's unit hydrograph, this is a logical choice for new watershed runoff analysis, even though the kinematic wave method might provide the same information.

5. **Fit model and verify the fit.** Each method that is included in HEC-HMS has parameters. The value of each parameter must be specified to fit the model to a particular watershed or channel before the model can be used for estimating runoff or routing hydrographs. Some parameters may be estimated from observation of physical properties of a watershed or channels, while others must be estimated by calibration–trial and error fitting.
6. **Collect/develop boundary conditions and initial conditions appropriate for the application.** Boundary conditions 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 HEC-HMS 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 method. Initial conditions are the known values at which the HEC-HMS equation solvers begin solution of the unsteady flow equations included in the methods. For channel methods, the initial conditions are the initial flows, and for watershed methods, the initial conditions are the initial moisture states in the watershed.

Both initial and boundary conditions must be selected for application of HEC-HMS. This may be a complex, time-consuming task. For example, the boundary condition required for analysis of runoff from a historical storm on a large watershed may be time series of mean areal precipitation (MAP) for subdivision of the watershed. These series would be computed from rainfall observed at gages throughout the watershed, so gage records must be collected, reviewed, reformatted, and processed for each of the gages. Similarly, selection of the initial condition may be a complex task, especially for design applications in which a frequency-based hypothetical storm is used. For example, if the 0.01 AEP flow is required and is to be computed from the 0.01 AEP hypothetical rainfall, the appropriate antecedent moisture condition must be selected. Should a very dry condition be used, or a very wet condition, or some sort of average condition? The choice will certainly have some impact on the model results and hence on the decisions made.

7. **Apply the model.** Here is where HEC-HMS shines as a tool for analysis. With its graphical user interface and strong data management features, the program is easy to apply, and the results are easy to visualize. As noted earlier, the details of applying the program are presented in the User's Manual (HEC, 2013b).
8. **Do a reality check and analyze sensitivity.** After HEC-HMS is applied, the results must be checked to confirm that they are reasonable and consistent with what might be expected. For example, the analyst might compare peaks computed for the 0.01 AEP storm from one watershed to peaks computed with the same storm for other similar watersheds. Similarly, the peaks might be compared with peaks computed with other models. For example, if quantiles can be computed with USGS (U.S. Geological Survey) regional regression equations, the results can be compared with the quantiles computed using HEC-HMS and hypothetical rainfall events. If the results are significantly different, and if no good explanation of this difference is possible, then the results from the HEC-HMS model should be viewed with suspicion, and input and assumptions should be reviewed carefully. (As with any computer program, the quality of the output depends on the quality of the input.)

At this point, the sensitivity of results to assumptions should also be analyzed. For example, suppose that the initial and constant loss rate method is used to compute quantiles for flood-damage reduction planning. In that case, the impact of changes to the initial loss should be investigated. If peaks change significantly as a consequence of small changes, and if this in turn leads to significant changes in the design of alternatives, this sensitivity must be acknowledged, and an effort should be made to reduce the uncertainty in this parameter. Similar analyses should be undertaken for other parameters and for initial conditions.

9. **Process results to derive required information.** In most applications, the results from HEC-HMS must be processed and further analyzed to provide the information required for decision making. For example, if EAD values are required for comparing flood-damage reduction alternatives, the peaks computed for various frequency-based storms must be found in multiple runs of HEC-HMS and must be collected to derive the required flow-frequency function. And if backwater influences the stage associated with the flow, then runs of an open channel flow model may be necessary to develop the necessary stage-frequency function.

ER 1110-2-1464 (USACE, 1994a) provides additional guidance on taking these steps.

## What is in the Rest of this Document?

The remainder of this document illustrates application of program HEC-HMS, following generally the steps described above. [Table 3](#) describes the examples used. Choices made for the examples illustrate use of various program features; they are not intended as guidance for model configuration, calibration, or application. A professional hydrologic engineer should be consulted for such guidance, as that must be tailored to and appropriate for the study at hand.

**Note:** Data for the examples presented herein were adapted from actual studies. However, the data have been modified as necessary to illustrate key points. Consequently no conclusions regarding decisions made in the actual studies should be drawn from the results presented.

**Table 3.** Document Contents

Chapter	Description of Contents
2	This chapter illustrates application of HEC-HMS in analysis of urban flooding. The goal of the study described is to evaluate the impact of changes in land use in a watershed. Historical data are used for calibration, and a frequency-based design rainfall event is the basis of comparison of runoff with and without the development.
3	Flood frequency study. Quantiles - flows of a specified annual exceedance probability - are required for a variety of studies. This chapter illustrates application of HEC-HMS to develop quantiles for an unengaged catchment.
4	Flood-loss reduction studies rely on flood-damage reduction benefit computations, and those require flow-frequency functions. HEC-HMS can be used to develop such functions, and this chapter illustrates that. Functions are derived for the without-project condition and for a damage-reduction alternative that includes a detention and diversion.
5	Flood warning systems can reduce flood damage in many watersheds by increasing warning time. HEC-HMS can provide information required to design and to evaluate such a system. The example in this chapter illustrates how HEC-HMS can be used to estimate the increase in warning time possible with such a system.
6	Capacity studies are undertaken to ensure that reservoir spillways can safely pass the probable maximum storm. This chapter illustrates configuration and application of HEC-HMS to develop the probable maximum flow and route it through a reservoir. An alternative spillway configuration is evaluated.
7	Increased vegetation, often a component of stream restoration projects, affects the hydrograph timing and the stage. HEC-HMS can provide hydrologic information needed to evaluate these projects. This chapter illustrates how HEC-HMS can be used to evaluate different levels of vegetation in a channel.
8	Surface erosion and sediment routing studies. This chapter describes how HEC-HMS can be used to generate important watershed erosion and sediment routing information. The results produced from an HEC-HMS erosion and sedimentation model can be a valuable resource in watershed management.

## Are Other Methods Required?

With the large set of included methods, HEC-HMS can provide information about runoff from historical or hypothetical events, with and without water control or other flood-damage reduction measures in a watershed, with fine or coarse temporal and spatial resolution, for single events or for long periods of record. But even with this flexibility, HEC-HMS will not provide all information required for all planning, designing, operating, permitting, and regulating decision making. For example, HEC-HMS does not include detailed hydraulic unsteady flow channel models, reservoir system simulation models, or flood damage models.

To meet these needs, HEC has developed a suite of software that provides additional capabilities, such as those listed in [Table 4](#). This suite of software can be integrated through a common database. For example, a discharge hydrograph computed with HEC-HMS can be used directly as the upstream boundary condition for HEC-RAS (River Analysis System) or as the reservoir inflow boundary condition for HEC-ResSim (Reservoir System Simulation). Similarly, a discharge-frequency function computed with HEC-HMS (as illustrated in Chapter 3 of this report) can be entered in the HEC-FDA (Flood Damage Reduction Analysis) interface and used subsequently to compute EAD.

**Table 4.** Other HEC Software that can be used Along with HEC-HMS to Perform a Hydrologic Analysis

Software	Description of Capabilities	Reference
HEC-RAS River Analysis System	Solves open-channel flow problems and is generally used to compute stage, velocity, and water surface profiles. Computes steady-flow stage profiles, given steady flow rate, channel geometry, and energy-loss model parameters. Computes unsteady flow, given upstream hydrograph, channel geometry, and energy-loss model parameters.	HEC, 2010a
HEC-FDA Flood Damage Reduction Analysis	Computes EAD, given flow or stage frequency function, flow or stage damage function, levee performance model parameters. Uses risk analysis (RA) methods described in EM 1110-2-1619 (USACE, 1996).	HEC, 2008
HEC-FIA Flood Impact Analysis	Computes post flood urban and agricultural flood damage, based upon continuous evaluation with flow or stage time series.	HEC, 2012
HEC-SSP Statistical Software Package	Performs statistical analysis of hydrologic data. Includes options for computing a Bulletin 17B (IACWD, 1982) analysis of annual peak flow as well as volume-duration data.	HEC, 2010b
HEC-ResSim Reservoir System Simulation	Simulates reservoir system operation, given description of reservoirs and interconnecting channels, reservoir inflow and local flow hydrographs, and reservoir operation rules.	HEC, 2013a

In the examples presented herein, the need for these other programs is identified and their role is described. However, this manual does not describe use of the programs; user's manuals and applications guides for these programs are available currently or are planned.

## References

### **HEC, 2000**

U.S. Army Corps of Engineers, Hydrologic Engineering Center. 2000. *HEC-HMS Hydrologic Modeling System*, Technical Reference Manual, CPD-74B. Hydrologic Engineering Center, Davis, CA.

### **HEC, 2008**

U.S. Army Corps of Engineers, Hydrologic Engineering Center. 2008. *HEC-FDA Flood Damage Reduction Analysis*, User's Manual, Version 1.2.4., CPD-72. Hydrologic Engineering Center, Davis, CA.

### **HEC, 2010a**

U.S. Army Corps of Engineers, Hydrologic Engineering Center. 2010. *HEC-RAS River Analysis System*, User's Manual, Version 4.1, CPD-68. Hydrologic Engineering Center, Davis, CA.

### **HEC, 2010b**

U.S. Army Corps of Engineers, Hydrologic Engineering Center. 2010. *HEC-SSP Statistical Software Package*, User's Manual, Version 2.0, CPD-86. Hydrologic Engineering Center, Davis, CA.

### **HEC, 2012**

U.S. Army Corps of Engineers, Hydrologic Engineering Center. 2012. *HEC-FIA Flood Impact Analysis*, User's Manual, Version 3.1, CPD-82. Hydrologic Engineering Center, Davis, CA.

### **HEC, 2013a**

U.S. Army Corps of Engineers, Hydrologic Engineering Center. 2013. *HEC-ResSim Reservoir System Simulation*, User's Manual, Version 2.2, CPD-81. Hydrologic Engineering Center, Davis, CA.

### **HEC, 2013a**

U.S. Army Corps of Engineers, Hydrologic Engineering Center. 2013. *HEC-HMS Hydrologic Modeling System*, User's Manual, Version 4.0, CPD-74A. Hydrologic Engineering Center, Davis, CA.

### **IACWD, 1982**

Interagency Advisory Committee on Water Data (IACWD). 1982. *Guidelines for Determining Flood Flow Frequency*, Bulletin 17B. Reston, VA, U.S. Geological Survey.

**Loague, 1985**

Loague, K.M., and Freeze, R.A. 1985. *A Comparison of Rainfall-Runoff Modeling Techniques on Small Upland Catchments*. Water Resources Research, AGU, 21(2), 229-248.

**USACE, 1993**

USACE, 1993. *Hydrologic Frequency Analysis*, EM 1110-2-1415. U.S. Army Corps of Engineers, Washington, D.C.

**USACE, 1994a**

USACE, 1994. *Hydrologic Analysis of Watershed Runoff*, ER 1110-2-1464. U.S. Army Corps of Engineers, Washington, D.C.

**USACE, 1994b**

USACE, 1994b. *Hydrologic Engineering Studies Design*, EP 1110-2-9. U.S. Army Corps of Engineers, Washington, D.C.

**USACE, 1996**

USACE, 1996. *Risk-Based Analysis for Flood Damage Reduction Studies*, EM 1110-2-1619. U.S. Army Corps of Engineers, Washington, D.C.

**USACE, 1999**

USACE, 1999. *Engineering and Design for Civil Works Projects*, ER 1110-2-1150. U.S. Army Corps of Engineers, Washington, D.C.



# CHAPTER 2

## Urban Flooding Studies

### Background

#### Objectives

Urban flooding studies are typically undertaken to analyze flooding problems in developed watersheds. Characteristics of these watersheds include:

- Engineered drainage systems throughout.
- Relatively short response times.
- Localized flood damage of properties adjacent to drainage channels.
- The objectives of urban flooding studies are to:
  - Characterize existing flood impacts.
  - Predict impact of future development.
  - Identify solutions to current and future flooding, including controls on land use.

#### Authority and Procedural Guidance

UASCE activities in urban flooding studies are authorized by:

- The Flood Control Act of 1936. This is the general authority under which the Corps is involved in control of floods (and associated damage reduction) on navigable waters or their tributaries. The 1936 Act and the Water Resources Development Act (WRDA) of 1986 stipulate details of Federal participation, including the requirement for benefits that exceed project costs.
- Section 206 of the Flood Control Act of 1960. This authorizes the Corps to provide information, technical planning assistance, and guidance in describing flood hazards and in planning for wise use of floodplains.
- Executive Order 11988. This directed USACE to take action to reduce the hazards and risk associated with floods.

- Section 73 of Public Law 93-251. This endorses USACE's consideration, selection, and implementation of nonstructural flood damage reduction measures.

The following USACE guidance on urban flooding studies includes:

- ER 1105-2-100 (USACE, 2000) provides guidance and describes procedures for all civil works planning studies.
- ER 1165-2-21 (USACE, 1980) defines USACE involvement in urban flood studies. A Federal interest exists for the portion of the watershed where the channel flow exceeds 800 cfs for the ten percent chance flood (0.10 AEP). However, if this criterion is not met, a Federal interest can exist for the portion of the watershed where the 1 percent chance flood exceeds 1,800 cfs.
- EM 1110-2-1413 (USACE, 1987) describes general considerations when evaluating interior areas, commonly found in urban watersheds protected by levees from large bodies of water.
- EM 1110-2-1417 (USACE, 1994a) describes methods, procedures, and general guidance for hydrologic analysis including rainfall, snowmelt, infiltration, transformation, baseflow, and stream routing.
- EP 1110-2-9 (USACE, 1994b) describes the components needed to develop the HEMP for the different phases of a study.

## **Study Procedures**

To meet the objectives of an urban flood study, typically peak flow, total runoff volume, hydrograph timing, peak stage, and floodplain delineations are required. These values are calculated for current development and future development conditions. In general, the procedure to develop a watershed model and calculate these values includes steps such as:

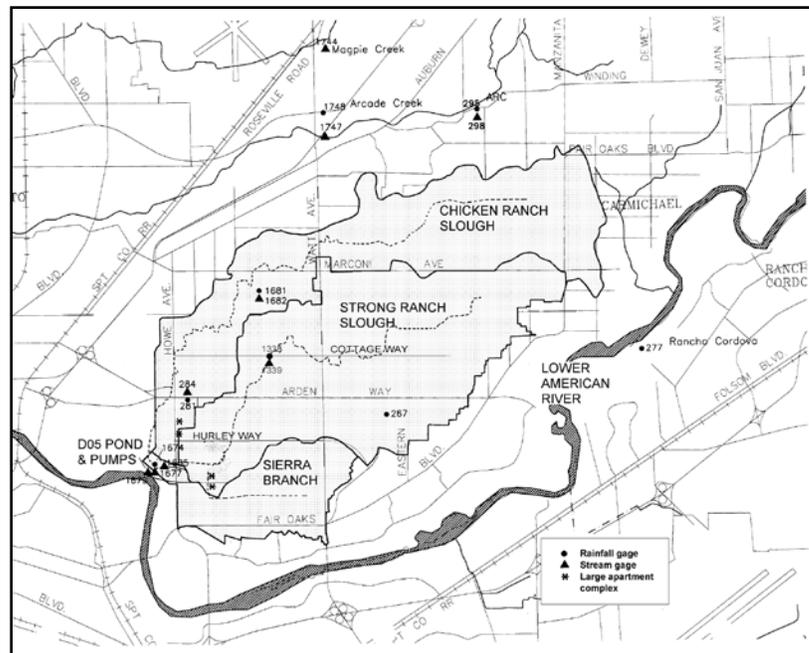
1. Select appropriate methods to represent watershed.
2. Collect watershed data and characteristics.
3. Utilize regional studies and equations to estimate parameter values.
4. Calibrate the model if historical data are available.
5. Exercise the model with various precipitation events, using either historical or hypothetical frequency based events as needed.

6. Analyze results to determine required values such as the peak flow or total runoff volume.
7. Modify the watershed model to reflect changes in the watershed.
8. Re-exercise the model with the same precipitation events.
9. Compare the results to quantify the impact of the watershed changes.
10. The development and modification of a watershed model to analyze the impacts of development is described herein.

## Case Study: Estimating Impacts of Urbanization in the CRS/SRS Watershed

### Watershed Description

The Chicken Ranch Slough and Strong Ranch Slough (CRS/SRS) watershed is an urban watershed of approximately 15 square miles within Sacramento County, in northern California. The watershed and surrounding area are shown in [Figure 1](#). The SRS and Sierra Branch portion of the watershed is 7.1 square miles and the CRS portion is approximately 6.8 square miles. The watershed is developed primarily for residential, commercial, and public uses. The terrain in the watershed is relatively flat. The soil is primarily of sandy loam. It exhibits a high runoff potential.



**Figure 1.** CRS/SRS Watershed

As shown in [Figure 1](#), the watershed is near the Lower American River (LAR). Levees along the LAR protect the watershed from the adverse impacts of high river stages. However, this line of protection restricts the natural flow from CRS and SRS into the LAR. To prevent interior flooding due to this restriction, the D05 interior-drainage facility was constructed. This facility collects interior runoff from the sloughs in a 100 acre-foot pond. From there, the water discharges to the LAR through either gravity outlets or pumps.

The CRS/SRS watershed is a good example of the problem often encountered in an interior watershed. As the LAR rises, the gravity outlets are ineffective at removing water from the pond. Once the LAR rises to the same elevation as the top of the pond, pumping is the only means to remove water from the pond. The pumping station has a total capacity of 1,000 cfs. This is less than the inflow to the pond for even small events. As a consequence, small interior events are likely to cause flooding because water in the pond creates a backwater effect in the channels, thus reducing their flow capacity. Subsequently, flow spills over the channel banks and causes flood damage. For the same storm, if the LAR was low (not restricting the flow through the gravity outlets), the flow would not build up in the pond. The effective flow capacity of the channels would then be greater, thus reducing the likelihood that flood damage would occur.

There are fifteen precipitation and stream gages in and adjacent to the CRS/SRS watershed, their locations are shown in [Figure 1](#). All gages are automatic-reporting ALERT (Automated Local Evaluation in Real Time) gages. The most recent flood events occurred 1995 and 1997. The data from these events will be useful for calibration of the watershed and channel model.

## **Decisions Required**

Located in the headwaters of CRS is a 320-acre (0.5 square mile) undeveloped area. As a result of increasing land values, the owners of the land are petitioning to rezone their land and develop it for new homes and businesses. In order for development to be allowed, the owners must mitigate for any increased runoff caused by the development. In this case, that requirement is imposed by the local authorities. However, a similar requirement is commonly included as a component of the local cooperation agreement for Federally-funded flood-damage-reduction projects. This ensures that future development in a watershed be limited so the protection provided by the project is not compromised. This requirement is especially important in the CRS/SRS watershed because there is already a flood risk near the outlet of the watershed (near the D05 facility).

In the previous reconnaissance phase of this project, a Federal interest in the watershed was identified. Therefore, USACE has now moved on to the feasibility phase. In this phase, USACE has been tasked with answering the questions:

- Will the development of the open area increase the peak runoff in the CRS watershed for the 0.01 AEP event?
- If so, how significant is the increase in flow, volume, and peak stage?

### **Information Required**

To answer the questions above, the following information is required:

- The without-development peak runoff for the selected event.
- The with-development peak runoff for the selected event.

To provide that information, USACE will use a watershed model to compute the peak flow for the different watershed conditions; HEC-HMS will be used. To develop the rainfall-runoff relationship, information on the watershed will need to be collected, such as:

- Soil types and infiltration rates.
- Land use characteristics and the percent of impervious area due to development.
- Physical characteristics of the watershed including lengths and slopes.
- Local precipitation patterns.
- Drainage patterns of the study area.
- Drainage channel geometry and conditions.

For this study, the information required was found using results of previous drainage studies in the area, USGS topographic and soils maps, and field investigations.

### **Spatial and Temporal Extent**

The study team is interested in evaluating the increase in runoff from CRS only. So, the portion of the watershed that contributes flow to SRS will not be analyzed in this phase of the study. In the reconnaissance phase, the study team identified the portion of the CRS watershed downstream of Arden Way as being influenced by backwater from the D05 pond. The flow in this lower portion of the watershed is a function of both the channel flow and downstream channel stage. So, this lower portion will also not be included in this phase of the study. Therefore, the study area

for this phase will be the portion of the watershed that contributes flow to CRS upstream of Arden Way.

Now that the study area has been defined, the next step is to use the information collected to divide the study area into subbasins. By doing so, the analyst will be able to compute the flow at critical locations along CRS. To delineate the subbasins and measure the physical parameters of the watershed, the USGS quadrangle map (1:24000 scale) of the watershed was used.

If a detailed digital elevation model (DEM) were available, the analyst could use the HEC-GeoHMS (Geospatial Hydrologic Modeling Extension) tools to delineate the subbasins, establish the flow paths, and calculate physical parameters of the watershed (such as length, centroid location, and average slope). However, the best DEM available for the watershed is a 30-meter DEM available from the USGS. (A DEM is a grid-cell representation of the topography. A 30-meter DEM is comprised of grid cells measuring 30-meters on each side. Each grid cell has a single associated elevation for its entire area). In this case, the topographic data source of the DEM is the same as the USGS quadrangle map. However, the quadrangle map provides contour lines that offer an additional degree of refinement that the DEM does not provide. This additional refinement is useful for flat terrain and for smaller watersheds. If the watersheds were larger and located in a hilly area where there was significant relief, the 30-meter DEM may be useful for a feasibility-level study.

Because gage data from historical events were available, the headwater subbasin was delineated such that the outlet point was at the Stream Gage 1682, located at Corabel Lane. These data will be useful in the calibration of the headwater subbasin in the watershed model. The study area was further delineated near points where flow-frequency data may be useful for future planning, at Fulton Avenue and at Arden Way. Once the subbasins were established, the analyst measured the areas ( $A$ ), slopes ( $S$ ), flow path length ( $L$ ), and length to the centroid ( $L_c$ ), from the topographic maps. These are watershed properties that are useful for estimation of the model parameters. The values are included in [Table 5](#).

**Table 5.** Subbasin Physical Properties

Description	ID	A (square mile)	S (feet/mile)	L (mile)	$L_c$ (mile)
CRS upstream of Corabel gage	COR	4.22	15.84	4.43	2.24
CRS downstream of Corabel gage, upstream of Fulton Avenue	FUL	0.30	12.67	0.61	0.07
CRS downstream of Fulton Avenue, upstream of Arden Way	ARD	1.00	11.62	1.9	0.67

## Model Selection

Once the watershed data were collected and the spatial and temporal extents had been determined, the analyst began constructing the HEC-HMS model. As shown in [Table 2](#), several methods are available for runoff-volume, direct-runoff, and channel routing. In all cases, two or more of the methods would work for this analysis.

**Infiltration.** The analyst chose the initial and constant-rate runoff-volume method. It is widely used in the Sacramento area. Regional studies have been conducted for estimating the constant loss rate. The studies, based upon calibration of models of gaged watersheds, have related loss rates to soil type and land use. Surveys of development in the region provide estimates of percent of directly impervious area as functions of land use. [Table 6](#) is an excerpt of the results of those studies. Other jurisdictions have similar results available. [Table 7](#) lists the estimates of percent of directly connected impervious area for CRS watershed.

**Table 6.** Infiltration Rates by Hydrologic Soil-Cover Groups (inches/hour)

Land Use	Percent Directly Impervious	Soil Group		
		B	C	D
Commercial, offices	90	0.16	0.08	0.05
Residential: 4-6 du/acre	40	0.18	0.10	0.07
Residential: 3-4 du/acre	30	0.18	0.10	0.07

**Table 7.** Unit Hydrograph Lag and Percent Impervious Estimates

Description	Identifier	Estimated Lag (hr)	Percent Directly Impervious
CRS upstream of Corabel gage	COR	1.78	50
CRS downstream of Corabel gage, upstream of Fulton Avenue	FUL	0.22	60
CRS downstream of Fulton Avenue, upstream of Arden Way	ARD	0.75	50

Other loss methods could have been selected, such as the SCS curve number method or Green and Ampt. Because this analysis considers only a single precipitation event, a soil moisture accounting model designed for continuous simulation would be less useful. Those models would require additional parameter estimates and would not help to answer the questions any better.

**Direct-runoff transform.** The analyst used Snyder's unit hydrograph direct-runoff transform method. This method is widely used in the Sacramento area. As with the loss method, regression studies have been conducted in Sacramento to estimate the lag of watersheds as a function of watershed properties. The regression equation is:

$$T_{lag} = 1560n \left( \frac{LL_c}{S^{0.5}} \right)^{0.33} \quad (1)$$

where:

- $T_{lag}$  = Snyder's standard lag, in minutes
- $S$  = watershed slope, in feet/mile
- $L$  = length of longest watercourse, in miles
- $L_c$  = length along longest watercourse to centroid, in miles
- $N$  = basin  $n$  coefficient

The basin  $n$  coefficient is a function of the percent imperviousness and the land use of the watershed. Table 8 is an excerpt of study results that estimated basin  $n$  values in the Sacramento area. Similar tables and equations are available for other jurisdictions. The lag value from Equation 1 is virtually the same as the value for the U.S. Bureau of Reclamation's (USBR) dimensionless unit hydrograph for urban basins (Cudworth, 1989). Using Equation 1, the lag was estimated; values are shown in Table 7.

**Table 8.** Basin  $n$  Values for Equation 1

Basin Land Use	Channelization Description	
	Developed	Undeveloped
Commercial, offices	0.031	0.070
Residential: 4-6 du/acre	0.042	0.084
Residential: 3-4 du/acre	0.046	0.088

Because the headwater basin is gaged, calibration can be used to estimate the Snyder peaking coefficient,  $C_p$ . During the calibration process, refinements to the lag estimate can be made as well.

**Baseflow.** Baseflow was not included in this analysis. It is not critical in most urban watersheds.

**Routing.** The analyst used the Muskingum-Cunge channel routing method because channel geometry and roughness values were available from previous studies. A primary advantage of the method is that it is physically based, which is useful because there are no downstream data available for calibration. If the study area was defined such that it extended to the D05 pond, the modified Puls method would have been used to model the portion of the CRS channel influenced by backwater from the pond.

## Temporal Resolution

The analyst needed to decide upon a temporal resolution for the analysis. Decisions required include selection of the time step to use and the

hypothetical precipitation event duration. In earlier watershed programs, the selection of the time step required was more critical due to array limitations and program computation time. These considerations are no longer needed when using HEC-HMS on a modern computer for a short duration storm. The analyst could use a one-minute time step; however this may provide unnecessary resolution. However, if the program were used for longer duration events or for continuous simulation, a larger time step would prevent excess data and would reduce computation time. To find the upper limit of an appropriate time step, the analyst must ensure that the peak of the hydrograph is captured. A time step that yields between five to ten points on the rising limb of the unit hydrograph for each subbasin is usually adequate. Using the approximate relationship that the lag time equals sixty percent of the time of concentration from EM 1110-2-1417 (USACE, 1994a), the analyst computed the time of concentration for each basin (based upon the lag time calculated with the regression equation) and divided the minimum of these values by ten points. This yielded a minimum approximate time step of two minutes, as follows:

$$(0.22 \text{ hrs} \times 60 \text{ min/hr}) / (0.6 \times 10 \text{ points on rising limb}) = 2 \text{ min time step} \quad (2)$$

The most common duration for hypothetical events in urban areas is 24 hours. The National Weather Service (NWS) found that most runoff-producing storms in the contiguous United States are greater than twelve hours (NWS, 1972). It is important that the storm duration is long enough that the entire watershed contributes to the runoff. This means it must be greater than the sum of the time it takes to satisfy the initial loss and the time of concentration. A general estimate for this time is four times the time of concentration of the watershed. Using this estimate yielded a twelve-hour event. However, the analyst decided that the time of concentration was likely underestimated because the headwater subbasin had a large drainage length in proportion to its area, so a larger 24-hour event was selected. If after calibration, the analyst's assumption proved incorrect and the lag was not underestimated, the analyst would change back to the 12-hour event. An alternative method to selecting the storm duration is to use a variety of storm durations with the completed model. Select the storm with the greatest peak flow.

## Model Calibration and Verification

Based upon the methods selected, the following parameters are required:

- Initial and constant loss rates and percent directly connected impervious area for the runoff-volume method.
- Lag time and peaking coefficient for the runoff transform.

- Roughness values for the channel routing method.

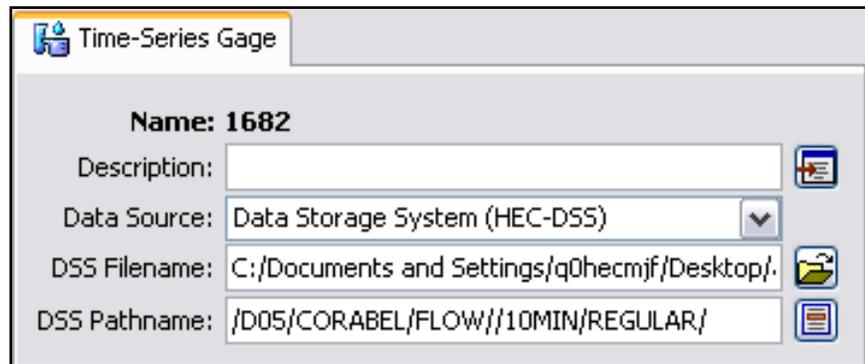
In addition, channel properties such as reach length, energy slope, and channel geometry need to be measured for the channel routing method. The lag time and percent impervious area were estimated as described above. The initial loss, constant loss rate, and peaking coefficient will be estimated using calibration. The initial estimate for the constant loss rate is based upon regional relationships. It is 0.07 inch/hour. Because the watershed is developed and has a high percent of impervious area, the runoff hydrograph is expected to rise and fall over a short period of time. As an initial estimate for the peaking coefficient, the upper limit suggested by the HMS Technical Reference Manual (HEC, 2000) of 0.8 was used.

The magnitude and AEP of historical events used for calibration should be consistent with the intended application of the model. Three significant events have occurred since the installation of the gages in the CRS/SRS watershed. The events are:

- January 10, 1995. This is about a 0.04 to 0.01 AEP event.
- January 22, 1997. This is about a 0.10 to 0.04 AEP event.
- January 26, 1997. This is about a 0.20 to 0.04 AEP event.

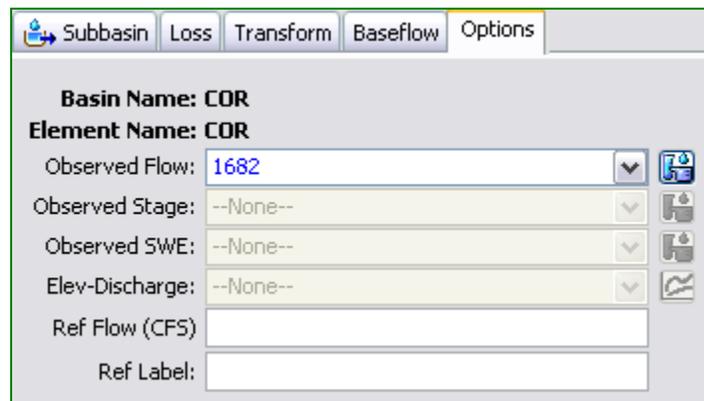
The first two of these events were used to refine the parameter values and the third was used to verify the final values. The analyst used the HEC-HMS **Optimization Manager** for the parameter estimation. To do so, the analyst:

1. Created a new **Basin Model** with a single **Subbasin** for COR, the subbasin to be used for the calibration and verification process.
2. Edited the subbasin to select the appropriate methods for **Loss**, **Transform**, and **Baseflow**, and entered the initial estimates for each method.
3. Added the observed flow by selecting the **Time Series Data Manager** from the **Components** menu. Then the analyst selected the **Discharge Gages** data type and clicked **New**. In this case, the observed values were in DSS (HEC's Data Storage System) format, so in the **Component Editor** the **Data Storage System (HEC-DSS)** was selected as the **Data Source**, as shown in [Figure 2](#). Then the DSS filename and pathname were selected. Additional instructions on adding a gage are included in the HEC-HMS User's Manual (HEC, 2013).
4. Associated the observed flow gage with the subbasin element. First the analyst selected the subbasin in the basin model map and



**Figure 2.** Creating a Discharge Gage with Historical Data

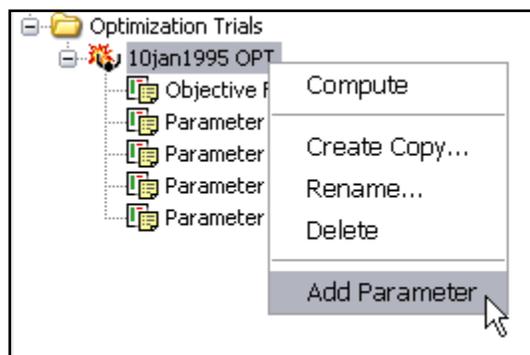
5. then selected the **Options** tab in the **Component Editor**. The appropriate gage was selected, as shown in [Figure 3](#).



**Figure 3.** Adding Observed Flow for Calibration

6. Created a new **Meteorologic Model** for the historical event. To do so, gaged precipitation data were entered. The **Meteorologic Model** used the user-specified gage weighting option. Gage weights for the recording ALERT gages were determined with Thiessen polygons. Refer to the HEC-HMS User's Manual (HEC, 2013) for instructions on creating a **Meteorologic Model**.
7. Created new **Control Specifications** for the historical event. To do so, specify the **Time Interval** and the starting and ending dates and times. Refer to the HEC-HMS User's Manual (HEC, 2013) for instructions on creating control specifications.
8. Created a new **Simulation Run** by selecting the **Create Simulation Run** option from the **Compute** menu. The simulation run must be created before an optimization trial can be created. The analyst computed the simulation run to make sure all model parameters were entered.

9. Created a new **Optimization Run** by selecting **Create Optimization Trial** option from the **Compute** menu.
10. Navigated to the **Optimization Trial** by selecting the **Watershed Explorer, Compute Tab** and expanding the **Optimization Trials** folder. Parameters to be included in the 10 January 10 1995 calibration event were the Snyder time to peak, Snyder peaking coefficient, initial loss, and constant loss rate. A parameter is added to the optimization trial by placing the mouse on top of the optimization trial name and clicking the right mouse button. Then select the **Add Parameter** option, as shown in [Figure 4](#). Additional instructions are included in the HEC-HMS User's Manual (HEC, 2013).



**Figure 4.** Adding a Parameter to Calibration Event

11. In the **Component Editor**, the analyst set the optimization trail time window to correspond with the rising and falling limb of the primary runoff hydrograph. This allowed the program to calibrate to the flood hydrograph and focus the optimization function on matching the peak flow.
12. Once the initial parameter values were specified, click **Optimiz** to begin the computations.
13. Studied the plots of the results, revised initial estimates as needed and repeat Step 11. The results from several iterations of adjusting the time window and fixing different parameters are shown in [Figure 5](#). The computed hydrograph appears to track with the observed flows. However, the computed peak flow is approximately ten percent less than the observed peak flow.

This process was repeated for the 22 January 1997 event. The calibration results for the 22 January 1997 event are shown in [Figure 6](#). This plot shows that the computed hydrograph matches well with the observed flows for that event, especially the peak flows.

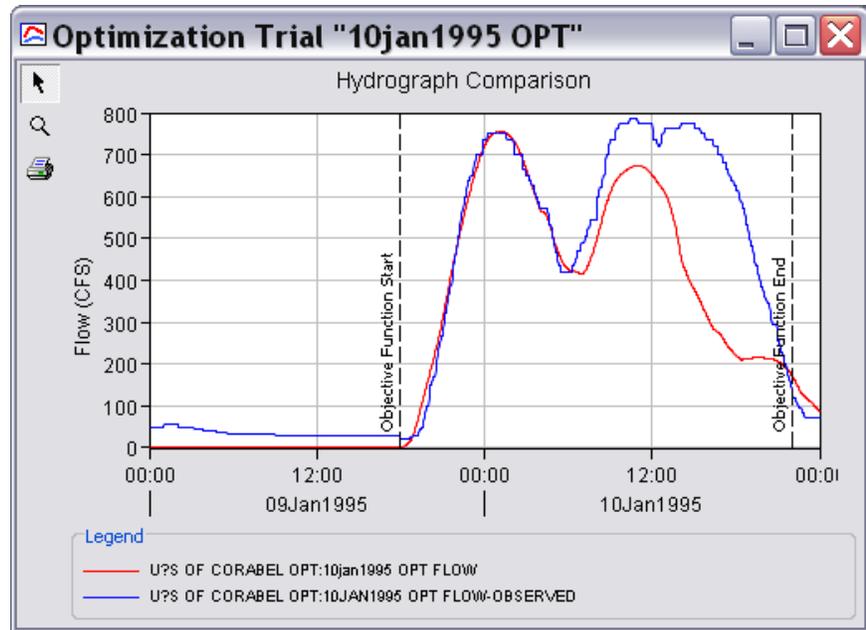


Figure 5. Calibration Results (10 January 1995)

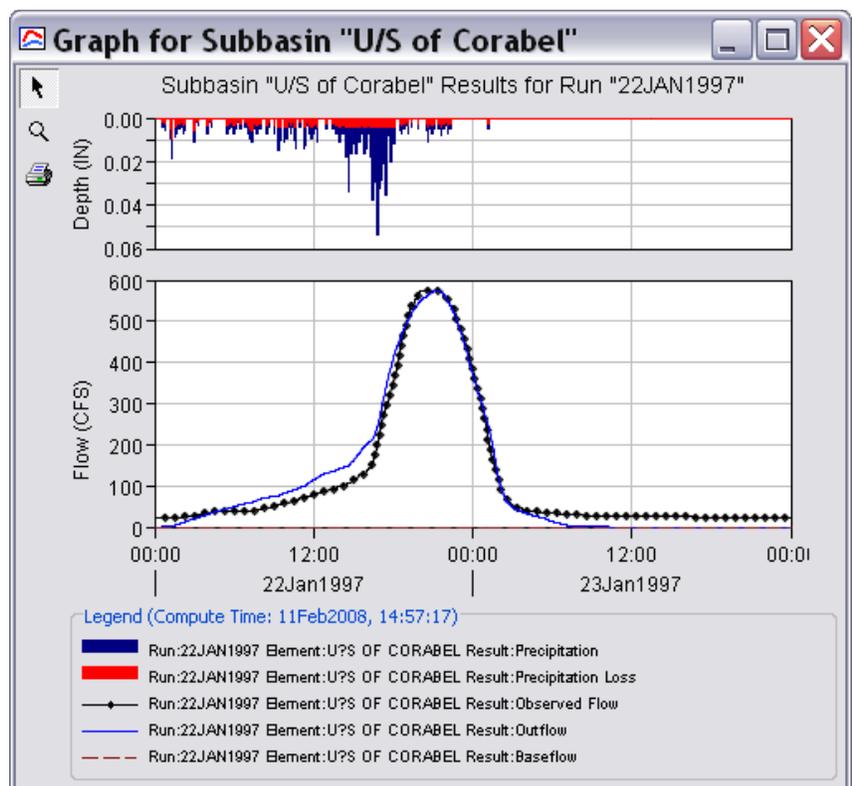


Figure 6. Calibration Results (22 January 1997)

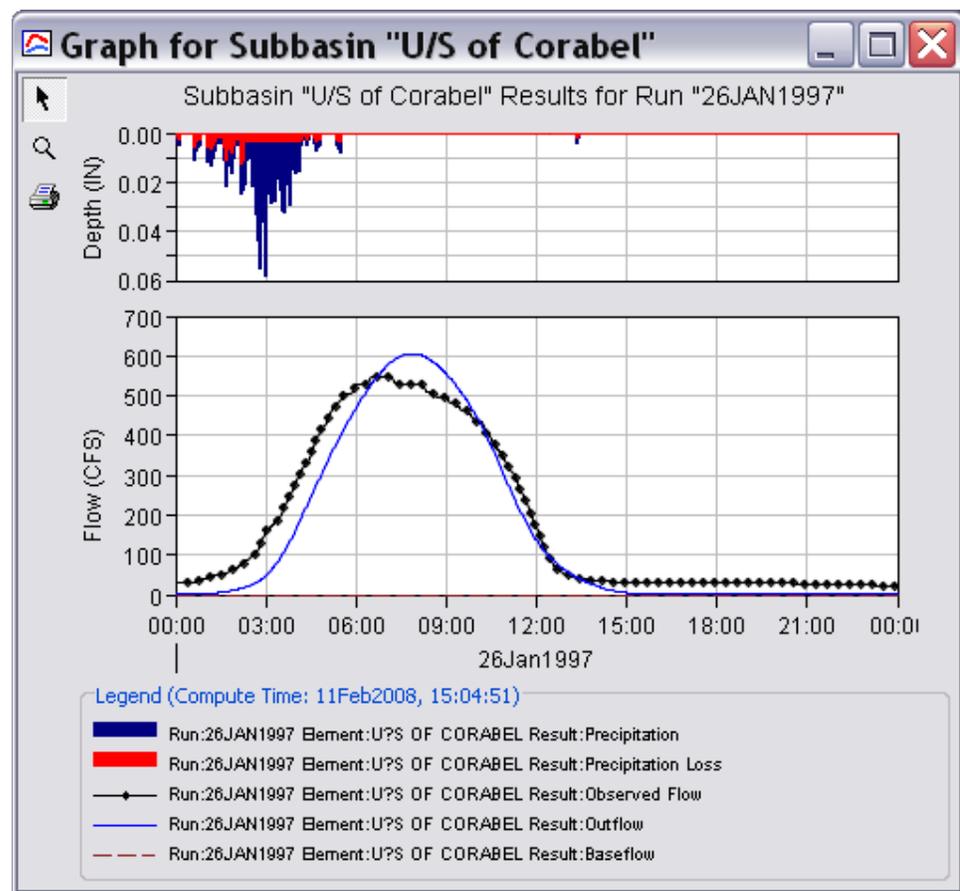
The parameter estimates resulting from the calibration to the January 10, 1995 event and the 22 January 1997 event are summarized in [Table 9](#). The values were averaged and verified using the observed precipitation and flow data for the 26 January 1997 event. To do so, the average lag

**Table 9.** Parameter Estimates from Calibration for the Headwater Basin

Calibration Event	Snyder Lag (hr)	Snyder $C_p$
10 January 1995	5.42	0.68
22 January 1997	4.68	1.00
26 January 1997	4.55	1.00
<b>Final average value</b>	<b>4.88</b>	<b>0.89</b>

and peaking value were input to the **Basin Model** and the **Optimization Trial** was computed again. This time, only the loss rate parameters were adjusted as specified in Step 9 above.

The results from the verification process are shown in [Figure 7](#). Because the rising limb of the observed data occurs earlier than the rising limb of the computed data, the analyst reasoned that the Snyder lag value may be

**Figure 7.** Verification of Estimated Parameters (26 January 1997)

too great. The lag value was added to the **Parameter** list in the **Optimization Trial** for the 26 January 1997 event, and the optimization process was repeated. HEC-HMS computed a value of 4.55 hours, which was similar to the parameter value computed for the 22 January 1997 event.

The average parameter estimates from the first two events did not compare well with the third event. Consequently, the analyst averaged the parameter values from all three events. By doing so, all three events are incorporated into the calibration of the model parameters. This provided the estimates shown in Table 9. The averages of the three values were used to represent the existing condition. However, the analyst may have chosen not to weight the parameter values evenly. Based on the quality of precipitation data, magnitude of the event, or other factors, more weight may be given to a particular historical event.

The Snyder lag values for subbasins FUL and ARD were adjusted from the values predicted with Equation 1 consistent with the calibration. The logic followed is that Equation 1, when compared to the calibration results, under predicts the lag for subbasins in the CRS watershed. By adjusting the parameters, the analyst fits the equation to basins found in this watershed. The resulting values are included in Table 10. The Snyder peaking coefficient,  $C_p$ , is usually taken as a regional value. As the subbasins are similar in slope and land use, the calibrated value was used for the other two subbasins.

**Table 10.** Subbasin Parameter Estimates

Identifier	Adjusted Snyder	
	Lag (hr)	$C_p$
COR	4.88	0.89
FUL	0.60	0.89
ARD	2.06	0.89

The channel properties and parameter values needed for the Muskingum-Cunge routing method must be defined also. The reach length, energy slope, and cross section geometry were estimated from available maps and survey data. The Manning's roughness parameter was estimated using published tables of values (Barnes, 1967). The Manning's roughness value could be refined through calibration if reliable gage data were available. There is a downstream gage at Arden Way. However, due to the backwater conditions from the D05 pond, the assumption of a single relationship between stage and flow is not appropriate. Further, the observed stages at the gage are influenced by a variety of other downstream factors such as pump operation and commingled water from SRS. Figure 8 shows the values used for the routing reach that extends from Fulton Avenue to Arden Way.

## Application

Once the without-development condition parameters were established, the analyst was ready to complete the HEC-HMS input and produce the information needed for decision making. For the comparison of land use conditions, the analyst used the 0.01 AEP storm event to estimate the 0.01

AEP flood. This is a standard procedure often used by the local authorities for evaluating land use changes.

**Figure 8.** Routing Reach Parameters

The initial loss values estimated during calibration were storm specific. The initial loss values used for hypothetical events are based upon studies in the Sacramento area. Values for a range of hypothetical events have been estimated and are shown in [Table 11](#). Other jurisdictions may have similar tables.

**Table 11.** Initial Loss Values for the Sacramento Area

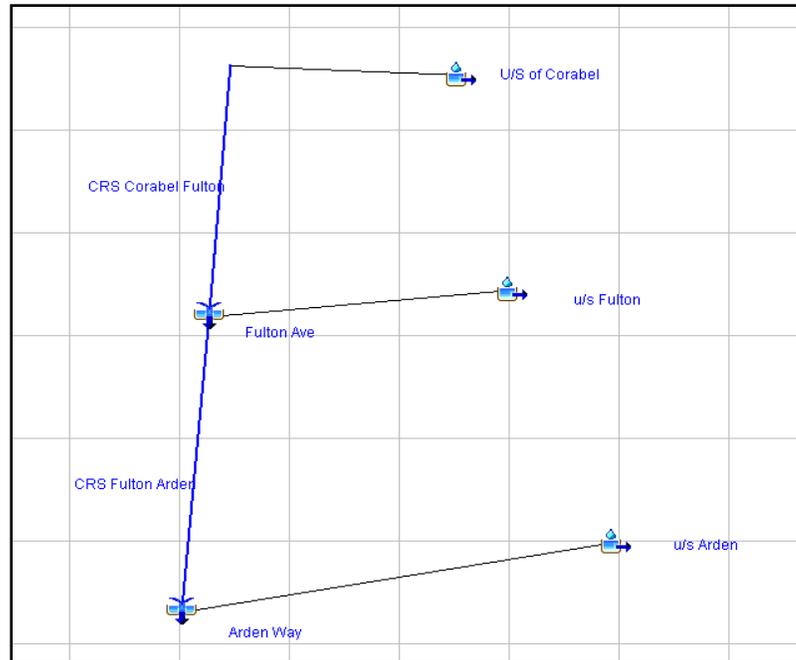
AEP	Initial Loss (in)
0.500	0.40
0.200	0.25
0.100	0.20
0.040	0.15
0.020	0.12
0.010	0.10
0.004	0.08
0.002	0.06

Calibration showed that the constant loss rate, which is a function of the soil characteristics and land use, is under predicted by the regional studies for the CRS watershed. The calibrated value will be used. The loss parameters to be used in the analysis are included in [Table 12](#). The values were added to the basin model.

**Table 12.** Loss Parameters for the 0.01 AEP Event

Subbasin ID	Initial Loss (in)	Constant Loss (in/hr)
COR	0.10	0.23
FUL	0.10	0.23
ARD	0.10	0.23

Adding the routing reaches and ungaged subbasins, as shown in Figure 9, completed the input. Steps followed to complete the **Basin Model** are included in the HEC-HMS User's Manual (HEC, 2013). New **Meteorologic Models** and **Control Specifications** were added.



**Figure 9.** CRS Basin Schematic

In order to complete the **Meteorologic Model** for the 0.01 AEP event, as shown in Figure 10, the analyst used depths from locally-developed depth-duration-frequency (DDF) functions. The DDF functions are based upon data from a NWS gage with a long period of record.

Once completed, the analyst computed a simulation run to calculate the combined outflow hydrograph at Arden Way for the 0.01 AEP event. The resulting peak flow and total runoff volume are included in Table 13.

**Table 13.** HEC-HMS Results at Arden Way for 0.01 AEP Event

Condition	Peak Flow (cfs)	Runoff Volume (acre-feet)
Existing (without development)	941	794.2
Future (with development)	1,003	828.8
<b>Percent increase</b>	<b>6.6%</b>	<b>4.4%</b>

To account for the development of the open area in the COR subbasin, the analyst modified the percent of impervious area, unit hydrograph, and loss rate values.

Based on current and proposed land uses, the analyst estimated that the impervious area for the entire subbasin would increase from 50 to 55 percent.

Precipitation	
<b>Name: 0.010 AEP</b>	
Probability:	1 Percent
Input Type:	Partial Duration
Output Type:	Annual Duration
Intensity Duration:	5 Minutes
Storm Duration:	1 Day
Intensity Position:	50 Percent
Storm Area (MI <sup>2</sup> ):	5.52
5 Minutes (IN)	0.44
15 Minutes (IN)	0.73
1 Hour (IN)	1.21
2 Hours (IN)	1.59
3 Hours (IN)	1.85
6 Hours (IN)	2.50
12 Hours (IN)	3.30
1 day (IN)	4.25
2 Days (IN)	
4 Days (IN)	
7 Days (IN)	
10 Days (IN)	

**Figure 10.** Meteorologic Model for the 0.01 AEP Event

Intuitively, the analyst expected that the unit hydrograph lag would decrease and  $C_p$  would increase. Using relationships from the Denver Lag Equation (<http://www.ems-i.com/wmshelp/WMSHELP.htm>), an increase from 50 to 55 percent impervious area would increase the  $C_p$  value by eight percent. This results in a modified value of 0.96 for the COR subbasin. Using Equation 1 (the regional lag equation), an increase of five percent of impervious area decreases the lag by four percent. This results in a modified lag of 4.68 hours. The loss rates are a function of the soil type. The soil type will not change with the development. So, the loss values will not change.

A duplicate basin model was created, and the percent impervious, Snyder's unit hydrograph lag, and  $C_p$ , were modified. Using the same boundary and initial conditions as the existing condition input, the future peak flow was calculated. The resulting peak flow and total runoff volumes are summarized in [Table 13](#).

## Sensitivity Analysis

The model results should be verified to determine that they agree reasonably well with related analyses and with expected results.

Independent data sources and parameter values from HEC-HMS input should be used for an unbiased comparison. Alternatives include comparison to other regional studies, regional estimates of flow per unit area, nearby gage statistics, and the USGS regional regression equations. The analyst chose the USGS regional regression equations for the comparison. The equations estimate peak flow for AEP events ranging from the 0.5 to 0.01. The USGS publishes these equations for locations all over the United States. For example, equations related to the state of Washington are published in *Magnitude and Frequency of Floods in Washington* (Sumioka, 1998).

The regression equations for California are published in *Magnitude and Frequency of Floods in California* (Waananen, 1977). There are six sets of equations for California. Each set is applicable to a specific region of the state. Sacramento lies in the Sierra flood-frequency region. The flood-frequency equation for the 0.01 AEP event in this region is:

$$Q_{1\%} = 15.7 A^{0.77} P^{1.02} H^{-0.43} \quad (3)$$

where:

- $Q_{1\%}$  = the flow for the 0.01 AEP event, in cf
- $A$  = the drainage area, in square miles
- $P$  = the mean annual precipitation, in inches
- $H$  = the average main channel elevation at ten and 85 percent points along the main channel length, in 1,000 feet

The application of the USGS equations is limited to a specified range of watershed characteristics. The range is based upon the characteristics of the watersheds used in developing the equations. For example, the equation for the Sierra flood-frequency region is applicable to watersheds that have a mean elevation between 100 to 9,700 feet, a mean annual precipitation between seven to 85 inches, and a drainage area between 0.2 to 9,000 square miles. In addition, the equations are not generally applicable to streams in urban areas affected by development. However, factors that account for urbanization have been developed and published by Rantz (1971), Sauer (1983), and Jennings (1994).

The CRS/SRS watershed is below the applicable range of mean elevation for the Sierra equation. So, for comparison sake, an elevation of 1,000 feet was assumed. Using an area of 5.5 square miles and a mean annual precipitation of eighteen inches, the analyst computed a 0.01 AEP peak flow of 1,113 cfs. If the urbanization factor by Rantz (1971) is applied, the peak flow is increased to 1,500 cfs. Considering the uncertainty and variance in the USGS regional equations, this compares reasonably to the approximately 1,000 cfs computed with HEC-HMS. Procedures for evaluating model and regression results are described in *Evaluation of Flood Frequency Estimates for Ungaged Watersheds* (Thomas, 2001). Using the functions developed by Rantz (1971), an increase in developed

watershed of five percent will reasonably increase the peak runoff of the 0.01 AEP event by six percent.

## Processing of Results

To determine how significant this increase in flow is to the peak stage in CRS, a channel model can be used to compute stage. To do so, the analyst could use the peak flow values from the HEC-HMS results as input to the HEC-RAS computer program. Using channel geometry and roughness data, the program computes water surface elevations based upon the flow input.

## Summary

The goal of this study was to identify whether the development of an open area in the CRS watershed increased runoff, and if so, how much. Using available watershed data and computer program HEC-HMS, the analyst was able to answer the questions. As shown in [Table 13](#), the development does increase the peak runoff and total volume of runoff. If the development is to be permitted, some water control features must be included to reduce the peak for the 0.01 AEP storm from 1,003 cfs to 941 cfs. There are many options available for reducing the flood peak and flood damage. Some of these options and how they are modeled using HEC-HMS are discussed in Chapter 4.

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# CHAPTER 3

## Flood Frequency Studies

### Background

#### Objectives

Flood frequency studies relate the magnitude of discharge, stage, or volume to the probability of occurrence or exceedance. The resulting flood-frequency functions provide information required for:

- Evaluating the economic benefits of flood-damage reduction projects.
- Sizing and designing water-control measures if a target exceedance level or reliability is specified.
- Establishing reservoir operation criteria and reporting performance success.
- Establishing floodplain management regulations.
- Developing requirements for regulating local land use.

#### Authority and Procedural Guidance

USACE flood frequency studies are authorized generally by:

- The Flood Control Act of 1936. This is the general authority under which the USACE is involved in control of floods (and associated damage reduction) on navigable waters or their tributaries.
- Section 206 of the Flood Control Act of 1960. This authorizes the USACE to provide information, technical planning assistance, and guidance in describing flood hazards and in planning for wise use of floodplains.
- Executive Order 11988. This directed USACE to take action to reduce the hazards and risk associated with floods.

The following USACE guidance is particularly relevant to the conduct of flood frequency studies:

- ER 1110-2-1450 (USACE, 1994b) describes the scope and general requirements for flood frequency studies.

- EM 1110-2-1415 (USACE, 1993) describes the procedure and computational guidelines for flood frequency studies. The procedures generally follow Bulletin 17B (IACWD, 1982) recommendations.
- EM 1110-2-1417 (USACE, 1994a) describes methods and general guidance for evaluating flood-runoff characteristics. Procedures for development of frequency-based estimates are included.

## Study Procedures

To meet the objectives of a flood frequency study, peak flows, stages, and volumes for specified annual exceedance probabilities (also known as quantiles) are required. The flow and stage frequency curves are often used for flood-damage calculations as discussed in Chapter 4. The volumes are often used for sizing flood control structures such as detention ponds. The values may be required for:

- Current development, without-project conditions.
- Future development, without-project conditions.
- Current development, with-project conditions.
- Future development, with-project conditions.

Here, the terms *current* and *future* are used to refer to watershed conditions existing at the time of the study and at some point later in time, respectively. The terms *without-* and *with-project* refer to the state of the watershed and channels if no action is taken and if a proposed action is taken, respectively. For example, the with-project condition might refer to construction of a proposed detention in the watershed, while the without-project condition refers to the absence of this detention. The without-project, future condition, therefore, is the project area's most likely future condition if no action is taken to resolve whatever problem is addressed by the study.

Frequency functions for current development, without-project conditions can be developed through statistical analysis of observations of flow, stage, or volume. As noted above, ER 1110-2-1450 (USACE, 1994b) and EM 1110-2-1415 (USACE, 1993) present procedures for such analysis, and the HEC-SSP software (HEC, 2010) implements those procedures.

The USGS (Sauer, 1983; Jennings, 1994) and others have performed regional flood-frequency studies for undeveloped and various levels of urbanizing watersheds. If the physical characteristics of the study watershed fall within the range of data used in the regional study, the regional relationships may be used to estimate flow frequencies for existing and future land use conditions.

As a general rule, annual maximum flow-frequency functions estimated from statistical analysis of long records of annual maximum flow are the most reliable frequency functions. However, long records of data are seldom available. Even if a long record was available, the watershed conditions may have changed dramatically due to urbanization or other non-stationary processes, or no large events may have occurred during the period of record. Therefore, an accurate flow-frequency function may not be derived from the historical data alone. A calibrated watershed model with precipitation events of known frequency is often used to develop a flow-frequency function and to compare with other estimates. The calibration of the model is typically based on available historical events of similar frequencies.

Furthermore, with-project condition frequency functions must be developed without statistical analysis. Gage records do not exist for these future, with-project watershed conditions. A commonly used method for this relies on application of a watershed model, such as HEC-HMS, with the so-called design storm assumption. Pilgrim (1975) describe this assumption as follows:

*...in the normal approach to design flood estimation, the intention is to estimate the flood of a selected frequency from a design rainfall of the same frequency...The basic premise [of this approach] is that if median or average values of all other parameters are used, the frequency of the derived flood should be approximately equal to the frequency of the design rainfall.*

The following steps are taken to develop a frequency function with this procedure:

1. Develop a rainfall-runoff-routing model that reflects the characteristics of a watershed and channels for the case of interest: current or future, without- or with-project condition. The current, without-project model should be calibrated to observed data if available, or verified using regional equations or flow estimates.
2. Collect precipitation data, conduct statistical analyses, and define depths of known frequency for the watershed. The results of the statistical analysis may be presented as an intensity-duration-frequency (IDF) function or DDF function, as a set of isohyetal maps, or as a set of equations that define depths for specified durations and frequencies. From these, storm hyetographs can be developed.

In many cases, this work has been done by NWS or by a local government agency. For example, the National Oceanic and Atmospheric Administration (NOAA) Atlas 2 and 14 presents isohyetal maps for six-hour and 24-hour durations, for the 0.50-,

0.20-, 0.10-, 0.04-, 0.02-, and 0.01 AEP events, for the Western United States (NOAA, 1973; NOAA, 2006). These documents also present methods for deriving depths for other durations. For the Central and Eastern United States, NWS TP-40 (NOAA, 1961); TP-49 (NOAA, 1964); and, HYDRO-35 (NOAA, 1977) provide similar information.

3. For a selected frequency, use the IDF or DDF information to define a precipitation hyetograph, then use the rainfall-runoff-routing model to compute peak flow, stage, or volume. Assign the frequency (AEP) of the precipitation to the peak flow, stage, or volume, following the design-storm assumption described above.
4. Repeat the process for a range of frequency events.
5. Assemble the results to yield a complete frequency function.
6. Use sensitivity analysis to determine the most important parameters if further adjustment of the frequency curve is needed.
7. Compare these storm frequency hydrologic model results with other methods (e.g., if available, flow statistics and regional regression equations) to determine the best estimate of the current, without-project flow-frequency curve.

Such application is the subject of the case study that follows.

## **Case study: Estimating Flood Frequency in the CRS/SRS Watershed**

### **Watershed Description**

This case study is an extension of that described in Chapter 2. There, a description is presented of how a USACE study team used HEC-HMS to evaluate the impact of development of 320 acres of open land in the CRS/SRS watershed. The CRS/SRS watershed is an urban watershed of approximately fifteen square miles within Sacramento County, in northern California. The HEC-HMS results indicated that development in the CRS watershed would increase the runoff by 6.6 percent (from 941 to 1,003 cfs) for the 0.01 AEP event at the Arden Way stream gage.

### **Decisions and Information Required**

While the change in runoff for the 0.01 AEP event provides a useful, simple measure of the impact of development, this index alone is not adequate for complete assessment of the impact. It fails to account for changed flood damage due to development, and it fails to reveal the

impact of the development with more- and less-frequent events. A complete frequency function is necessary for the latter, and flood damage analysis as described in Chapter 4 is necessary for the former.

Thus, as part of the feasibility study introduced in Chapter 2, a USACE analyst has been tasked with answering the question: What is the increase in flood damage in CRS as a result of the development of 320 acres of open area?

Flood damage evaluations require development of an annual maximum flow frequency function, for the current- and future-development conditions. As described below, these functions were developed at each of the subbasin outlets identified in Chapter 2; the locations are shown in Table 14.

**Table 14.** Subbasin Outlets, Location of Flow-Frequency Points

Subbasin ID	Outlet Location
COR	CRS at Corabel Lane
FUL	CRS at Fulton Avenue
ARD	CRS at Arden Way

## Model Selection, Temporal Resolution, and Spatial and Temporal Extent

The study team used computer program HEC-HMS with frequency storms to develop all required frequency functions: current and future, without- and with-project. The current-condition, without project, frequency curve can be developed by other methods as previously noted, but the hydrologic model will be required to develop the future condition with- and without-project frequency curve. Alternative methods for estimating the existing-condition frequency curve are discussed at the end of this case study.

The HEC-HMS watershed and channel model described in Chapter 2 was used for this portion of the study. The model contained:

- Infiltration: Initial and constant-rate runoff-volume method.
- Direct-runoff transform: Snyder's unit hydrograph transform method.
- Baseflow: None used.
- Routing: Muskingum-Cunge eight-point channel routing method.

The spatial and temporal extents defined in Chapter 2 were the same for this portion of the study.

A two-minute computational time step, selected as described in Chapter 2, was used for this study. The 24-hour storm duration, selected as described in Chapter 2, was used for the frequency-based storms herein.

## Model Calibration and Verification

Chapter 2 describes how the rainfall-runoff-routing model was calibrated and verified. The same parameters were used to develop the frequency function. Note that the initial conditions vary, as shown in [Table 11](#), depending on the event. For example, the initial loss of the 0.01 AEP event is less than that used for the 0.50 AEP event. In northern California, a 0.01 AEP event will not occur suddenly, on a sunny day. Instead, it will occur after a longer period of precipitation, caused by storms moving from the Pacific Ocean. Consequently, the soil is likely to be saturated when such a large event occurs, in which case, the initial loss would be small.

## Application

To provide the information required the study team:

- Developed a range of hypothetical (frequency) precipitation events.
- Used the precipitation events as the boundary condition to the watershed model.
- Computed a peak flow for each frequency event for the current and future development conditions and assembled the results to obtain the desired frequency functions.
- Compared storm-frequency hydrologic-model results with other methods to obtain the best estimate of the current development frequency curve.

To develop the flow-frequency function, a range of hypothetical (frequency) precipitation events was used within the watershed model. The 0.01 AEP event used in Chapter 2 was based upon depths from locally-developed DDF functions. The same DDF functions were used to develop the precipitation-frequency functions needed here for seven other events; [Table 15](#) shows these. HEC-HMS has eight predefined options for frequency storms. The specific frequencies provide adequate resolution of the frequency function. The frequencies correspond to the same annual exceedance probabilities shown in [Table 15](#). All eight of the precipitation frequencies listed were used as a boundary condition for the watershed model, thus yielding eight quantiles for the frequency functions.

**Meteorologic Models** for the eight frequency events were added to the HEC-HMS input. (Because the 0.01 AEP precipitation input was completed in Chapter 2, only seven additional models need be created.) To create a **Meteorologic Model**, the analyst:

**Table 15.** Depth-Duration-Frequency (DDF) Functions (inches)

Duration	Depth (in) for Specified Annual Exceedance Probability							
	0.50	0.20	0.10	0.04	0.02	0.01	0.004	0.002
5 min	0.13	0.20	0.25	0.32	0.38	0.44	0.49	0.58
10 min	0.19	0.29	0.36	0.46	0.54	0.62	0.70	0.82
15 min	0.23	0.35	0.43	0.55	0.64	0.73	0.82	0.96
30 min	0.32	0.47	0.57	0.72	0.83	0.94	1.04	1.22
1 hour	0.45	0.64	0.77	0.94	1.07	1.21	1.33	1.53
2 hours	0.64	0.88	1.04	1.26	1.42	1.59	1.76	2.00
3 hours	0.77	1.04	1.23	1.47	1.66	1.85	2.03	2.31
6 hours	1.06	1.40	1.65	1.95	2.22	2.50	2.75	3.10
12 hours	1.43	1.91	2.25	2.67	3.00	3.30	3.60	4.00
24 hours	1.90	2.50	2.98	3.46	3.85	4.25	4.60	5.20
36 hours	2.25	3.02	3.54	4.15	4.60	5.09	5.53	6.24
2 days	2.51	3.40	3.95	4.65	5.15	5.70	6.20	7.00
3 days	3.00	4.07	4.65	5.50	6.20	6.80	7.50	8.40
5 days	3.61	4.91	5.76	6.85	7.63	8.42	9.20	10.29
10 days	4.73	6.44	7.54	8.96	9.97	11.01	11.95	13.46

1. Selected **Meteorologic Model Manager** from the **Components** menu and clicked the **New** button.
2. Entered a **Name** for the **Meteorologic Model** and a **Description**. The analyst used the AEP as the name so that it would be easily identified and clicked **Create**.

Such care in naming models is critical in a complex study such as this. The model may be needed in subsequent analyses or by another analyst, and finding and retrieving the model will be easier if a meaningful name is selected now. Similarly, record names used by HEC-HMS for storing data and results in the DSS data management system are formed from model and project names. If the records are to be retrieved for use with HEC-RAS or another tool, selection of informative model names is essential.

3. Connected the **Meteorologic Model** to the **Basin Model** by selecting the **Basins** tab in the meteorologic model **Component Editor** and switching the **Include Subbasins** option from **No** to **Yes**, as shown in [Figure 11](#).
4. Specified the **Precipitation Method** by selecting **Frequency Storm** from the list, as shown in [Figure 12](#).
5. Entered the precipitation depth-duration data from [Table 15](#), as illustrated in [Figure 13](#).

Here, the actual exceedance **Probability** is selected for each of the eight boundary conditions. (Note that the options shown in the HEC-HMS form are actually exceedance probability multiplied by 100, expressed as a percentage.) The **Input Type** identifies the series type of the depth-duration input data. This can be either **Partial** or **Annual** duration. The **Output Type** identifies the series type of the resulting flow data. If the **Input Type** and **Output Type** are the same then no changes are made to

Meteorology Model Basins	
Name: 0.020 AEP	
Basin Model	Include Subbasins
COR	No
Corabel2	No
CRS 0.002 current	No
CRS 0.002 future	No
CRS 0.004 current	No
CRS 0.004 future	No
CRS 0.010 current	No
CRS 0.010 future	No
CRS 0.020 current	No
CRS 0.020 future	No
CRS 0.040 current	No
CRS 0.040 future	No
CRS 0.100 current	No
CRS 0.100 future	No
CRS 0.200 current	No
CRS 0.200 future	No
CRS 0.500 current	No
CRS 0.500 future	No
CRS WithProject1997	No
CRS WOPC 1997	No
	Yes
	No

Figure 11. Connected Meteorologic Model to Basin Model

Meteorology Model Basins	
Name: WOPC 100yr	
Description:	Without-project condition 100 year 24 hour eve
Precipitation:	Frequency Storm
Evapotranspiration:	--None--
Snowmelt:	Frequency Storm
Unit System:	Gage Weights
	Gridded Precipitation
	Inverse Distance
	SCS Storm
	Specified Hyetograph
	Standard Project Storm

Figure 12. Selection of Appropriate Precipitation Method

the specified values. If they are different, then HEC-HMS converts the specified values to partial or annual duration series depths for the 0.50, 0.20, and 0.10 AEP events. Multipliers from TP-40 (NOAA, 1961) are used for this.

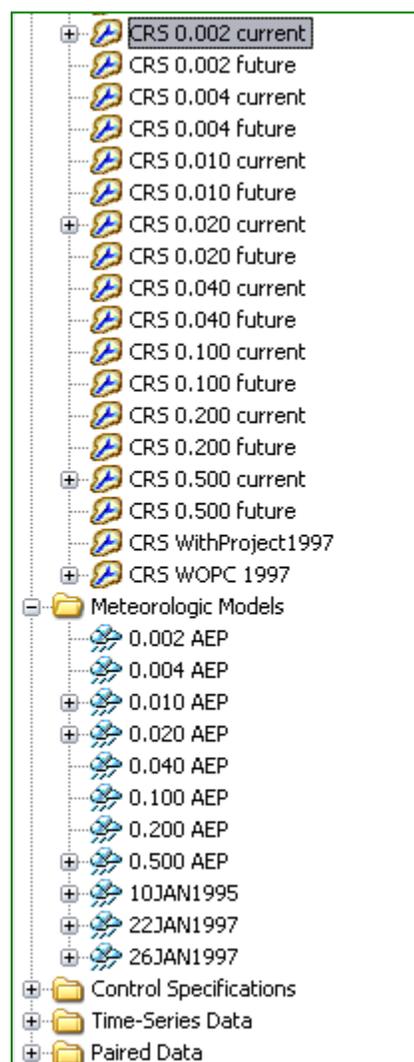
Precipitation	
<b>Name: 0.500 AEP</b>	
Probability:	50 Percent
Input Type:	Partial Duration
Output Type:	Annual Duration
Intensity Duration:	5 Minutes
Storm Duration:	1 Day
Intensity Position:	50 Percent
Storm Area (MI2)	5.52
5 Minutes (IN)	0.13
15 Minutes (IN)	0.23
1 Hour (IN)	0.45
2 Hours (IN)	0.64
3 Hours (IN)	0.77
6 Hours (IN)	1.06
12 Hours (IN)	1.43
1 day (IN)	1.90
2 Days (IN)	
4 Days (IN)	
7 Days (IN)	
10 Days (IN)	

**Figure 13.** Input for 0.500 AEP Precipitation Event

The **Intensity Duration** controls how HEC-HMS defines the frequency storm; it uses the procedures described in the HEC-HMS Technical Reference Manual (HEC, 2000). The duration selected should equal the computation time step, as specified in the **Control Specifications**, if possible. Otherwise, a duration near the time step should be selected. If fifty percent is selected for **Intensity Position**, the peak intensity is at fifty percent of the duration - at twelve hours for a 24-hour storm. The **Storm Duration** is the total duration of the precipitation event that is to be analyzed. Note that this may be less than the maximum duration for which a depth is specified in the table, but it should not be greater. The analyst must ensure that the duration of the analysis, as defined by the **Starting Date, Starting Time, Ending Date, and Ending Time** in the **Control Specifications**, exceeds the duration of the rainfall. Otherwise, the entire watershed will not contribute to runoff. The **Storm Area** is used for depth-area correction, as described in the HEC-HMS Technical Reference Manual (HEC, 2000). For the CRS/SRS analysis, the **Storm Area** is the entire watershed area upstream of Arden Way. However, the areal correction for precipitation on this relatively small watershed is negligible.

Steps 1 - 5 were repeated to develop the eight required **Meteorologic Models** - one for each exceedance probability.

As described above, the initial loss varies with the hypothetical event. The less frequent the event is, the less the initial loss. To model this, the analyst created eight copies of the current development condition **Basin Model** (developed in Chapter 2) and eight copies of the future development condition **Basin Model** (developed in Chapter 2). Each was assigned to a hypothetical event and named accordingly. Then the initial loss value for each was modified. The resulting models are shown in Figure 14.



**Figure 14.** Watershed Explorer

The analyst then created the **Control Specifications** to be used for all hypothetical events. To do so, the analyst:

1. Selected the **Control Specifications Manager** from the **Components** menu and clicked the **New** button.
2. Entered a name for the **Control Specifications**, gave a brief **Description**, and clicked **Create**.
3. Entered the **Starting Date**, **Starting Time**, **Ending Date**, and **Ending Time** for the runoff simulation. If the analyst were using an historical event, the known starting and ending dates and times would have been entered. However, with a hypothetical event, any starting date and time can be specified. (After all, the analyst is predicting what will happen if the event occurs, not forecasting when the event will occur.)

The **Ending Date** and **Ending Time** must be carefully selected. The simulation time window (**Starting Time** to **Ending Time**) must be great enough to permit simulation of the entire rainfall event plus the response of the entire watershed and channel system. This duration is the storm duration plus the travel time of the entire watershed. Here, to be safe, the analyst added twelve hours for the travel time to the 24-hour storm duration, and selected an **Ending Time** 36 hours after the **Starting Time**. The analyst also selected the appropriate **Time Interval**. [Figure 15](#) shows the completed control specifications.

Control Specifications	
<b>Name:</b>	Hypothetical Event
Description:	<input type="text"/>
Start Date (ddMMMYYYY)	01Jan2002
Start Time (HH:mm)	00:00
End Date (ddMMMYYYY)	02Jan2002
End Time (HH:mm)	12:00
Time Interval:	2 Minutes

**Figure 15.** Control Specifications for Hypothetical Events

The analyst computed the simulation runs for the current development condition **Basin Models**, being sure that the frequency of the **Basin Model** corresponded to the frequency of the **Meteorologic Model**. The without-project, current development peak flow from each of the eight frequency events at Arden Way is shown in [Table 16](#).

## Adopting a Frequency Curve

[Table 17](#) shows alternative methods for deriving a frequency function for this watershed; these may be employed to check the reasonableness of the function derived as described above. All of those methods together with

**Table 16.** Without-Project, Current Development Peak Flow at Arden Way

AEP	Peak Flow for Current Development Condition (cfs)
0.500	285
0.200	450
0.100	571
0.040	715
0.020	826
0.010	944
0.004	1,060
0.002	1,230

**Table 17.** Alternative Methods for Deriving Flow-Frequency Functions

Method	Description
Frequency storms	Precipitation events of known frequency are used with a rainfall-runoff model to compute a flow-frequency function. The example presented in this chapter uses this method.
Regional regression equations for frequency curve quantiles	These estimate the peak flow for specific frequency events (the quantiles). For example, as described in Chapter 2, the USGS has developed equations to estimate peak flow for areas in California (Waananen, 1977). In this case, equations are available to estimate the 0.50, 0.20, 0.01, 0.04, 0.02, and 0.01 AEP flow peaks. Similar equations are available for the rest of the United States (Sauer, 1983; Jennings, 1994).
Regional regression equations for frequency function parameters	Regional equations estimate the parameters of a frequency function, using watershed characteristics as the independent variables. For example, these equations may provide an estimate of the mean and standard deviation based upon watershed length, slope, and area. With these statistics, the frequency function can be defined, and quantiles determined.
Statistical analysis or frequency-based runoff calculations for "hydrologically" similar watersheds	A frequency function derived for a "hydrologically" similar watershed can be transferred to (factored based upon drainage areas) and used for the watershed of interest. This option is more qualitative than the others.
Frequency function fitted to gaged data	If a sufficient period of record is available, a frequency function can be fitted, using, for example, HEC-SSP (HEC, 2010). From this, both quantiles and confidence limits can be defined.

the frequency-storm method just developed provide important information about the frequency curve. Selecting an individual curve or a subjectively weighted average of some curves will depend on the confidence one has in the data available and the frequency method's use of that data. The process of adopting a frequency curve is described in TD-11 (HEC, 1980).

If gage records are good and exist for a relatively long period, say longer than thirty years, then a frequency function fitted to the data may be the method one has the most confidence in. But, fitting a frequency function

to a short period of record is of limited use because of the significant uncertainty, especially when providing information for design or operation when public safety is an issue of concern. The guidelines in ER 1110-2-1450 (USACE, 1994b) recommend avoiding statistical analysis for short samples of data. Here, for illustration though, the analyst used the ten years of annual maximum flow at the Corabel Gage and developed a flow-frequency function using computer program HEC-SSP (HEC, 2010). [Table 18](#) shows results. Column 1 shows the exceedance probability, and Columns 2 and 4 show the corresponding peak flows computed with HEC-HMS and HEC-SSP, respectively. The 0.50 AEP value compares well. However, the 0.01 AEP flow from HEC-HMS is about half the value predicted by fitting a distribution to the data.

**Table 18.** Comparison of Results to Fitted Flow-Frequency Function and USGS Regression Equations

AEP	HEC-HMS Computed Peak Flow (cfs)	USGS Regression Equation Peak Flow (cfs)	HEC-SSP Quantiles		
			Expected (cfs)	95% Confidence (cfs)	5% Confidence (cfs)
0.500	285	140	282	195	410
0.100	571	564	718	440	1,278
0.010	944	1,500	1,808	741	3,482

The analyst also looked at the peak flows estimated by the USGS regression equations, as described in Chapter 2. These flows are included in Column 3 of [Table 18](#).

The values predicted with HEC-HMS do fall well within the 95 and 5 percent confidence bands from HEC-SSP. These confidence bands describe the uncertainty about the fitted function. For example, the confidence bands for the 0.01 AEP event show that the probability is 0.90 that the true 0.01 AEP event is between 741 and 3,482 cfs. The HEC-HMS and USGS values fall in that range. If the HEC-HMS values did not, the analyst should reconcile this difference, seeking reasons why and perhaps correcting the HEC-HMS model. The USGS value for the 0.500 AEP event is below the lower end of the confidence band. However, as noted in Chapter 2, the watershed is below the applicable range of mean elevation for the equations. This and the uncertainty and variance in the equations may account for the low value.

Considering the short period of flow data available, the uncertainty and variance in the USGS regression equations, and that a calibrated rainfall-runoff model is available, the analyst decided the best-estimate flow-frequency curve would be from the frequency-storm method. Therefore, the peak flows computed by HEC-HMS were adopted as the flow-frequency curve.

## Computing Future Development Frequency Functions

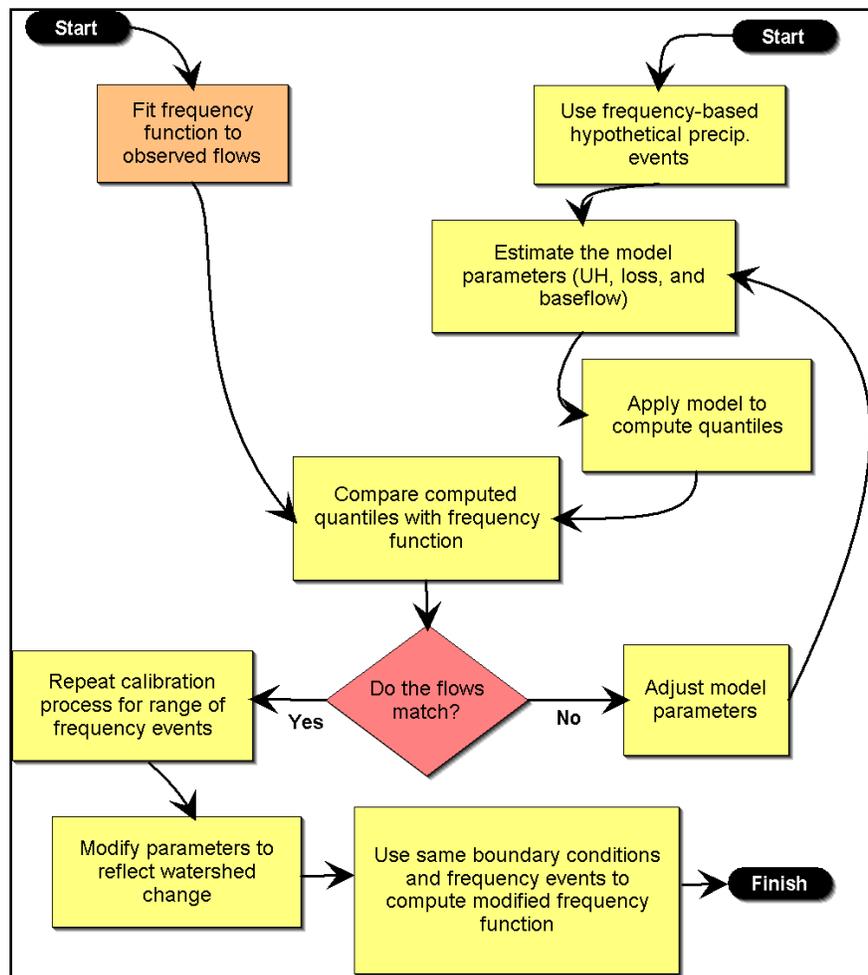
Once the current development frequency function was adopted, the next step was to develop the future development frequency function. The future development function is developed using a hydrologic model. The hydrologic model must be correlated to the adopted frequency curve. In so doing, changes in the frequency function can be calculated by modifying the model and using the same precipitation events. Modifications to the model may result from changes in land use or construction of flood control projects. The two basic methods to correlate the hydrologic model are to:

- Calibrate the peak flow from the hydrologic model to match the desired frequency.
- Assign a frequency to the peak flow from the hydrologic model based on the adopted frequency curve.

The adopted frequency curve in the example used the frequency storm method, so the precipitation frequency and the resulting flow frequency were assumed to be the same. This is the first method. Therefore, changes to the frequency curve due to future development were calculated by modifying the watershed characteristics and exercising the model with the same precipitation events used for the current development condition. Both methods are described in more detail herein.

**Method 1.** The steps included below describe an approach that entails fitting a watershed model to an adopted frequency function using loss values as the calibration parameter. This process is schematically shown in [Figure 16](#) and requires the following steps:

1. Adopt a frequency curve. Use one or more of the methods from Table 17 to develop the best-estimate frequency curve, see previous section.
2. On a parallel path, use IDF or DDF functions as boundary conditions in HEC-HMS, as done in the example here. Use reasonable estimates of initial conditions and model parameters. Use the HEC-HMS model to compute runoff peaks for the frequency events.
3. Compare the runoff peaks computed by HEC-HMS (Step 2) for a given frequency event to the flow from the adopted curve (Step 1) for the same frequency. For example, compare the runoff peak from HEC-HMS using the 0.01 AEP precipitation event with the 0.01 AEP peak flow from the adopted frequency function. (In the



**Figure 16.** Method 1 Steps to Calibrate Models to Flow-Frequency Function

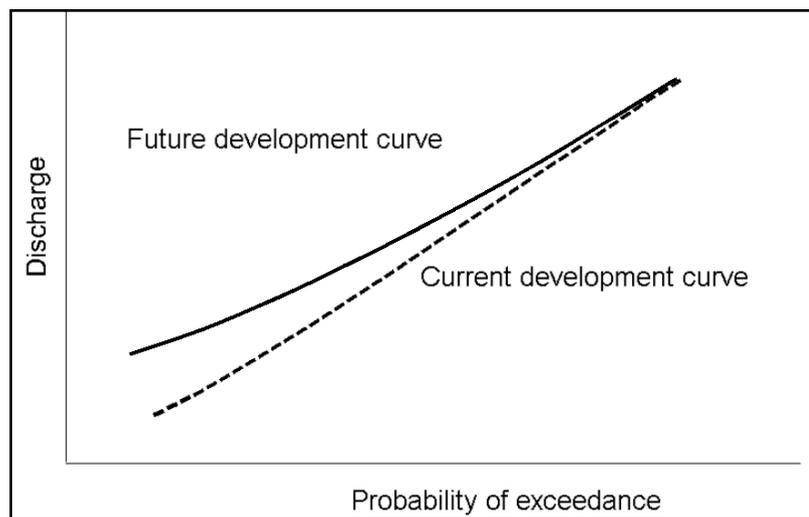
case where the frequency storm method is the adopted method, they are one in the same.)

4. Adjust the HEC-HMS model parameters such that for a given frequency, the peak flow computed by HEC-HMS matches the peak flow from the function generated by HEC-SSP. The goal is to have the flow due to the  $n$ -AEP precipitation event (derived from precipitation-frequency studies) equal the  $n$ -AEP flow (from the adopted curve). Typically, the initial loss parameter is used as a calibration parameter because it represents antecedent moisture, which is a major factor in flood magnitude.
5. Repeat this calibration process for a range of frequency events (the 0.50 to 0.002 AEP events).
6. Modify the model parameters to reflect the future condition (land use change or with-project).

7. Use the calibrated initial loss value for each frequency event along with the same frequency-based precipitation event to compute a future flow-frequency function. The resulting peak flows for the example are shown in Table 19. The changes to the frequency curve are schematically shown in Figure 17.

**Table 19.** Without-Project, Future Development Peak Flow at Arden Way

AEP	Peak Flow for Future Development Condition (cfs)
0.500	308
0.200	481
0.100	608
0.040	760
0.020	876
0.010	999
0.004	1,120
0.002	1,296

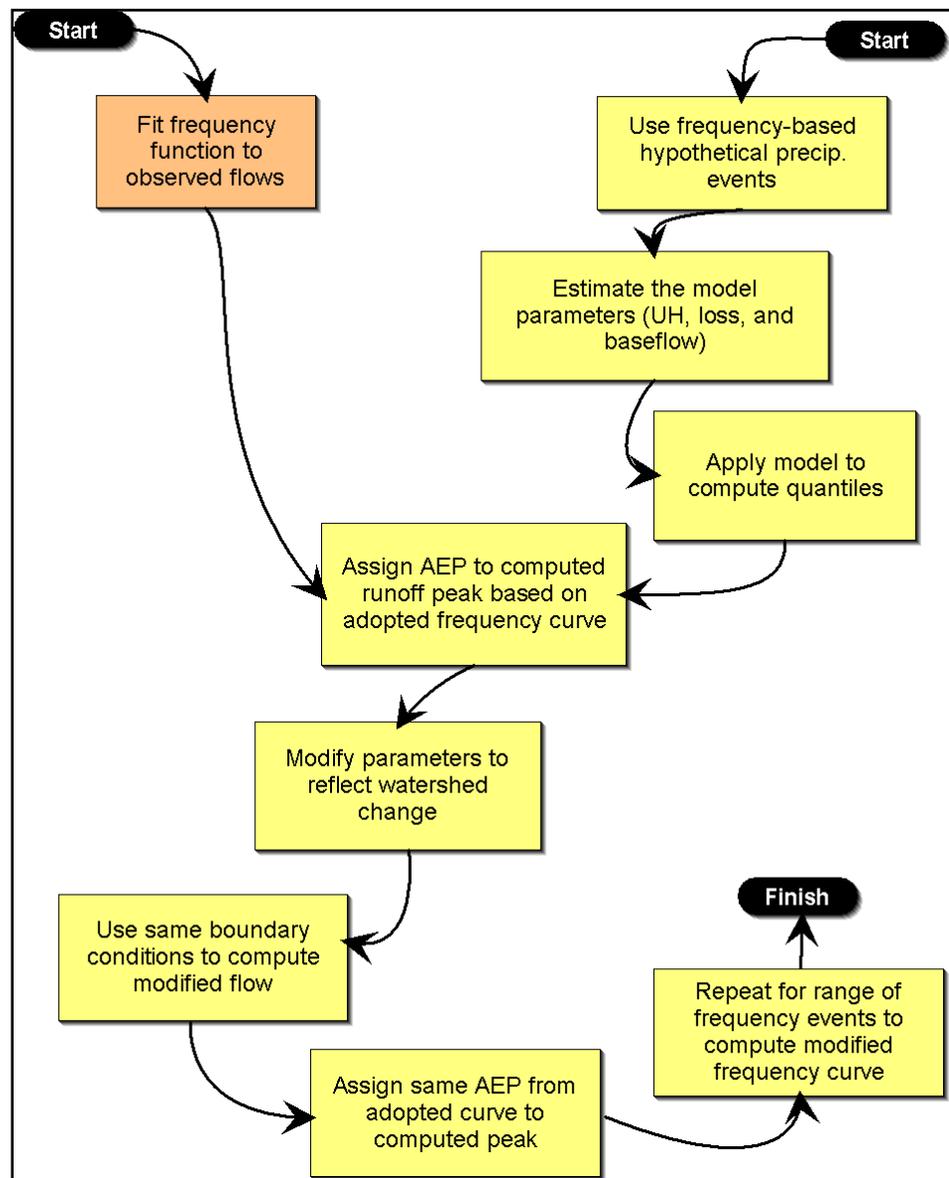


**Figure 17.** Comparison of Without-Project Flow-Frequency Curves

When using the procedure described, care must be taken to ensure that the calibrated loss model parameters match regionally acceptable values. For example, when attempting to match the 0.10 AEP flow from the watershed model to the 0.10 AEP flow from the flow-frequency function, suppose a large initial loss is needed. If the value is beyond a reasonable range, then an alternative method needs to be considered.

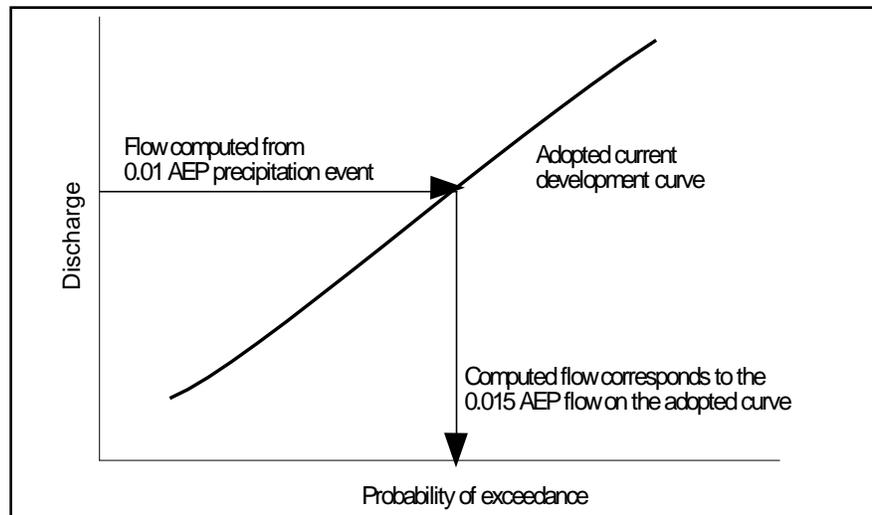
**Method 2.** An alternative method to develop a modified frequency curve based upon an adopted frequency curve is schematically shown in Figure 18 and is as follows:

1. Adopt a frequency curve for the current-development, without-project condition.



**Figure 18.** Method 2 Steps to Calibrate Models to Flow-Frequency Function

2. Use IDF or DDF functions as boundary conditions in HEC-HMS. Use estimates of model parameters that are consistent with regional studies and data. For example, select an initial loss that is consistent with the soil type and precipitation volume. Use the HEC-HMS model to compute a runoff peak for the hypothetical frequency event.
3. Assign an AEP to the runoff peak based upon the adopted frequency curve. Note that this frequency may differ from that of the boundary condition IDF and DDF functions. For example, as shown in [Figure 19](#), the 0.01 AEP precipitation event may yield the 0.015 AEP peak flow.



**Figure 19.** Example of Using Adopted Frequency Curve to Assign AEP

4. Adjust the HEC-HMS model parameters to reflect the modified conditions. The modifications may reflect land use changes or projects for damage-reduction alternatives.
5. Use the same boundary conditions with the modified model and compute a runoff peak.
6. Assign the same AEP from the adopted frequency curve, as described in Step 3, to this computed runoff peak. Compare the change in peak flow as a result of the watershed modifications based upon the same boundary conditions.
7. Repeat this process for a range of frequency events (the 0.50 to 0.002 AEP events).

### Using Frequency Curves in Project Analysis

For the CRS/SRS analysis, computer program HEC-FDA was used to compute EAD. This program requires two main categories of input: (1) hydrologic and hydraulic data and (2) economic data. To complete the hydrologic and hydraulic data requirements, the computed peak flows must be converted to water surface elevations. For CRS/SRS, the analyst used computer program HEC-RAS to compute water surface elevations for the given flow rates from the frequency function. The analyst must then gather the required economic data. This includes information identifying vulnerable structures, first-floor elevations, structure value, content value, etc.

Once the EADs for the current and future development conditions are computed, the increase in EAD as a result of the land developed can be calculated.

In the Chapter 4, flood-damage reduction alternatives will be considered. The EAD computed in this chapter will serve as the basis for identifying the most cost-effective alternative for reducing flood damage.

## Summary

The objective of this study was to determine current and future development condition frequency functions to be used in flood damage analysis in the CRS watershed. To calculate the increase in EAD, the analyst needed the current and future development flow-frequency functions. Using an available calibrated rainfall-runoff model of current development in the watershed, precipitation events of known frequency were used in HEC-HMS to compute a flow-frequency function. Eight precipitation frequency events were used to ensure adequate resolution of this function. The flow-frequency function developed was then compared to functions developed using alternative procedures. A best-estimate current development condition frequency function was then adopted. Then, the rainfall-runoff model was altered to reflect the watershed with future development. For both development conditions, several methods, as shown in [Table 17](#) and discussed in the text, can be used to compute flow-frequency functions.

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# CHAPTER 4

## Flood-Loss Reduction Studies

### Background

Flood-frequency functions developed following procedures described in Chapter 3 provide quantitative information about the risk of flooding in a watershed. If the flow-frequency functions are combined with rating and elevation-damage information, expected annual damage can be computed. This computation is the foundation for assessment and comparison of the effectiveness of flood-loss reduction plans. This chapter illustrates how HEC-HMS can be used in the context of such a study.

### Authority and Procedural Guidance

USACE activities in flood-loss reduction studies are authorized by:

- The Flood Control Act of 1936.
- Section 206 of the Flood Control Act of 1960.
- Executive Order 11988.
- Section 73 of Public Law 93-251.

In addition to technical guidance identified in earlier chapters, relevant USACE guidance for hydrologic engineering analyses in flood-damage reduction studies includes:

- EP 1110-2-10 (USACE, 1994) provides an overview of flood-damage reduction studies and describes the basic principles of the analyses required throughout a study. It addresses the role of various computer programs in those analyses.
- ER 1110-2-1419 (USACE, 1995) identifies possible damage reduction measures and summarizes typical hydrologic engineering studies required for formulation and evaluation of each.
- EM 1110-2-1619 (USACE, 1996) describes procedures for decision-making under uncertainty - a requirement for all flood-damage reduction studies. It describes how, for example, the impact a lack of gaged flow data has on the frequency function and proposes how this uncertainty can be modeled and accounted for in planning.

## Study Objectives

Flood-loss reduction studies are typically undertaken to find the optimal plan to reduce flood damage for a particular watershed - in this case, the optimal plan is the plan that yields the maximum net benefit. As described in EM 1110-2-1419 (USACE, 1995), net benefit, NB, of a proposed plan is computed as:

$$NB = B_L + B_I + (E[D_{without}] - E[D_{with}]) - C \quad (4)$$

where:

- $B_L$  = annual equivalent location benefit of the plan;
- $B_I$  = annual equivalent intensification benefit of the plan;
- $E[D_{without}]$  = EAD in the watershed without the plan;
- $E[D_{with}]$  = EAD with the plan in place;
- $C$  = annual equivalent cost of implementing, operating, maintaining, repairing, replacing, and rehabilitating all components of the plan.

The without-plan condition represents existing and future conditions in the absence of the plan, and the with-plan condition represents conditions if a damage reduction plan is implemented.

EM 1110-2-1415 (USACE, 1993) describes how the EAD for an urban area, both without and with a plan, is computed by integrating the appropriate annual damage-frequency function. The damage-frequency function is developed by first transforming the flow-frequency function with a rating curve (relationship of flow and elevation), thus yielding an elevation-frequency function. This, in turn, is transformed with an elevation-damage function, yielding the required damage-frequency function.

The flow-frequency, flow-elevation, and elevation damage functions used in the EAD computation are not known with certainty. For example:

- Uncertainty about future hydrologic events and watershed conditions, uncertainty regarding the choice of a statistical distribution, and uncertainty regarding values of parameters of the distribution lead to uncertainty about the frequency function.
- Uncertainty that arises from the use of simplified models to describe complex hydraulic phenomena, from the lack of detailed geometric data, from misalignment of a hydraulic structure, from material variability, and from errors in estimating slope and roughness factors leads to uncertainty about the rating function.

- Economic and social uncertainties, including lack of information about the relationship between depth and inundation damage, lack of accuracy in estimating structure values and locations, and lack of ability to predict how the public will respond to a flood, cause uncertainty about the elevation-damage function.
- Uncertainty about structural and geotechnical performance of water-control measures when these are subjected to rare stresses and loads caused by floods, cause further uncertainty about flood elevations.

Traditionally in USACE planning studies, these uncertainties have not been considered explicitly in plan formulation and evaluation. Instead, the uncertainties have been accounted for implicitly with factors of safety and freeboard. EM1110-2-1619 (USACE, 1996) now calls for explicit acknowledgement and description of the uncertainties and for quantitative risk analysis in the EAD computation. In simple terms, a description of uncertainty in each of the functions is included in the transformation and integration. Such a distribution might reveal, for example, that the probability is 0.05 that the error in predicting the 0.01 probability discharge is greater than 500 cfs.

With such a description of the error or uncertainty, a description of the uncertainty of the EAD value can also be derived, reported, and weighed in the decision making.

Table 20 lists damage reduction measures, both structural and nonstructural, and shows how each will alter the frequency, rating, or damage function. Complex plans that include multiple measures will alter more than one of the functions. The impact of measures that alter the frequency function can be evaluated conveniently with HEC-HMS. Evaluation of the impacts of others may require use of other software that is listed in Table 4.

Table 20. Damage Reduction Measures and their Impact

Measures that Reduce Flow for Specified Frequency	Measures that Reduce Water Surface Elevation in Floodplain for Specified Flow	Measures that Reduce Damage for Specified Elevation
Reservoir/detention Diversion Watershed management	Channel improvement Levee/floodwall	Relocation of property (temporary or permanent) Flood warning and preparedness planning Land-use and construction regulation

## Study Procedure

The study procedure is straightforward:

1. Develop the without-plan flow-frequency function, including a description of the uncertainty. HEC-HMS may be used to develop the function.
2. Combine that frequency function with the without-project rating and damage functions, which are also known without certainty. Computer program HEC-FDA (HEC, 2008) can be used for this combination and computation.
3. Select one of the proposed plans and develop the with-project frequency function for that condition, including a description of the uncertainty.
4. Combine that frequency function with the with-project rating and damage functions and compute the with-project damage frequency function, including a description of uncertainty, and EAD for the plan.
5. Determine intensification and location benefits, the cost of the plan, and the net benefit.
6. Compute other indices of plan performance, following guidance in EM 1110-2-1619 (USACE, 1996).
7. Repeat Steps 3, 4, 5, and 6 for all other proposed plans.
8. Compare the results to select the optimal plan.

Note that these steps require significant interaction amongst members of the study team: hydrologic and hydraulic engineers will provide the frequency functions, economists will provide the elevation-damage information, and cost estimators will provide costs of construction.

## Case Study: Evaluation of Inundation-Reduction Benefits in the CRS Watershed

### Watershed Description

Chapters 2 and 3 described the CRS/SRS watershed. Flooding may occur there during events that exceed approximately the 0.04 AEP event. This history of flooding led to an effort to provide relief to the residents and property owners, and a flood-damage reduction study was initiated. The

hydrologic engineering component of the study is to provide the flow-frequency functions and related uncertainty for without and with-project conditions. The with-project condition includes the following flood-damage reduction alternatives proposed by a USACE study team and the local sponsor:

**Alternative 1:** Detention pond upstream of Fulton Avenue.

**Alternative 2:** Off-stream detention. Similar to Alternative 1, but with an upstream diversion that is designed to pass some portion of the flow without detention.

**Alternative 3:** Diversion from CRS into SRS at Fulton Avenue.

**Alternative 4:** Floodwall along CRS, from D05 pond to Arden Way.

**Alternative 5:** Raise low-lying structures near Howe Avenue.

The specific dimensions and configuration of the measures included in the plans will be determined iteratively, using results of the hydrologic engineering and economic analysis. However, initial candidate dimensions were nominated and properties of the features of the measures were proposed by the study team. For example, a candidate outlet configuration of the detention pond was identified, and a candidate capacity of the pond for Alternative 1 was proposed.

## **Required Decisions and Necessary Information**

The question that must be answered is: Which of the proposed plans, if any, should be funded and implemented? The information required to make that decision includes the inundation reduction benefit of each alternative plan. Computation of that benefit requires without and with-project frequency, rating, and damage functions.

The process for developing the best estimate frequency curve for the without-plan condition is described in Chapter 3. In this case, Methods 1 and 2 are the same because the best-estimate flow-frequency curve is the frequency storm procedure. Thus, to develop the with-plan frequency curve, the analyst used the same hypothetical storms of known frequency with modified HEC-HMS models of the watershed and channels that reflect the appropriate alternative. The additional steps in the analysis with HEC-HMS include:

1. Use developed meteorologic models with frequency storms for the watershed, as illustrated in Chapter 3.

2. Develop a basin model for Alternative 1, including in that model a representation of the proposed detention pond.
3. Exercise that model with the frequency storms from Step 1 to develop the with-plan flow-frequency function for Alternative 1.
4. Repeat Steps 2 and 3 for alternatives 2, 3, 4, and 5.

The analyst described the uncertainty associated with the frequency functions using an estimate of the equivalent years of record. Guidelines for estimating the equivalent years of record are found in Table 4 and 5 of EM 1110-2-1619 (USACE, 1996).

### Model Selection, Fitting, and Boundary and Initial Conditions

For this analysis, the HEC-HMS model that is described in Chapters 2 and 3 was used for runoff computations. The analyst did complete the model by adding the remaining portion of the watershed. This was done following the same procedures and techniques described in Chapter 2. The completed model is shown in Figure 20. The hypothetical storms needed were developed as described in Chapter 3.

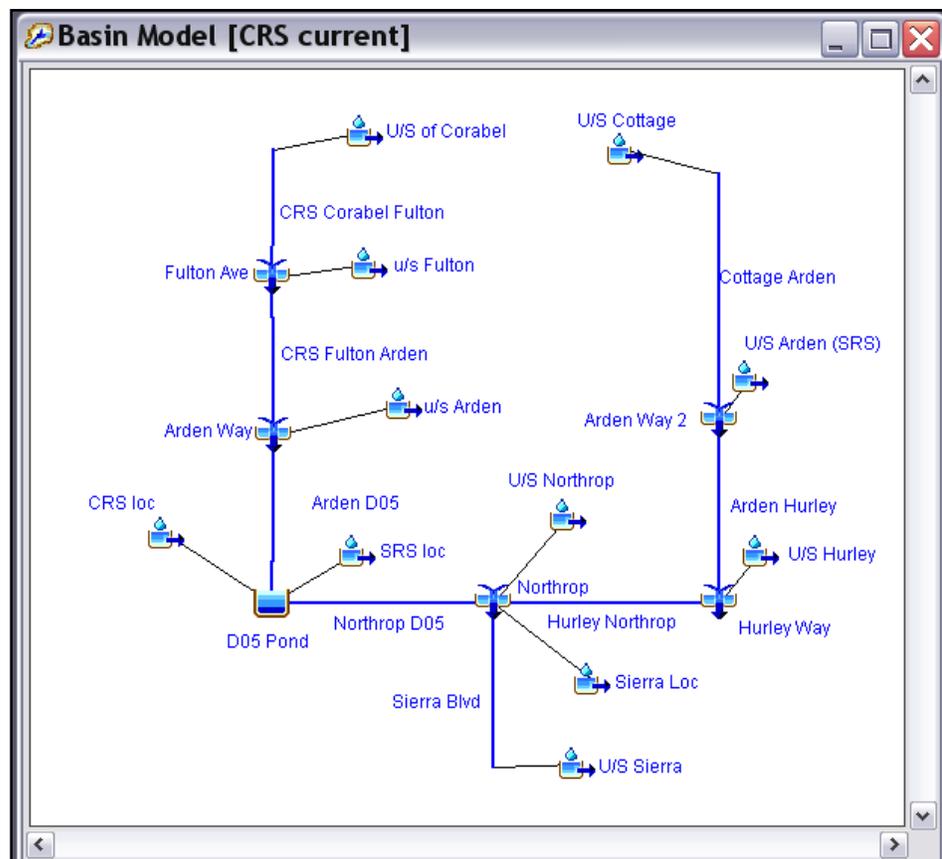


Figure 20. Complete Basin Model of CRS/SRS Watershed

Additional model components required for analysis of the with-project condition include a model of the detention pond, a model of the diversion, a model of the floodwall, and a model of the impact of raising (elevating) the structures. HEC-HMS includes a simple detention model. To use this, the analyst can define a storage-outflow relationship for the pond and outlet works. Similarly, HEC-HMS includes a diversion model; the properties of the diversion can be described with a function that predicts flow into the diversion channel, given the flow in the channel at the point of diversion.

Modeling the floodwall alternative with HEC-HMS creates a technical challenge. For the without-project condition, the channel was modeled in HEC-HMS with the Muskingum-Cunge channel routing method; channel geometry and roughness values were available from previous studies. If the channel is modified by the proposed plan, the flow-frequency function may change slightly, especially if the modification limits spill into the floodplain. The spill creates a storage effect that is similar to the effect of a detention pond. Consequently, eliminating spill will remove storage in the watershed and perhaps increase downstream flows. To account for this, the geometric data used in the routing model, specifically the eight-point cross section, can be modified to represent the floodwall.

However, the most significant impact of the floodwall - the modification to the rating function - cannot be simulated well with HEC-HMS. Thus, for this study an HEC-RAS model of the channels was developed and used to compute stages. Rating curves were computed using HEC-RAS for both the without- and with-project condition. In each case, the appropriate flows from HEC-HMS were used as input to the hydraulic model. The floodwall also alters the stage-damage function in HEC-FDA, up to the point where the floodwall overtops.

Similarly, simulating the impact of elevating the structures cannot be accomplished with HEC-HMS. This raising will have an insignificant impact on the frequency function, as it will do little to change watershed runoff or river routing characteristics. Instead, it will alter the damage incurred as water reaches a specified elevation in the floodplain. This is represented by the elevation-damage function, an input to the EAD computations. Thus, modeling this alternative is accomplished by changing no features of the HEC-HMS model, but by changing the economic function used by HEC-FDA.

## **Application**

Here is how the analyst completed the steps shown to define the required flow-frequency functions:

1. **Meteorologic models.** The required frequency storms were developed using the HEC-HMS option for such. Depths for various durations for the eight hypothetical events were entered in the appropriate forms; [Figure 13](#) is an example of this.
2. **Basin model for the without-plan condition.** The without-plan condition model was developed as described in Chapter 2. However, for this study, the entire watershed was modeled rather than just a portion. Note that the study team here had to account for the forecasted land use change in the watershed by creating two models: one for current without-plan condition and another for future without-plan condition. Runoff from the hypothetical events was computed using both models.
3. **Without-plan flow-frequency function.** In Chapter 3, a without-plan, current development frequency curve was adopted. HEC-HMS was used to develop the without-project, future development frequency curve. Both curves are shown in [Table 21](#). These frequency functions were used as input to HEC-FDA to compute without-plan EAD. The HEC-FDA program includes an algorithm to account for the changing EAD as watershed conditions change and to compute the present value and equivalent average annual value of the inundation damage. Uncertainty in the frequency curves is accounted for in the EAD computations.

**Table 21.** Without-Project Flow Frequency Curve at Arden Way

AEP	Peak flow for current development condition (cfs)	Peak flow for future development condition (cfs)
0.500	285	308
0.200	450	481
0.100	571	608
0.040	715	760
0.020	826	876
0.010	944	999
0.004	1,060	1,120
0.002	1,230	1,296

4. **Basin model for Alternative 1.** The basin model for Alternative 1 was developed, for both current and future condition. To do so, the analyst duplicated the subbasins and routing reaches included in the without-plan basin models, with the same model parameters and initial conditions.

The analyst added the proposed forty-acre detention pond, using the reservoir element, to these basin models, as shown in [Figure 21](#). To specify the properties of the detention, the analyst selected the reservoir element in the basin model map and then edited the

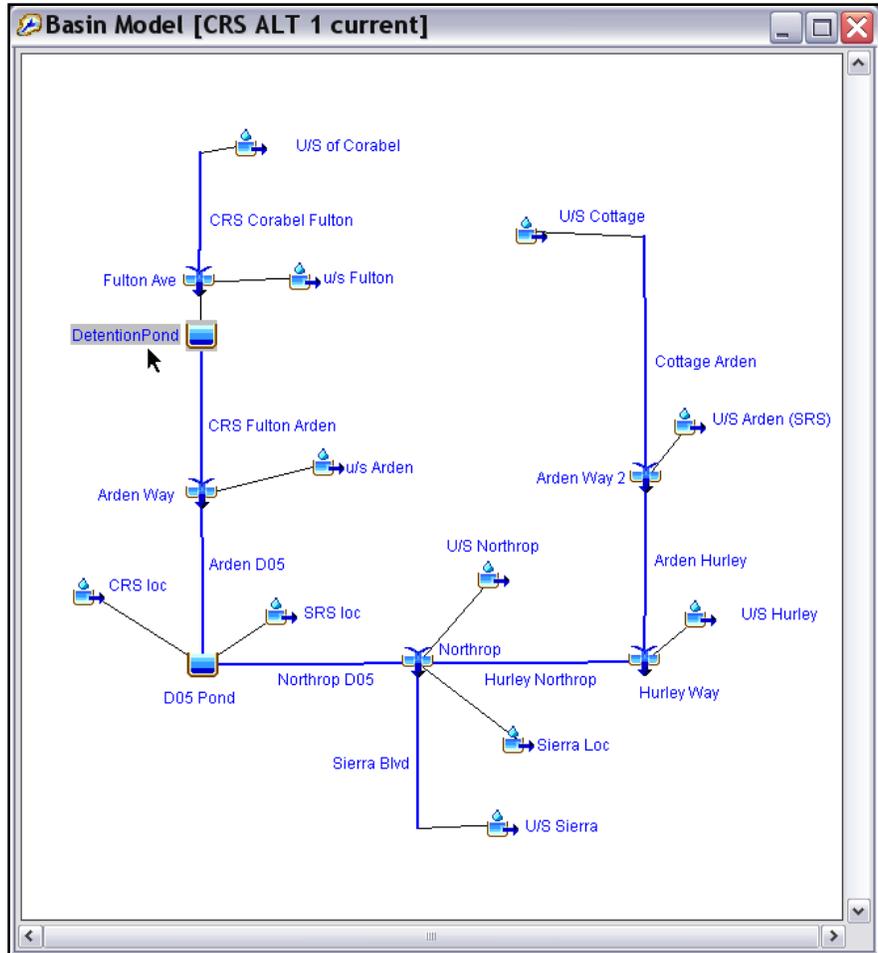


Figure 21. Basin Model for Alternative 1

element properties in the **Component Editor**. As shown in Figure 22, the **Outflow Curve** reservoir routing method was chosen. When this method is chosen the analyst must compute the outflow

Reservoir		Options
<b>Basin Name: CRS ALT 1 current</b>		
<b>Element Name: DetentionPond</b>		
Description:	40 acre inline detention pond	[Icon]
Downstream:	CRS Fulton Arden	[Icon]
Method:	Outflow Curve	[Icon]
Storage Method:	Elevation-Area-Discharge	[Icon]
Elev-Area Function:	D05 Pond(CRS ALT 1 current)	[Icon]
Elev-Dis Function:	D05 Pond(CRS ALT 1 current)	[Icon]
Primary:	Elevation-Discharge	[Icon]
Initial Condition:	Inflow = Outflow	[Icon]

Figure 22. Component Editor for Specifying Properties of Detention Pond

from the reservoir for a range of elevations and storage volumes. Other reservoir routing options include **Outflow Structures** and **Specified Release**. When **Outflow Structures** is selected, the analyst must define an elevation-storage or elevation-area relationship and enter properties for outlet structures. Outflow structures in HEC-HMS include low-level outlets, culverts, spillways, pumps, and dam overtopping. When **Specified Release** is selected, the analyst must define an elevation-storage or elevation-area curve and a time-series of outflow from the reservoir. This option is useful when calibrating a model and observed releases from the reservoir are known. In this example, the analyst selected the **Elevation-Area-Outflow** storage method. Then the analyst selected the elevation-area and elevation-discharge curves associated with the reservoir. The specified elevation-area relationship is used by the program to compute the storage-volume relationship of the pond. The pond design considered is an excavated pond, so the required relationship was determined with solid geometry computations completed in a spreadsheet program. The elevation-discharge curve contains the total outflow from the pond for the corresponding elevations. Finding these values was a bit more difficult, as it required modeling the hydraulic performance of the pond's outlet and emergency spillway. The spillway was modeled in a spreadsheet, using the weir equation, to develop the elevation-discharge relationship.

Modeling the performance of the detention pond's normal outlet was more difficult, as the performance of that outlet depends upon the flow condition. The outlet proposed is a culvert. Culverts can flow under inlet control or outlet control. In the first case, the discharge through the culvert, and hence the outflow from the detention, is a function of the cross-sectional area of the culvert, the inlet configuration, and the headwater elevation. In the second case, the discharge is a function of the tailwater elevation (which in turn, may be a function of the discharge) and the properties of the culvert, including slope, roughness, and length. In this case, the analyst used nomographs to develop the rating for the culvert. Another option would be to let HEC-HMS compute the outflow from the reservoir. The reservoir element has options to let the analyst define outlet structures; culvert and spillway outlets are included.

5. **With-plan flow-frequency function for Alternative 1.** After the basin models with the detention pond in place were developed, the analyst ran the models with eight hypothetical storms and computed runoff peaks for the present and future conditions. In Chapter 3 the analyst created a separate **Basin Model** for each hypothetical storm because the initial loss differed for each event. This time, the analyst chose to use one basin model and change the initial loss

before each run. To simplify this procedure, the parameter values were altered in the global parameter editor. To do so, from the **Parameters** menu, point to **Loss**, and click **Initial and Constant**. The values were then set in the resulting window, as shown in [Figure 23](#).

Subbasin	Initial Loss (IN)	Constant Rate (IN/HR)	Impervious (%)
CRS loc	0.2	0.23	90
Sierra Loc	0.2	0.23	60
SRS loc	0.2	0.23	90
u/s Arden	0.2	0.23	50
U/S Arden (SRS)	0.2	0.23	60
U/S Cottage	0.2	0.23	30
u/s Fulton	0.2	0.23	60
U/S Hurley	0.2	0.23	40
U/S Northrop	0.2	0.23	50
U/S of Corabel	0.2	0.23	50
U/S Sierra	0.2	0.23	25

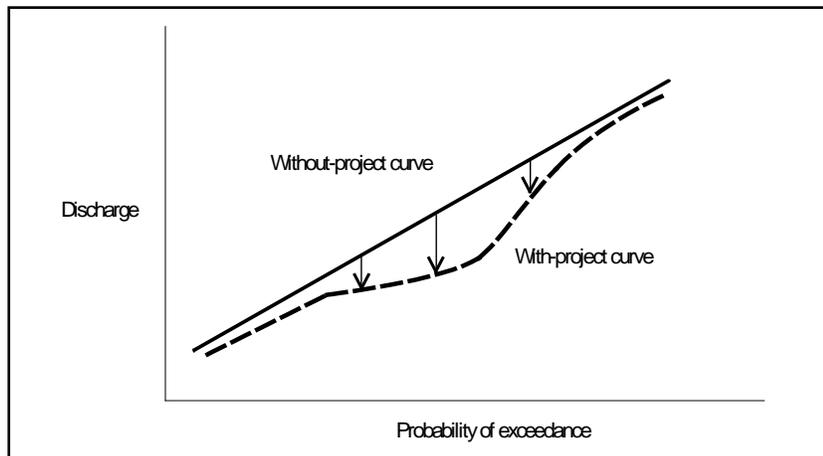
**Figure 23.** Alter Parameter Values using the **Global Parameter** Editor

The peak runoff results for each hypothetical storm are shown in [Table 22](#). Column 2 shows the flow-frequency curve with the

**Table 22.** Alternative 1 Flow-Frequency Functions at Arden Way for Current and Future Land Development Conditions

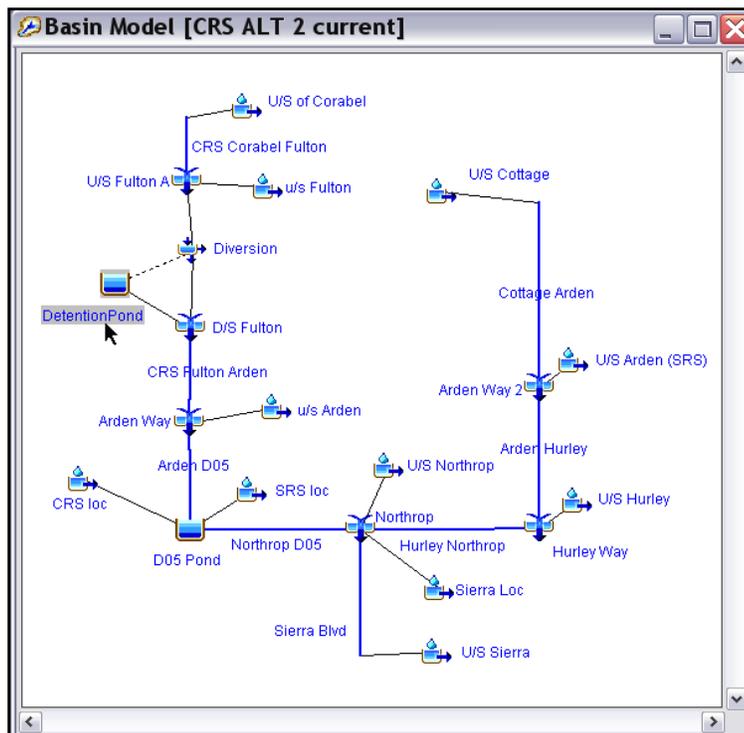
AEP	Peak Flow for Current Land Development Condition (cfs)	Peak Flow for Future Land Development Condition (cfs)
0.500	231	249
0.200	357	375
0.100	441	464
0.040	540	569
0.020	616	649
0.010	694	731
0.004	771	812
0.002	936	1,004

detention pond in place, for the current development condition. Column 3 shows the flow-frequency curve with the detention pond in place, for the future development condition. Comparing these curves to the without-project condition curves, as shown in [Table 16](#) in Chapter 3, it is evident that the flow from each hypothetical storm is reduced. This is conceptually shown in [Figure 24](#).



**Figure 24.** Conceptual Effect of Alternative 1 on the Flow-Frequency Curve

6. **Basin model for Alternative 2.** Alternative 2 is similar to Alternative 1, but the study team thought that the size of the detention area might be reduced from forty acres to twenty acres if a diversion was included to bypass the detention, permitting flow equal the 0.04 AEP event. This configuration was included in the basin model by again beginning with the without-project subbasin model and adding a diversion, a twenty-acre detention area, and a junction downstream of the pond. The flows that bypass the pond are retrieved and added to the downstream channel at this junction. [Figure 25](#) illustrates the resulting basin model.



**Figure 25.** Basin Model for Alternative 2

To describe the diversion, a performance function must be entered in the paired data **Component Editor**, illustrated in [Figure 26](#). This shows how much flow enters the diversion channel, given flow entering the diversion control structure. For this alternative, the structure is designed to divert portions of the flows that exceed the 0.04 AEP event at Fulton Avenue, or approximately 600 cfs.

Paired Data		Table	Graph
Inflow (CFS)		Diversion (CFS)	
	0.0		0.0
	600.0		0.0
	700.0		100.0
	800.0		200.0
	900.0		300.0
	1300.0		700.0

**Figure 26.** Inflow-Diversion Function for Diversion Element

For this alternative, the detention is described as before, specifying the elevation-area-outflow relationship for the reduced capacity pond.

The junction added downstream of the detention permits retrieval of the diverted hydrograph, which is added to the outflow hydrograph from the pond. As the outflow from the pond is less than the unregulated flow into the pond, the downstream flow rate will be less than the without-project flow rate.

- 7. With-plan flow-frequency function for Alternative 2.** After the basin models with the detention and diversion in place were developed, the analyst ran the models with the eight hypothetical storms and computed runoff peaks for the present and future conditions. The results are shown in [Table 23](#).

**Table 23.** Alternative 2 Flow-Frequency Functions at Arden Way for Current and Future Land Development Conditions

AEP	Peak Flow for Current Land Development Condition (cfs)	Peak Flow for Future Land Development Condition (cfs)
0.500	285	309
0.200	445	483
0.100	569	610
0.040	713	762
0.020	824	868
0.010	928	977
0.004	988	1,041
0.002	1,063	1,140



The properties of the diversion were specified by the analyst in the inflow-diversion function table. The rating in this case was derived by developing an HEC-RAS model of the spillway with proposed dimensions and running that model with a range of steady flows to determine the flow rate into the diversion. The resulting diversion performance is shown in [Table 24](#).

**Table 24.** Flow Diversion at Fulton Avenue

Inflow to diversion (cfs)	Diverted flow (cfs)
500	0
700	100
800	150
1,200	300
2,000	600

9. **With-plan flow-frequency function for Alternative 3.** After the basin models with the diversion in place were created, the analyst ran the models with the eight hypothetical storms and computed runoff peaks for the present and future conditions. The results are shown in [Table 25](#).

**Table 25.** Alternative 3 Flow-Frequency Functions at Arden Way for Current and Future Land Development Conditions

AEP	Peak Flow for Current Land Development Condition (cfs)	Peak Flow for Future Land Development Condition (cfs)
0.500	285	309
0.200	445	483
0.100	569	610
0.040	713	751
0.020	804	830
0.010	882	911
0.004	957	989
0.002	1,071	1,113

10. **Basin model for Alternative 4.** As noted above, the changes in channel geometry due to the proposed floodwall will have little impact on the flow-frequency curves in the basin. Instead, the most significant impact will be in changes to the rating curves and the stage-damage curves used with HEC-FDA for EAD computations. Nevertheless, for completeness, the analyst created basin models for Alternative 4 to compute flow-frequency curves. These were identical to the without-plan models in their configuration, but the analyst modified the eight-point cross-section representation for the Muskingum Cunge routing reaches to represent the floodwall.
11. **With-plan flow-frequency function for Alternative 4.** After the basin models with the modified channel data were created, the analyst ran the models with the eight hypothetical storms and

computed runoff peaks for the present and future conditions. The results at Arden Way are shown in [Table 26](#). Note that Arden Way is the upstream end of the proposed floodwalls; therefore, the frequency function is the same as the without-project condition at that point in the system.

**Table 26.** Alternative 4 Flow-Frequency Functions at Arden Way for Current and Future Land Development Conditions

AEP	Peak Flow for Current Land Development Condition (cfs)	Peak Flow for Future Land Development Condition (cfs)
0.500	285	309
0.200	445	483
0.100	569	610
0.040	713	762
0.020	824	879
0.010	941	1,003
0.004	1,057	1,124
0.002	1,226	1,301

12. **Basin model for Alternative 5 and with-plan flow-frequency function.** As noted above, Alternative 5 does not alter the frequency functions, so modeling its impact does not require modifying any of the features of the HEC-HMS models. Instead only the elevation-damage data that are specified with HEC-FDA will be changed. Therefore, the basin model for Alternative 5 is identical to the basin model for the without-plan condition, and the with-project frequency functions are identical to the without-project functions.

For completeness, and to avoid any confusion in the future, the analyst created basin models for Alternative 5. There are multiple ways to copy basin models in HEC-HMS. As shown in [Figure 29](#), the analyst place the mouse over the existing basin model name in the **Watershed Explorer** and clicked the right mouse button. In the resulting pop-up menu the analyst selected the **Create Copy** menu option.

## Processing of Results

After all original alternatives were analyzed with HEC-HMS, the required frequency functions were available for EAD computations. Additional analyses with HEC-RAS were required to establish the elevation-discharge functions, and an extensive data collection effort was required to establish the elevation-damage functions. Then computer program HEC-FDA was run, and the EAD values for the CRS/SRS watershed were computed. [Table 27](#) shows the results. During the EAD computations, a description of the uncertainty was included in the form of an estimate of

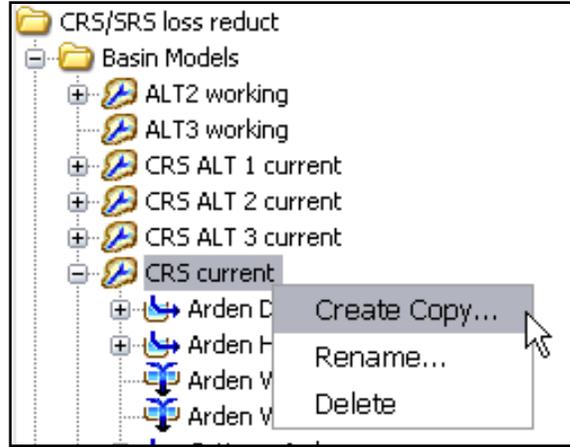


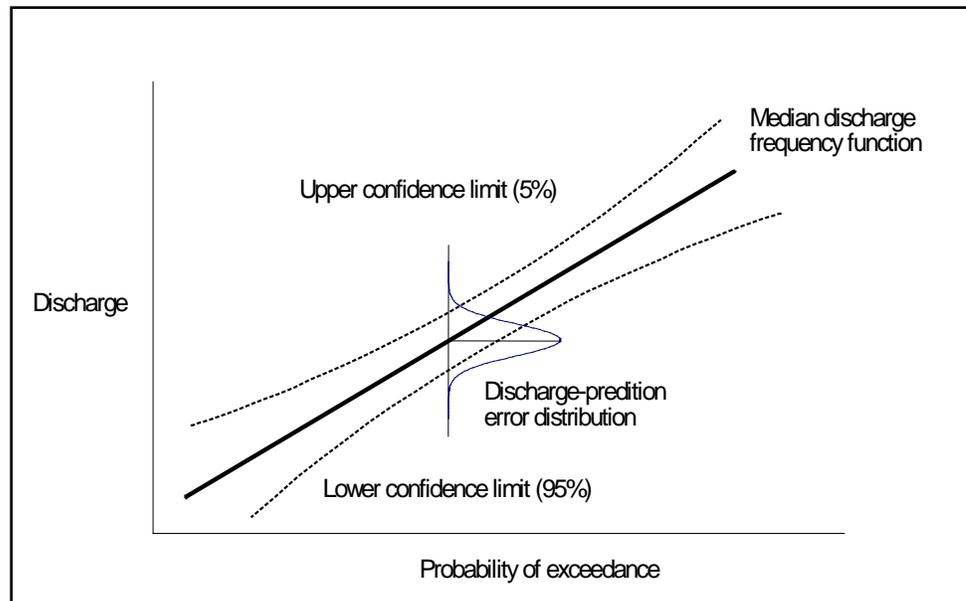
Figure 29. Copying a Basin Model

Table 27. EAD and Inundation Reduction Benefit of Alternatives

Condition	EAD (\$1,000)	Inundation Reduction Benefit (\$1,000)
Without-project	606	---
Alternative 1	228	378
Alternative 2	322	284
Alternative 3	477	129
Alternative 4	501	105
Alternative 5	559	47

the equivalent years of record. Guidelines from EM 1110-2-1619 (USACE, 1996) were used to estimate the equivalent years of record to be ten. The result is that a probability distribution is used to estimate the flow for a given frequency event, as illustrated in Figure 30. Based upon this distribution, confidence intervals can be drawn to illustrate the associated uncertainty in the values.

The frequency functions computed for the proposed plans provide other information useful for systematic comparison of the plans and for selection of the optimal plan. For example, the probability of capacity exceedance of each plan (commonly referred to as the level of protection) can be found from the frequency functions. With this, the long-term risk associated with each plan can be computed. EM 1110-2-1619 (USACE, 1996) describes the computational methods. EM 1110-2-1619 (USACE, 1996) calls also for modeling the uncertainty in the frequency functions; this uncertainty is a consequence of lack of data and lack of certainty about models and parameters. With models of the uncertainty, sampling methods will permit computation of conditional non-exceedance probabilities. For example, with the uncertainty models and sampling, the analyst can determine the probability that a floodwall designed to eliminate flooding due to the 0.01 AEP event will, in fact, be overtopped by the 0.04 AEP event. This error in design is not a consequence of



**Figure 30.** Flow-Frequency Function with Uncertainty Distribution

carelessness; rather, it is a consequence of uncertainty about the true value of the 0.01 AEP water surface elevation.

## Summary

The goal in the example was to decide which inundation-reduction alternative provides the greatest net benefit in the watershed. A required component of this is to develop flow-frequency functions for each alternative. By modifying the watershed characteristics and geometry within HEC-HMS, the analyst was able to develop the needed functions. These functions were then used with HEC-FDA (HEC, 2008) to compute expected annual damage. To complete the net benefit calculations, the next step will be to estimate the annual cost of each alternative and solve Equation 4. The alternative with the greatest net benefit will then be recommended.

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U.S. Army Corps of Engineers, Hydrologic Engineering Center. 2008. *HEC-FDA Flood Damage Reduction Analysis, User's Manual, Version 1.2.4.*, CPD-72. Hydrologic Engineering Center, Davis, CA.

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**USACE, 1996.**

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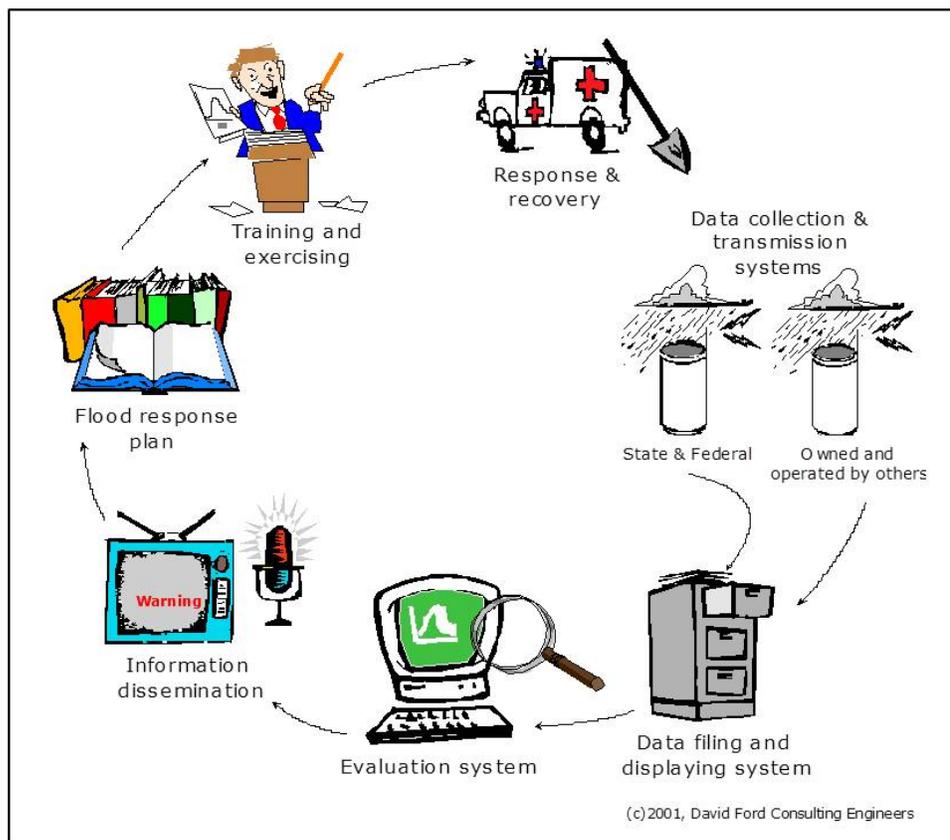
# CHAPTER 5

## Flood Warning System Planning Studies

### Background

#### Overview of Flood Warning Systems

What is it? A flood warning system (FWS) is an integrated package of data collection and transmission equipment, forecasting models, response plans and procedures, and human resources. Together these increase flood warning (lead) time. With increased lead time, public officials and citizens can take actions to reduce damage and to protect lives. A FWS is classified as a nonstructural damage reduction alternative: it will not reduce flood flows or flood stages, but it will reduce damage incurred due to a specified stage. A complete flood warning system includes components that are illustrated conceptually in [Figure 31](#).



**Figure 31.** Components of Flood Response and Emergency Preparedness System (©2001 David Ford Consulting, Inc., used with permission)

Flood warning begins with data collection and transmission. The instruments in the data collection and transmission system measure current rainfall depths, water levels, and other indices of watershed conditions. Rainfall commonly is measured in a FWS with a tipping-bucket rain gage. Such a gage records the clock time at which a small bucket of known volume within the gage fills and tips. From this time series, the rate of and cumulative depth of rainfall can be calculated. For water-level observation, a pressure transducer or shaft encoder is commonly used. The former measures the pressure at the bottom of the channel, from which the depth can be inferred. The latter measures the distance from a fixed reference point above the channel to the water surface; from this, the depth can be inferred.

The environmental condition measurements form the basis of flood-threat recognition. In the simplest case, the observed water level can be compared with a predetermined threshold. This threshold may be the elevation at which water will flow out of bank and damage property or threaten lives. If the level reaches the threshold, then a warning is issued.

The comparison of observations to thresholds may occur at the observation site or elsewhere. In the latter case, the data transmission components relay the observations to a central site - a base station at a flood warning center - for analysis. The data transmissions may use radio, microwave, dedicated or leased telephone lines, or satellite. The most common transmission type for local FWS in the United States is UHF or VHF radio broadcast using the ALERT standard. This standard was established by NWS and its cooperators (NOAA, 1997; NHWC, 2001). With an ALERT-based FWS, the transmitters send a signal for each "event". An event, in this case, is a water level rise or fall of a pre-selected distance (typically 0.1 foot) or a tip of the rain gage (about 0.04 inches of rainfall). The transmission is a forty-bit radio signal that identifies the sensor and includes the current rainfall depth or water-level reading in coded form. Transceivers (repeaters), situated on high ground with line-of-sight to gages and base stations, reconstitute and forward the signals, thus increasing the areal extent of the system. Equipment for ALERT systems is available commercially, off-the-shelf, or from various vendors.

The data transmitted from the sensors are received, encoded, managed, and analyzed at a central site. This site is equipped with a base station that includes an antenna and cabling; a modem, radio, or satellite receiver; a decoder - a microprocessor that converts the signal from the gages into digital data appropriate for filing or analyzing with a computer; and a dedicated computer with add-ons and peripheral devices.

The rainfall and streamflow data are managed at the flood warning center with base station software: database-management software that translates and stores data for subsequent analysis and reporting and visualization

software that expedites examination of the data for threat detection. Commercial, off-the-shelf base station software systems are available. If the data are needed at more than a single site to ensure proper response, a data dissemination system redistributes the observations from the central site to other sites, using, for example, a local area network or a web site.

The evaluation system includes tools to display and inspect the incoming data to determine if a flood threat exists and tools to forecast occurrence of a future threat. For example, an evaluation system might be configured to compare observed water levels at the gage on Updah Creek to the level at which water is known to overflow and inundate structures nearby on Penny Lane. If the level reaches that threshold, then a warning is issued and appropriate actions are taken.

The evaluation system, at a minimum, includes (1) stage or rainfall depth thresholds that will be used to identify flood threats, and (2) the method by which the threat recognition plan is implemented. Alternatives for this threat recognition include (1) an operator-monitored procedure in which a human operator inspects the incoming data, compares the values with the thresholds identified in the threat recognition plan, and takes appropriate action, or (2) an automated procedure, in which software automates examination and comparison of the incoming data to rules of the threat recognition plan. If a threat exists, or is expected, an information dissemination system communicates an appropriate warning, first to emergency responders, and then to the public.

In many cases, threat recognition based upon observations alone will not provide adequate time to respond appropriately. In those cases, a forecasting system may extend the lead time by using mathematical models of watershed and channel behavior, along with the rainfall and water level observations, to predict future response of the watershed and channel. Alternatives to mathematical models for forecasting include (1) empirical forecasting models, which predict future stages based upon patterns inferred statistically from analysis of historical stages, historical rainfall depths, or both, and (2) conceptual models, which use a mathematical representation of underlying physical properties to forecast future stages, given observations of historical stage, historical rainfall, or both.

When a threat is recognized, either through observation or prediction, and warnings are issued, actions begin in earnest to protect lives and property. For efficiency, these actions should follow procedures spelled out in a flood response plan.

## **Authority and Procedural Guidance**

USACE activities in flood warning are authorized by:

- The Flood Control Act of 1936.
- Section 206 of the Flood Control Act of 1960.
- Executive Order 11988.
- Section 73 of Public Law 93-251.

The following USACE guidance is particularly relevant to flood warning system planning and design:

- ER 1105-2-100 (USACE, 2000) provides guidance and describes procedures for all civil works planning studies.
- EM 1110-2-1417 (USACE, 1994) describes procedures for flood runoff analysis, which is critical to FWS planning.
- ETL 1110-2-540 (USACE, 1996) describes components of flood warning systems including preparedness programs and the general operation and maintenance procedures required for accurate flood warning.

## Study Objectives

Hydrologic engineering studies play a critical role in providing information for planning, designing, implementing, and operating a flood warning system. Studies are required to:

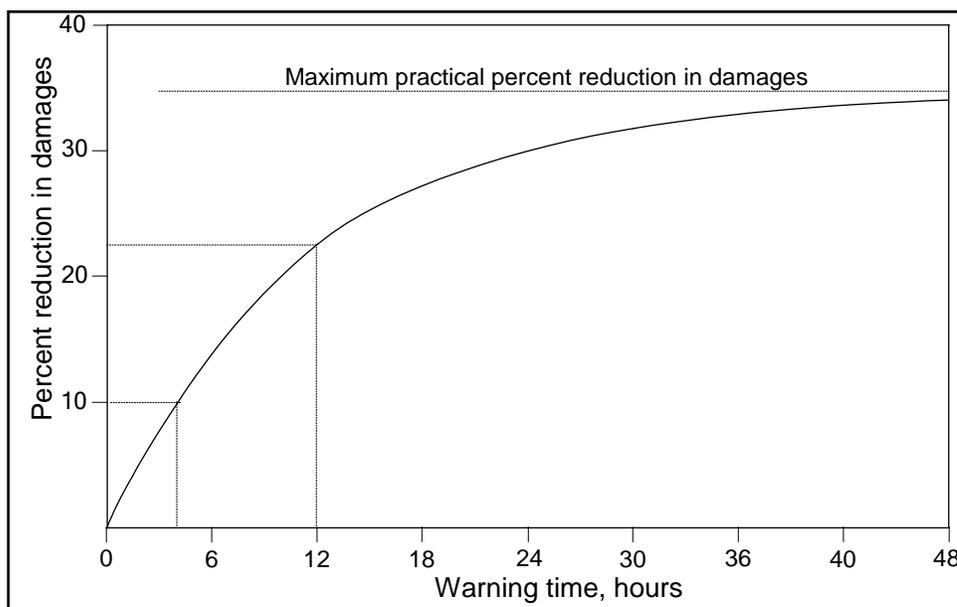
- Identify vulnerable areas for which flood warning is an effective flood damage reduction alternative.
- Establish rainfall and water-level thresholds for threat recognition.
- Link the thresholds to the vulnerability assessment, so that those who should be notified can be identified.
- Identify locations for rainfall and water-level sensors.
- Develop and provide the tools for forecasting.

For every FWS in which the USACE plays a role, the first step is to assess the benefit of the FWS, so that the federal interest in implementation can be determined. For this federal interest to exist, the system's benefit must exceed its cost. The benefit of a flood warning system is due to actions taken or actions deferred as a result of the warnings issued. The benefit can be categorized as either tangible benefit, which can be assigned a monetary value, or intangible benefit, which cannot be assigned a monetary value but may be otherwise quantified or described (IWR, 1994). The FWS benefit may be categorized further as direct or indirect. Direct benefit accrues to floodplain occupants who are "protected" by the

system. This benefit includes inundation-reduction benefit and emergency response and recovery cost avoided. Conversely, indirect benefits result from externalities: impacts outside the floodplain or impacts secondary to the response system's design goals. Assessment of the direct tangible benefit is the subject of this chapter.

## Study Procedures

The guidance cited above directs planners to assess the FWS benefit, but it does not stipulate how this is to be done. A common approach is to use the so-called Day Curve (USACE, 1996), which is shown in [Figure 32](#).



**Figure 32.** Day Curve

The Day Curve was developed from empirical analysis of property in a floodplain, considering the value and spatial distribution of property and the likely response of property owners when warned. The Day Curve estimates damage reduction (as a percentage of total inundation damage) as a function of the warning (mitigation) time - the time available for citizens and emergency responders to protect lives and property. For example, the curve shows that if the warning time is zero hours, a FWS provides no direct tangible benefit. If the warning time is twelve hours, the curve predicts that the damage will decrease by 23 percent. The curve also suggests that no matter how great the warning time, the maximum possible reduction is about 35 percent of the total damage due to the flood. This is logical, as some property, including most structures, simply cannot be moved.

To use the Day Curve, the analyst must estimate the mitigation time attributable to the FWS. ETL 1110-2-540 (USACE, 1996) provides guidance for this. It defines first a maximum potential warning time; this

is the time that passes between the first detectable or predictable precipitation and the time at which the stage exceeds the threshold for damage or threat to life at a critical location. The actual warning time, better known as mitigation time, is the maximum potential warning time less (1) the time required actually to detect and recognize the threat, (2) the time required to notify emergency responders and for those responder to make decisions about actions, and (3) the time required to notify the public. For example, if the maximum potential warning time is ten hours, and if six hours are required for detection, one hour required for emergency responder notification and decision making, and one hour required for public notification, then the actual mitigation time is only two hours. If the FWS reduces the detection time to four hours, the mitigation time increases to four hours.

The detection and notification times depend upon the design of the FWS, the equipment included, the efficiency of the response plans, and so on. The maximum potential warning time, on the other hand, depends on the characteristics of the watershed and storms that occur. A watershed model such as HEC-HMS will provide the information to estimate that. The case study that follows illustrates this.

## **Case Study: Estimating Benefit of a Proposed FWS for an Urban Watershed**

### **Objective of Hydrologic Engineering Study**

A large city in the southeastern United States recently has experienced significant flooding in dense urban watersheds, as illustrated in [Figure 33](#). Local newspapers described floods of 1995 and 1997 as having "deadly force". Flood insurance claims for the 1995 event totaled \$4 million, and an additional \$1 million was issued as loans to repair damaged property.



**Figure 33.** Urban Flooding in the Watershed for which FWS is Proposed

The 1997 flood caused \$60 million in damage and took three lives. Total rainfall in the area ranged from 3.87 to 9.37 inches in the 1995 event, and rainfall depths of as much as 11.40 inches in 24 hours were reported in the 1997 event. High water marks indicated that water levels increased by as much as twenty feet in some locations.

Citizens asked USACE to assist with planning, designing, and implementing damage reduction measures. Structural measures, such as those described in Chapter 4, are proposed and will be evaluated. However, for completeness, and to offer some relief in portions of the watershed for which structural measures may not be justified, USACE planners will consider flood warning.

### **Decision Required**

A FWS design has been proposed by a flood-warning specialist. The reconnaissance-level design includes preliminary details of all components shown in [Figure 31](#). To permit comparison of the proposed FWS with other damage-reduction alternatives and to determine if the federal government should move ahead with more detailed design of the FWS, the economic benefit must be assessed. To do so, USACE planners will use the Day curve, and for this, they must determine the warning time in the watershed, without and with the proposed FWS.

As no system exists currently, the expected warning time without the FWS project is effectively zero. The warning time with the FWS will be computed as potential warning time less the detection and notification times. The response and notification times have been estimated as a part of the design, based upon the expected performance of the system components. Thus, the missing components are the recognition and maximum potential warning times. With these times, the warning time will be calculated as the maximum potential warning time less the recognition time.

### **Watershed Description**

One of the critical watersheds for which the FWS will provide benefit is a 4.7 square mile watershed. The watershed is located in a heavily urbanized portion of the city. It has been developed primarily for residential and commercial uses. Several large apartment complexes and homes are adjacent to the channels.

Physical characteristics of the watershed were gathered from the best available topographic data. The watershed is relatively flat with a slope of 0.003. The length of the longest watercourse is 3.76 miles. The length to the centroid is 1.79 miles. The percentage of impervious area is approximately 90 percent.

## Procedure for Estimating Warning Time

The maximum potential warning times varies from storm to storm and location to location in a watershed. For example, if damageable property in the watershed is near the outlet, and if a short duration thunderstorm is centered near the outlet, the maximum potential warning time will be small. On the other hand, if the storm is centered at the far extent of the watershed or if a forecast of the precipitation is available before it actually occurs (a quantitative precipitation forecast), the maximum potential warning time for this same location will be greater.

Likewise, the watershed state plays a role in determining the maximum potential warning time. If the watershed soils are saturated, the time between precipitation and runoff is less than if the watershed soils are dry. Accordingly, for this study, the analyst decided to compute an *expected* warning time for each protected site, as follows:

$$E[T_w] = \int T_w(p) dp \quad (5)$$

where:

$E[T_w]$  = the expected value of warning time

$P$  = AEP of the event considered

$T_w(p)$  = the warning time for an event with specified AEP

The actual value will be computed with the following numerical approximation:

$$E[T_w] = \sum T_w(p_i) \Delta p_i \quad (6)$$

where:

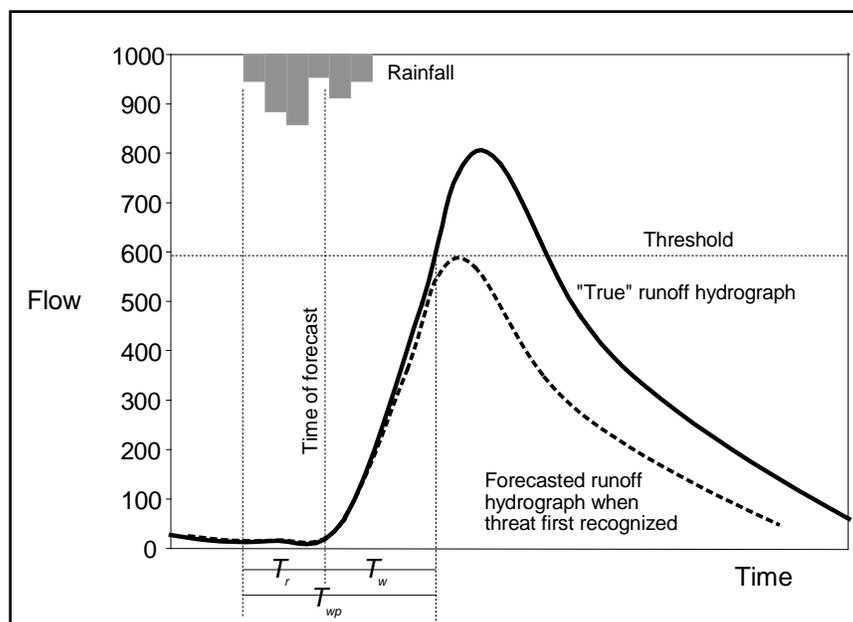
$p_i$  = AEP of event  $i$

$T_w(p_i)$  = the warning time for event  $i$

$\Delta p_i$  = range of AEP represented by the event

Storm events for this analysis are defined by frequency-based hypothetical storms, as described in Chapter 3. With this procedure, the analyst will consider the system performance for the entire range of possible events.

Figure 34 illustrates how the warning time will be computed for each event. First, the analyst used the entire rainfall hyetograph with HEC-HMS to compute a "true" runoff hydrograph, as shown by the solid line in Figure 34. This is the runoff that will occur when the entire rainfall event has occurred. The time that passes between the onset of the rainfall and the exceedance of the threshold is the maximum potential warning time,  $T_{wp}$ , as shown.



**Figure 34.** Warning Time Components

The question that must be answered is this: If the FWS is implemented, when will the system operators be able to detect or forecast that exceedance? That is, what is  $T_r$  in Figure 34 - the time that passes before the threshold exceedance can be detected (recognition time)? Without the FWS,  $T_r$  will approach  $T_{wp}$ , and little or no time will remain for notification and action. The maximum mitigation time,  $T_w$ , is then the difference between  $T_{wp}$  and  $T_r$ .

To estimate  $T_r$  and  $T_{wp}$  for each location in the watershed for which warnings are to be issued, the analyst did the following:

1. Identified the flow threshold at the site, using topographic information, elevation data for vulnerable structures and infrastructure, and simple channel models.
2. Selected a time interval,  $\Delta T$ , to represent the likely interval between successive forecasts or examination of data for identifying a threat with the FWS. With an ALERT system, data constantly are collected, transmitted, received, and analyzed. For simplicity in the analysis, the analyst used  $\Delta T$  of thirty minutes and interpolated as necessary.
3. Set up an HEC-HMS model of the watershed, as described below.
4. Ran HEC-HMS for each of the selected frequency-based storms. For each storm, the program was run recursively, using progressively more rainfall data in each run. For example, in the first run with the 0.01 AEP rainfall event, only sixty minutes of the storm data were used. With these data, a hydrograph was computed

and examined to determine if exceedance of the threshold would be predicted. If not, thirty minutes of additional data were added, and the computations repeated. When exceedance of the threshold is detected, this defines the earliest detection time for each frequency storm. (Some interpolation was used.) For simplicity, the time of the computation in each case is referred to herein as the time of forecast (TOF). The difference in time from the start of the precipitation event to the TOF where an exceedance was detected is  $T_r$ .

5. This computation scheme simulates the gradual formation of a storm, observation of the data over time, and attempted detection of the flood event.
6. Ran HEC-HMS for each of the selected frequency-based storm using the entire precipitation event. The difference in time from the start of the precipitation event to the exceedance of the threshold is  $T_{wp}$ .
7. After finding the detection time and maximum potential warning time for each frequency-based event, computed the expected values, using Equation 6.

## Model Selection and Fitting

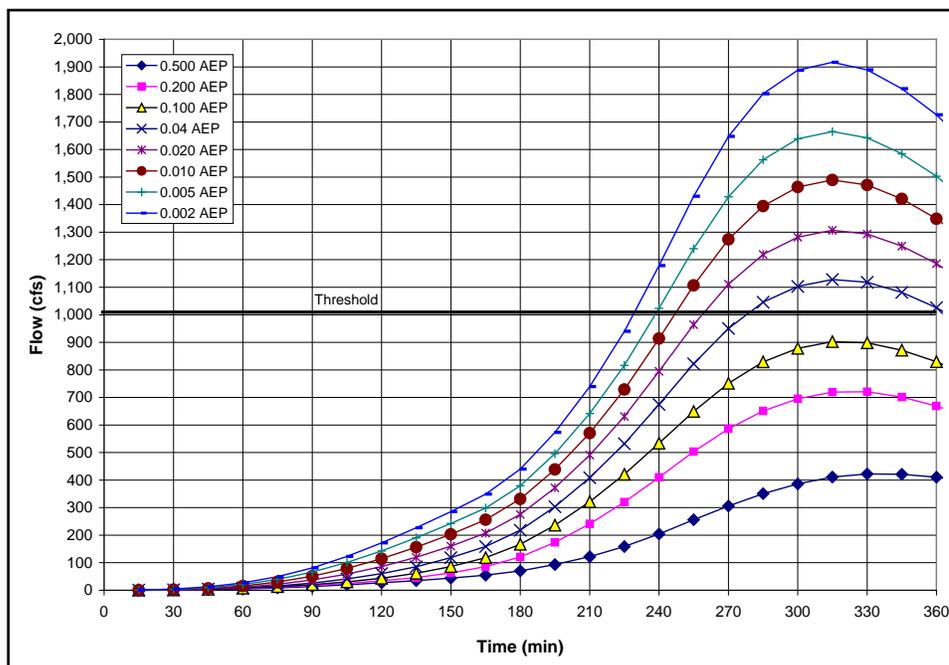
The analyst hoped to use HEC-HMS for computing runoff from the rainfall. However, the analyst found that model selection for assessment of the FWS benefit presented an interesting challenge. The goal of the study was to determine *if* a federal investment in developing a FWS was justified. The procedure for estimating the benefit requires an estimate of maximum warning time. Estimating the maximum warning time, as proposed above, requires a model of the watershed. However, developing such a detailed, well-calibrated model of the watershed requires a significant effort, which will not be justified if no federal interest exists. Accordingly, the analyst here developed a quick-and-dirty HEC-HMS model, using regional estimates of parameters. The SCS unit graph transform was used, with lag estimated from an empirical relationship. The initial and constant-rate loss method was selected, and the initial condition and constant-rate loss parameter were estimated with predictors similar to those described in Chapter 2. Rainfall DDF functions were defined by referring to appropriate publications from NWS.

## Application

The analyst reviewed available topographic information, channel models, and structure data and determined that the threshold for the downstream location in the first watershed of interest is 1,100 cfs. If the discharge rate

exceeds this, water will spill from the channel and threaten life and property. Thus a warning would be issued. The goal of the HEC-HMS application is to determine how far in advance a warning of exceedance can be issued.

To identify the detection and maximum potential warning times, the analyst created an HEC-HMS project with a single basin model, but with a large number of meteorologic models. The basin model included representation of all the subbasins, channels, and existing water-control measures. Each meteorologic model defined a portion of a frequency-based hypothetical storm. Eight design storms were used, ranging from the 0.500 AEP event to the 0.002 AEP event. The maximum duration of the storms was six hours. Figure 35 shows the computed hydrograph for the eight storms, using the rainfall for the entire six-hour duration. Events smaller than the 0.040 AEP event do not exceed the threshold. Column 2 of Table 28 shows the maximum potential warning time for each of the events: the time between initial precipitation and threshold exceedance.



**Figure 35.** Flow Hydrographs at the Critical Location using Entire Precipitation Events

To complete the runoff analysis as proposed, each of the hypothetical events was analyzed with increasing duration of rainfall data, starting with sixty minutes of data. The analyst increased the duration of data in thirty-minute blocks and computed a runoff hydrograph.

To create the meteorologic models for this, the analyst used HEC-HMS and a combination of HEC-DSSVue (HEC Data Storage System Visual Utility Engine) data management tools. First, the DSS file in which

**Table 28.** Sample Calculation of Warning Time

Annual Exceedance Probability of Storm	Maximum Potential Warning Time, $T_{wp}$ (min)	Detection Time, $T_r$ (min)	Maximum Mitigation Time <sup>1</sup> , $T_w$ (min)
0.500	— <sup>2</sup>	— <sup>2</sup>	— <sup>2</sup>
0.200	— <sup>2</sup>	— <sup>2</sup>	— <sup>2</sup>
0.100	— <sup>2</sup>	— <sup>2</sup>	— <sup>2</sup>
0.040	300	255	45
0.020	270	205	65
0.010	255	190	65
0.004	245	180	65
0.002	235	175	60

<sup>1</sup> Maximum mitigation time is  $T_{wp} - T_r$ . Time for notification, decision making, etc. will reduce time actually available for mitigation.

<sup>2</sup> Threshold not exceeded by event shown.

rainfall hyetographs from the total storm are stored was identified. To accomplish this, the analyst used Windows® Explorer to examine the contents of the HEC-HMS project folder. There the USACE analyst found a file with the extension ".dss". In each project, HEC-HMS stores all the computed and input time series for the project, including, in this case, the rainfall hyetograph for the six-hour design storm. Next, the analyst used one of the available tools for manipulating DSS files. In this case, the analyst used DSSUTL (DSS Utility software), and with it created duplicates of the DSS record in which the six-hour storm hyetograph is stored.

Records stored in a DSS file have a unique name (pathname) consisting of six parts. This so-called pathname serves as the primary key in the database, permitting efficient retrieval of records. The parts of the pathname are referred to as the A-Part, B-Part, and so on. Some parts serve a specific purpose. For example, the C-Part identifies the type of data stored in the record, and the E-Part identifies the reporting interval for uniform time series data. The F-Part may be assigned by a user, so the analyst assigned an F-Part unique to the duration of data available to each copy of the rainfall. For example, the sixty-minute sample of the 0.01 AEP event was assigned an F-Part of *01AEP60MIN*, while the 300-minute sample for the 0.002 AEP event was assigned an F-Part of *002AEP300MIN*. Then the analyst used DSSUTL with an editor compatible with DSS to delete values from the series, shortening each to include just the duration of rainfall required. For example, the final 300 minutes of the six-hour storm was deleted to create the sixty-minute sample, and the final 270 minutes was deleted to create the ninety-minute sample. Eleven such copies were made for each of the eight frequency based storms, for durations from sixty minutes to 360 minutes.

Next, a meteorologic model was created in HEC-HMS for each of the samples. To accomplish this, the analyst identified each shortened sample as a precipitation gage and used the gage to define a hyetograph. While

the series are, in fact, not gaged data, they may be used as such for runoff computations with HEC-HMS. (This useful feature also permits computation of runoff from any hypothetical storm types that are not included in HEC-HMS.) To create the precipitation gage time-series, from the **Components** menu, the analyst clicked **Time-Series Data Manager**. The **Time-Series Data Manager** (Figure 36) opens; the analyst selects a data type from the **Data Type** list - **Precipitation Gages**. To create a precipitation gage, click **New**, the **Create a New Precipitation Gage** window will open. Provide a name; click **Create**, the **Create a New Precipitation Gage** window closes. In order to add data to the precipitation gage, the analyst opened the precipitation gage **Component Editor**.

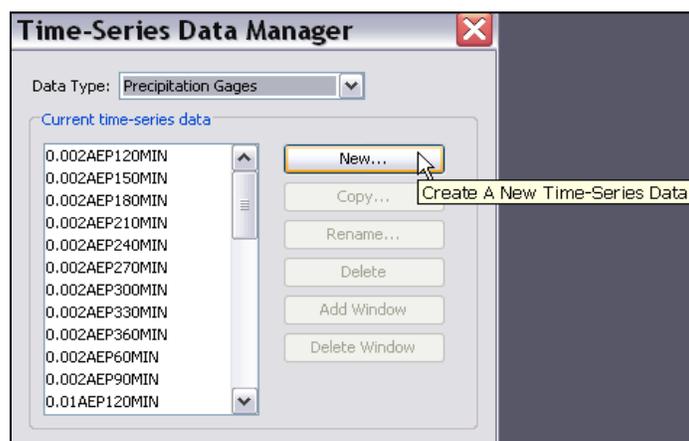


Figure 36. Precipitation Gage Manager

When a new time-series gage is added to a project, the HEC-HMS user has the option to retrieve the values from an existing DSS file or to enter manually the ordinates by typing the data in a table. From the **Data Source** list the analyst has selected **Data Storage System (HEC-DSS)** this will allow retrieve of values from an existing DSS file (Figure 37).

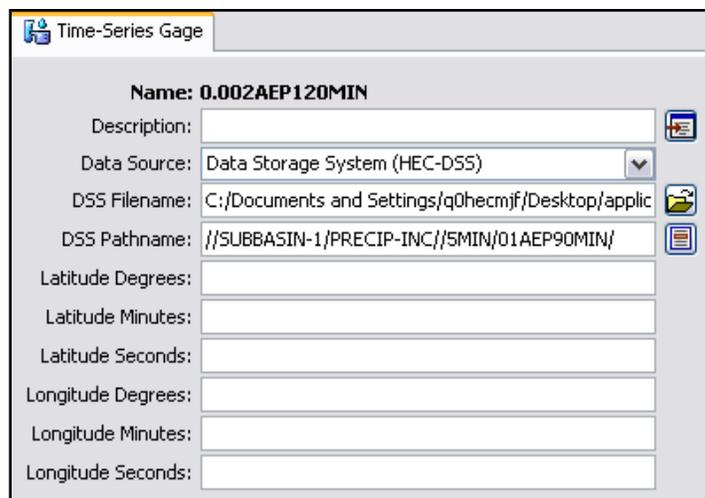
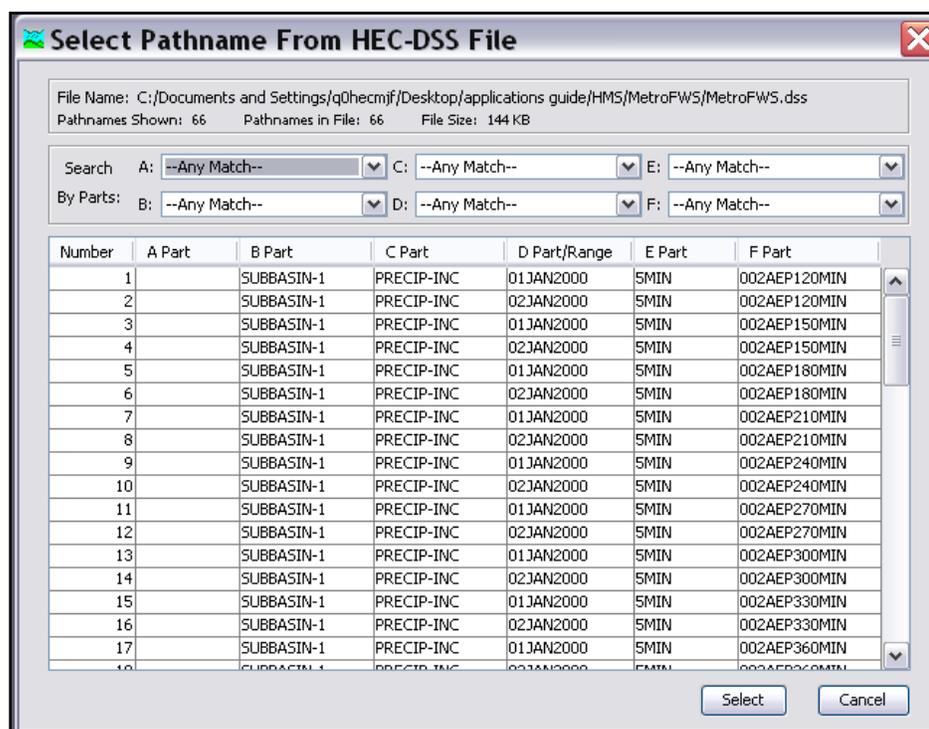


Figure 37. Component Editor for a Precipitation Gage

The next step for using time-series data in an existing DSS file is to select the DSS file and the correct data record within the file. Click the **Open File Chooser** , a browser (**Select HEC-DSS File**) will open, select the appropriate DSS file, click **Select**. The selected DSS file will now display in the **DSS Filename** text box. After the correct DSS file has been selected, click **Select DSS Pathname** , the **Select Pathname From HEC-DSS File Editor** (Figure 38) will open. From the Editor, the list of data records in the selected DSS file is displayed. With this, almost any record from any DSS file to which the analyst has access can be selected and assigned to the gage.

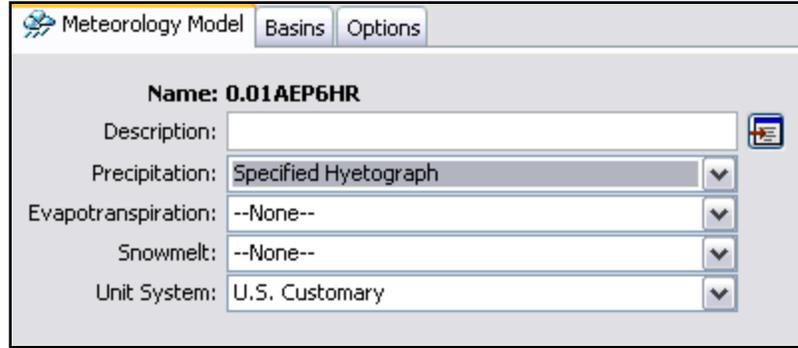


**Figure 38.** Select Pathname From HEC-DSS File Editor

This task of adding a gage and associating data from a DSS file, was repeated to create one gage for each combination of storm and duration. The result was a set of 88 gages for the eight frequency-based storms.

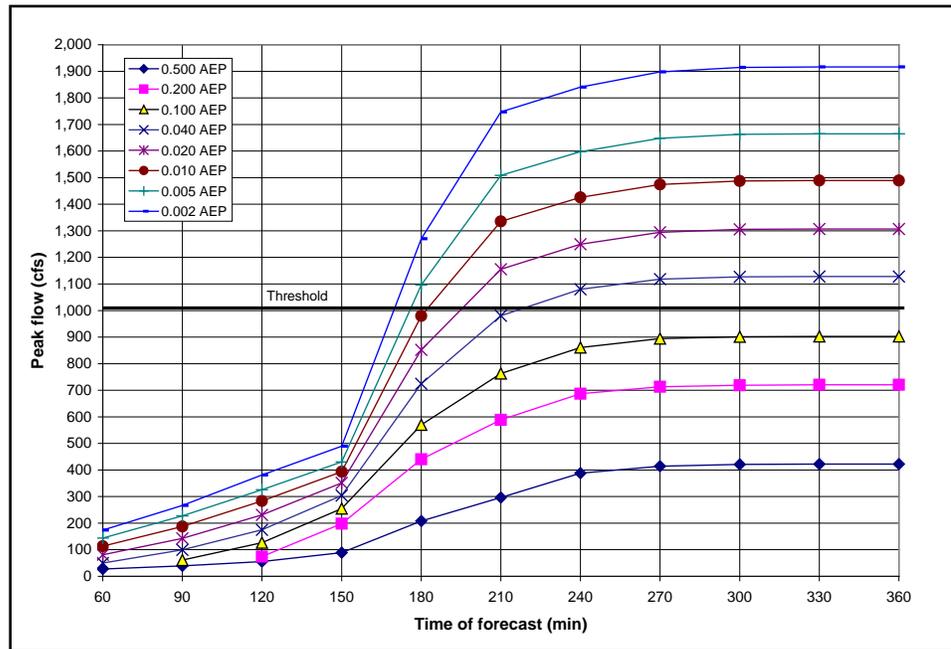
Next, the analyst created a meteorologic model for each of the cases, using the 88 precipitation gages to define the hyetographs. To do so, the analyst created a meteorologic model and selected **Specified Hyetograph**, as shown in Figure 39. In the specified hyetograph **Component Editor**, the analyst selected from among the 88 precipitation gages.

At last, the preparation for the computations was complete, and the runoff hydrographs could be computed and inspected. To insure that this computation was complete, the analyst used the **Simulation Run Manager** to "assemble" the simulation runs, using the basin model and control specifications in combination with each of the rainfall events.



**Figure 39.** Specified Hyetograph Precipitation Method is Selected

Figure 40 shows the results of the HEC-HMS simulation runs. For each frequency storm, the computed peak with each portion of the six-hour event is plotted. From this, the time at which threshold exceedance (flow greater than 1,100 cfs) would be detected can be identified. For example, with the 0.002 AEP event, exceedance would not be detected with only



**Figure 40.** TOF versus Peak Flow at the Critical Location

150 minutes of rainfall data, but it would be detected with 180 minutes of data; interpolation yields an estimate of 175 minutes as the detection time for that event. The estimated values for all events are shown in Column 3 of Table 28. Column 4 of that table shows the maximum mitigation time. The actual mitigation time would be less if time is required for notification and decision making. For example, if twenty minutes is required for notification and decision making, the mitigation time available for the 0.002 AEP event will be only forty minutes. Forty minutes provides little opportunity for property protection, but it is enough time to evacuate, if plans have been made and if the evacuees are well prepared.

## Results Processing

The objective of the study, of course, is not to run HEC-HMS to compute hydrographs, so the work required is not complete. Now the analyst must process the results, using Equation 6, to estimate an average warning time with which benefit can be estimated.

At the onset of this computation with Equation 6, the analyst realized that events smaller than the 0.04 AEP event did not exceed the threshold. Including those in the expected value calculation might bias the results, as the warning time would be infinite (no warning would be issued because the threshold was never exceeded). Accordingly, the analyst decided to use conditional probability in the computations, reasoning that the expected value should use only events that exceeded the threshold.

So, by integrating the maximum mitigation time-frequency curve, as directed by Equation 6, and using conditional probability to account for events more frequent than the 0.04 AEP event, the analyst computed expected warning time. For the proposed system, the maximum expected warning time was 55 minutes at the critical location. Based on the system design, the flood-warning specialist optimistically estimates the notification and decision-making time to be twenty minutes. Thus, the actual expected mitigation time is 35 minutes. The resulting damage reduction is slight, but the cost of a local FWS is also relatively small, so a federal interest may exist.

## Summary

The goal of this study was to estimate the economic benefit of a flood warning system in order to identify if a federal government interest exists. To do so, the expected mitigation time resulting from the system was estimated with HEC-HMS. The next step is to use the Day Curve to estimate the damage reduction and compare this benefit to the annual cost of the system. Then, the net benefit can be calculated as with other damage-reduction alternatives shown in Chapter 4. For this example, the expected warning time was estimated at one location. In actuality, this process may be repeated for various vulnerable areas throughout a watershed.

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# CHAPTER 6

## Reservoir Spillway Capacity Studies

Economic analysis, as illustrated in Chapter 4, is appropriate for sizing most features of damage reduction projects. However, when human life is at risk, USACE policy is to design project features - especially spillways of reservoirs - to minimize catastrophic consequences of capacity exceedance

This chapter describes how HEC-HMS can be used to provide information for such design and for review of designs of existing structures.

### Background

#### Objectives

Reservoir design demands special care because of the potential risk to human life. The economic efficiency objective described in Chapter 4 requires that the location and capacity of a reservoir be selected so that the net benefit is maximized. However, the capacity thus found may well be exceeded by rare meteorologic events with inflow volumes or inflow rates greater than the reservoir's design capacity. In fact, simple application of the binomial equation demonstrates that if the USACE constructed and operates 200 independent reservoirs, each designed to provide protection from the 0.005 AEP (200-year) and smaller events, the probability of capacity exceedance at one or more reservoirs in any given year is 0.63.

This capacity exceedance may present a significant risk to the public downstream of the reservoir. Unless the reservoir has been designed to release the excessive water in a controlled manner, the reservoir may fill and overtop. This may lead to catastrophic dam failure. Accordingly, USACE policy is to design a dam, and particularly the dam's spillway, to pass safely a flood event caused by an occurrence of a rare event - one much larger than the design capacity of the reservoir (ER 1110-8-2 - USACE, 1991). A spillway capacity study provides the information necessary for this design.

Spillway capacity studies are required for both proposed and existing spillways. For proposed spillways, the studies provide flow rates required for sizing and configuring the spillway. For existing spillways, the studies ensure that the existing configuration meets current safety requirements. These requirements may change as additional information about local meteorology becomes available, thus changing the properties of the likely

extreme events. Further, as the watershed changes due to development or natural shifts, the volume of runoff into the reservoir due to an extreme event may change, thus rendering a historically safe reservoir unsafe. In that case, the spillway will be modified or an auxiliary spillway may be constructed.

## Extreme Events

Performance of a water-control measure can be evaluated with 3 broad categories of hydrometeorologic events: (1) historical events; (2) frequency-based events; and (3) an estimated limiting value event. Evaluation with historical events is useful for providing information that is easily understood by and relevant to the public. For example, a useful index of performance of a USACE reservoir is a report of the damage reduction attributable to that reservoir during the flood of record. The utility of frequency-based events was demonstrated in earlier chapters of this document; they permit computation of EAD and regulation or operation to meet risk-based objectives. The final category of event, the estimated limiting value, is described by Chow (1988) as follows:

*The practical upper limit on the hydrologic design scale is not infinite...Some hydrologists recognize no upper limit, but such a view is physically unrealistic. The lower limit of the design scale is zero in most cases...Although the true upper limit is usually unknown, for practical purposes an estimated upper limit may be determined. This estimated limiting value (ELV) is defined as the largest magnitude possible for a hydrologic event at a given location, based upon the best available hydrologic information.*

Thus the utility of the ELV event is to demonstrate how a damage reduction measure would perform in the worst reasonable case - a case that is very unlikely, but still possible. This is the approach used for spillway studies.

The ELV used for USACE studies is the probable maximum precipitation (PMP) event, and the corresponding probable maximum flood (PMF). The PMP is the "...quantity of precipitation that is close to the physical upper limit for a given duration over a particular basin" (WMO, 1983). In the United States, the properties of a PMP commonly are defined by NWS.

## Procedural Guidance

The following USACE guidance is particularly relevant to reservoir design studies:

- EM 1110-2-1411 (USACE, 1965) provides background for the development of the standard project precipitation method used by USACE.
- ER 1110-8-2 (USACE, 1991) describes the regulations for selecting the appropriate inflow design flood for dam safety. Required assumptions, such as initial water surface elevation and operation of control structures for reservoir analysis, are also described.
- EM 1110-2-1603 (USACE, 1992) provides guidance for the hydraulic design of spillways for flood control or multipurpose dams.

## Study Procedures

To meet the objective of a reservoir spillway capacity study, the following steps are typically taken:

1. Develop a model of the contributing watershed and channels.
2. Define the extreme-event rainfall: the PMP.
3. Compute the inflow hydrograph to the reservoir: the PMF.
4. Develop a model of the performance of the reservoir and spillway.
5. Use the model to simulate reservoir performance with the hydrograph from Step 3, routing the PMF through the reservoir, over the spillway, and through downstream channels.
6. Compare the performance of the spillway to the established criteria to determine if the spillway adequately meets the criteria.

HEC-HMS is a convenient tool to use for this analysis. Its application within this procedure is illustrated with the case study below.

## Case Study: PMF Evaluation of Spillway Adequacy for Bonanza Reservoir

### Watershed and Reservoir Description

Bonanza Dam and Reservoir are located on Hoss Creek in the central Sierra Nevada mountain range of California. The reservoir was completed in 1958 with the construction of Bonanza Dam, a rockfill structure. The reservoir and dam are shown in [Figure 41](#).



**Figure 41.** Photograph of Bonanza Dam and Spillway (courtesy of California Department of Water Resources)

The reservoir was constructed primarily to store water for power generation, but it provides incidental flood control and water supply. Releases are made also for fish and wildlife needs downstream. The top of the dam is at elevation 4,192 feet. The reservoir is connected by an 11,000-foot tunnel to a power plant at elevation 2,300 feet. Usable reservoir capacity is 123,286 acre-feet between elevations 3,900 feet (invert of the power tunnel) and 4,184 feet, the spillway crest elevation. The contributing watershed area to the reservoir is 39.7 square miles.

### **Decisions and Information Required**

The Bonanza Dam spillway initially was designed to carry safely a large event - thought to be approximately the 0.001 AEP (1,000-year) flood event. However, the risk of failure is of concern, so the spillway capacity is to be the PMP and PMF. USACE analysts have been asked to answer the following questions:

Will the existing spillway pass the PMF? That is, will the dam be overtopped if the PMF flows enter the reservoir? If not, how can the dam and spillway be modified to pass safely the PMF?

To answer the questions, the PMF must be computed and routed. The spatial extent of the analysis was limited to the portion of the watershed that contributes flow to the reservoir, to the reservoir itself, and to the area immediately downstream. This contributing area had been defined in the

design studies; otherwise the analyst could have used topographic data to delineate the watershed. In this case, the model extended downstream of the reservoir only a short distance. However, if development in the downstream floodplain is such that dam failure poses a significant risk, the model should be extended further. Only by doing so will information be available for assessing the risk and for developing emergency plans.

## Model Selection and Parameter Estimation

For this analysis, runoff-volume and direct-runoff transform methods are needed. As shown in [Table 2](#), a variety of options are available with HEC-HMS. Here the analyst selected the following:

- **Runoff volume method.** The analyst chose the initial and constant-rate runoff volume method. This method was used to represent the watershed characteristics during dam design. During PMF analysis, a common assumption is that the antecedent moisture saturates the soil before the PMP occurs. When this happens, the rate of infiltration approaches a constant value. The advantage of the initial and constant-rate method is that this physical condition can be represented well with the model. Another advantage is the simplicity of the method, which has only two parameters. Like many watersheds upstream of remote dams, the Hoss Creek watershed has no stream gage and few rain gages. Of course, inflows to the dam could be inferred from records of release and storage. However, the lack of rainfall data makes calibration of a more complex runoff volume method impossible. It could be argued that the analyst should use the CN loss method. However, the analyst felt that locally-developed predictors of the constant loss rates as a function of land use and soil type were preferable to the CN predictors, which have been developed as national averages. With the CN loss method, the loss rate is continuously decreasing towards zero as opposed to being a constant rate. Also, the CN loss method is not sensitive to rainfall intensity.
- **Transform method.** The analyst selected Clark's unit hydrograph. Again, this is the method that was used previously to represent the watershed characteristics in design studies. This method requires two parameters: time of concentration,  $T_c$ , and storage coefficient,  $R$ . Studies by the California Department of Water Resources yielded predictors for these parameters. The analyst did use the rather limited rainfall data for three historical events and computed reservoir inflow hydrographs using the Clark unit hydrograph method. When compared with inflow hydrographs inferred from reservoir records, the analyst judged the fit adequate.

- **Baseflow method.** The analyst did not include baseflow in the model.
- The PMF represents runoff from the most severe combination of critical meteorologic and hydrologic conditions for the watershed. During such events, travel times tend to be significantly shorter. Consequently, it is common to adjust unit hydrograph parameters to "peak" the unit hydrograph (USACE, 1991), increasing the maximum runoff and shortening the runoff time. As a general rule of thumb, reservoir inflow unit hydrographs for PMF determinations have been peaked 25 to fifty percent. The analyst here did so, after reviewing observed runoff hydrographs from other severe storms in the region. Ultimately, the analyst shortened  $T_c$  and reduced  $R$  to achieve a unit hydrograph peak approximately fifty percent greater than that found with the original best-estimates of the parameters. The values selected for PMF analysis were  $T_c = 2.0$  hours and  $R = 4.6$  hours. The analyst selected a fifteen-minute simulation time interval, consistent with this estimated time of concentration.

### Boundary Condition: PMP Development

The NWS has developed PMP calculation procedures for all regions of the United States in Hydrometeorological Reports (HMR). For example, the eastern United States is covered by HMR No. 51 (NOAA, 1978) and No. 52 (NOAA, 1982). Because this is such a large area with many USACE projects, HEC developed software (HMR52 - HEC, 1984) to perform the storm analysis; the resulting hyetograph is stored in DSS for input to HEC-HMS.

As the availability of data increases, the PMP estimates from NWS HMR may require adjustment in order to better define the conceptual PMP for a specific site. Therefore, it is appropriate to refine PMP estimates with site specific or regional studies performed by a qualified hydrometeorologist with experience in determining PMP; the analyst here turned to the local office of the NWS for assistance with this. PMP data provided by the meteorologist are given in [Table 29](#).

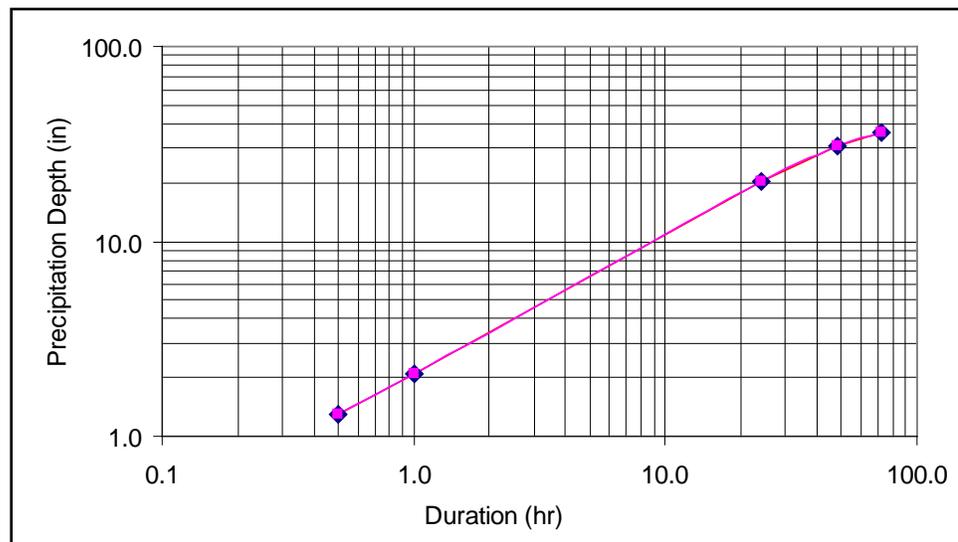
**Table 29.** Summary of PMP Depth Duration Data Provided by Project Meteorologist

Duration (hr)	Depth (in)
0.5	1.30
1	2.10
24	20.43
48	30.55
72	36.41

The PMP estimates were provided as a 72-hour storm, divided into six-hour increments. These six-hour values can be arranged into a storm temporal distribution that is front-, middle-, or end-loaded. (Here, the analyst checked each temporal distribution, making runs with the rainfall peak at the center of the distribution and at the 33 and 67 percentage points. This analyst found the timing of the rainfall peak had little effect on the PMF peak discharge for this particular watershed. The maximum computed reservoir water surface elevation was the same for all cases. This may not be the case in other watersheds.)

The four six-hour intervals with greatest depth were grouped into a 24-hour sequence, and the remaining intervals were arranged as described below to complete definition of the rainfall event. Within the peak 24-hour sequence, the four six-hour values are distributed in an alternating block sequence, with largest values in the center.

For this watershed, the computation time interval selected was fifteen minutes, so depths for durations shorter than six hours and for intervals less than six hours are needed. To develop these, the analyst plotted the logarithms of depths and durations, as shown in Figure 42, and interpolated for intermediate durations. Some smoothing of the plotted function was required. Interpolated depths are shown in Table 30.



**Figure 42.** PMP Depth-Duration Curve for Bonanza Dam

To specify the PMP depths, the analyst used the **Frequency Storm** precipitation method. The **Component Editor**, which is shown in Figure 43, does not permit entry of a 72-hour rainfall depth, so depth for a duration of 96 hours (four days) was estimated and entered. The peak volume stored in the reservoir is a function of the PMF peak discharge. A two-day event could have been selected rather than the four-day event. The two-day event would yield the same peak discharge, stage, and volume of water in the reservoir.

**Table 30.** Extended PMP Depth-Duration Data for HEC-HMS Input

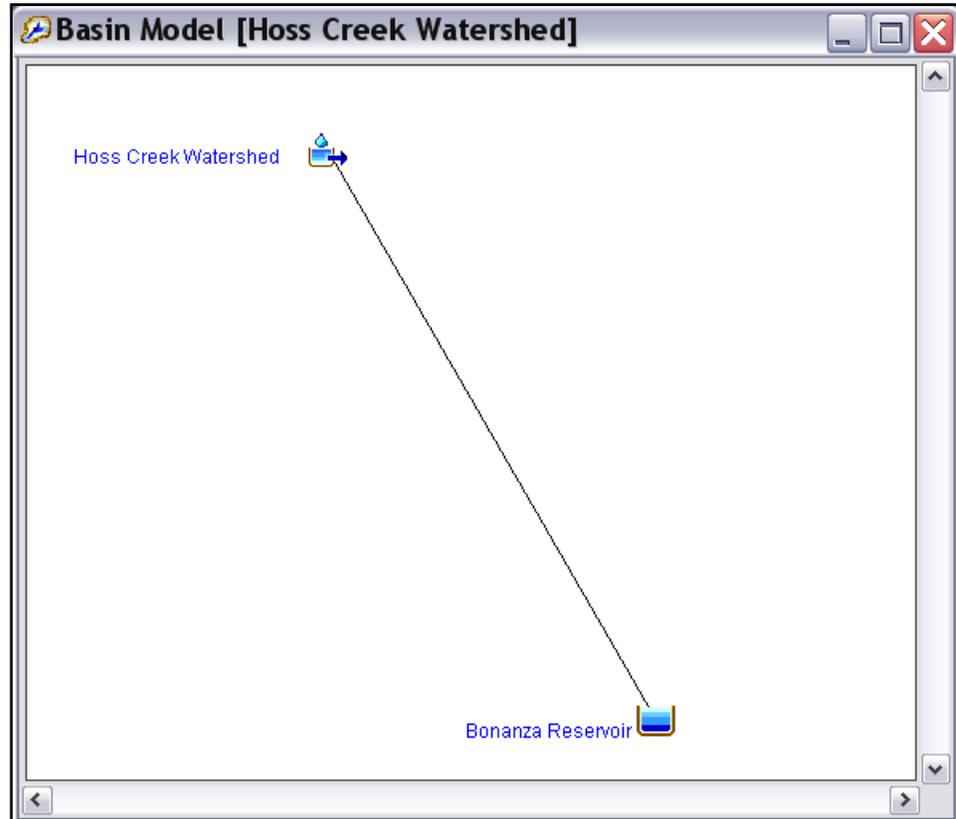
Duration (hr)	Depth (in)
0.25	0.81
0.5	1.30
1	2.10
2	3.48
3	4.65
6	7.67
12	12.63
24	20.80
48	30.55
72	36.41
96	42.00

**Figure 43.** PMP Rainfall Input

## Reservoir Model

In addition to the model of runoff, the analyst also developed a model of the reservoir and dam in HEC-HMS. The resulting basin model is shown in [Figure 44](#).

[Table 31](#) shows the elevation-storage curve for Bonanza Reservoir. The existing spillway crest is at elevation 4,184 feet and the crest length is 175



**Figure 44.** Basin Model for PMP Evaluation

**Table 31.** Elevation-Storage Data for Bonanza Reservoir

Elevation (ft)	Reservoir Storage (ac-ft)
3,900	0
4,100	31,400
4,184	123,300
4,185	124,680
4,186	126,300
4,187	127,950
4,188	129,650
4,189	131,420
4,190	133,150
4,191	135,000
4,192	136,700
4,195	141,800

feet. This information was found in the original design documents. However, if the data had not been available, the elevation-volume relationship would be developed from topographic and bathymetric surveys.

In this analysis, the analyst consulted dam-safety regulations followed by the state. Per these regulations, any low level outlets through the dam are assumed not operable, and all out flow from the reservoir must pass over the spillway. The analyst also considered the possibility of tail water

control. However, because all flow would pass over the elevated spillway, tail water was not a factor.

The reservoir was modeled using the **Outflow Structures** routing method. The elevation-storage curve shown in [Table 31](#) was used along with a spillway outlet. The spillway outlet was modeled using a **Broad-Crested Spillway** with a spillway crest elevation at 4,184 feet, a spillway length of 175 feet, and a discharge coefficient of 3.2.

## Initial Conditions

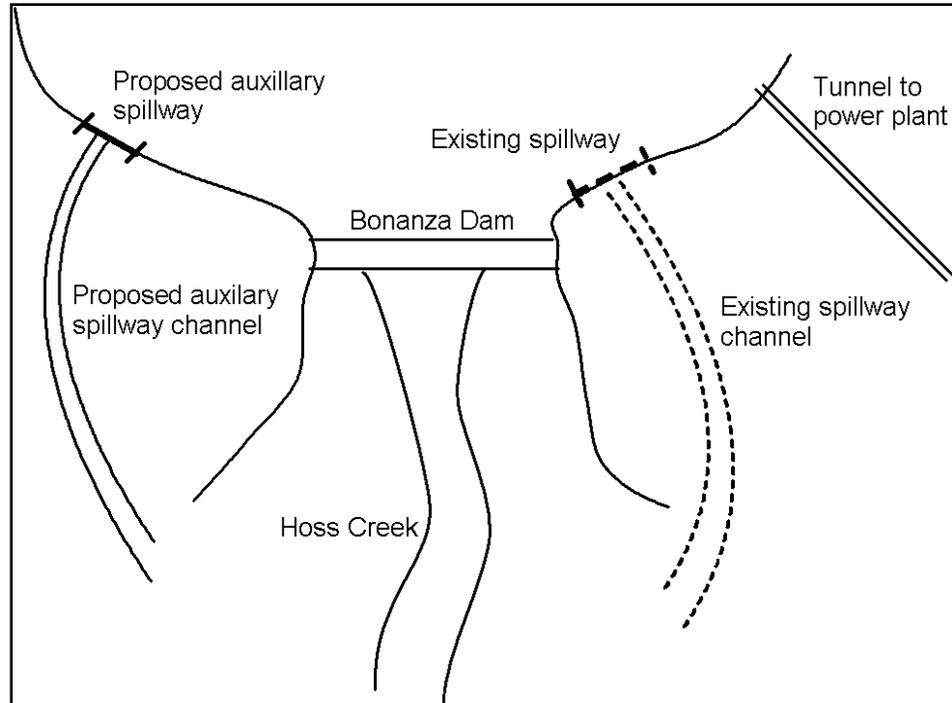
The analyst had to select two initial conditions for the analysis: (1) the initial state of the watershed, and (2) the initial state of the reservoir. For the first condition, the analyst reasoned that the watershed was likely to be saturated when an extreme event occurred, and thus set the initial loss equal to 0.00 inches. For the second condition, the analyst consulted state dam safety regulations and found that these specified that the initial reservoir water surface elevation should equal the spillway crest elevation. Thus spillway flow is initiated with inflow. This conservative initial condition was accepted and implemented by specifying **Initial Elevation** of 4,184 feet in the reservoir **Component Editor**.

## Application

The HEC-HMS model was completed, and the event simulated. A peak spillway discharge of 13,300 cfs was computed. The maximum water surface elevation in the reservoir was 4,192.3 feet. Top of the dam is at 4,192 feet, this means that the dam would be overtopped by the event. In addition, the analyst recognized that if precipitation depths were underestimated, if the unit hydrograph was not peaked adequately, or if the reservoir performance was modeled a bit optimistically, the pool elevation, in fact, would be greater. Further, the analyst knew that other factors, such as wind-driven waves, could well increase the pool elevation even more.

Research revealed that local dam safety regulations require a minimum difference of 1.5 feet to account for uncertainty in estimates. Thus the dam was considered unable to pass reliably the spillway design event.

Because the current configuration of the spillway did not pass safely the PMF, the analyst formulated an alternative design. This design increases the reservoir outflow capacity with an unlined auxiliary spillway in a low area on the ridge near the west abutment of the dam. [Figure 45](#) is a schematic of the proposed design. Water discharging over this auxiliary spillway will be carried to Hoss Creek at a point about 1,000 feet downstream from the dam in order to avoid endangering the dam.



**Figure 45.** Plan View of Proposed Design to Increase Spillway Capacity

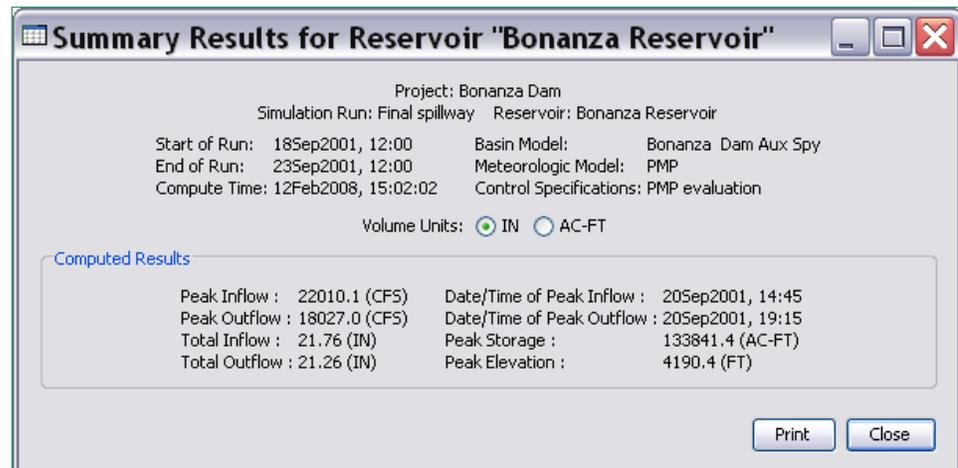
At the inlet, the auxiliary spillway would have a rectangular section. The maximum width possible is 300 feet; this is constrained by geological formations at the dam site. The optimal depth, presumably, will be the minimum depth, as the rock must be removed to create the spillway channel. The analyst found this depth by iteration as follows:

1. An additional spillway was added to the reservoir element. The additional spillway was modeled using a **Broad-Crested Spillway**, with a length of 300 feet and a discharge coefficient of 3.2.
2. A candidate auxiliary-spillway crest elevation was proposed.
3. A simulation run was computed with the new auxiliary spillway. As before, the analyst specified an **Initial Elevation** equal to 4,184 feet, indicating that the reservoir is initially full.
4. If the resulting maximum pool elevation was not at least 1.5 feet below the elevation of the top of dam, the crest elevation was lowered slightly, and Steps 2 and 3 were repeated.

[Table 32](#) shows the existing and auxiliary spillway characteristics for Bonanza Dam. This crest elevation for the auxiliary spillway satisfied the criterion for freeboard. The maximum reservoir water surface elevation was 4,190.4, as shown in [Figure 46](#); this is 1.6 feet below the top of the dam.

**Table 32.** Existing and Proposed Spillway Characteristics for Bonanza Dam

Spillway	Elevation (ft)	Length (ft)	Discharge Coefficient
Existing	4,184	175	3.2
Auxiliary	4,186	300	3.2

**Figure 46.** Summary Results for Bonanza Reservoir

## Summary

Because of the risk to human life, the adequacy of a spillway of a dam upstream of a population center is judged with the PMF, which is the result of the PMP. The PMP is an estimated limiting value event: *the largest magnitude possible for a hydrologic event at a given location, based upon the best available hydrologic information.*

HEC-HMS can be used to compute the PMF, using PMP depths as input for the hypothetical storm. Common loss and transform methods can be used, but adjustments to the parameters may be required to represent the worst-case condition. Likewise, dam and spillway performance can be simulated with the reservoir model included in HEC-HMS. For that, the analyst must derive and specify functions that describe how the reservoir will perform.

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# CHAPTER 7

## Stream Restoration Studies

USACE recognizes *...that the environmental effects of development during the last century are now ripe for remedial action. While the quality of our lives has improved in many ways, our ability to sustain that quality of life requires that we restore many of the natural structures and functions within the environment that have been damaged and disrupted* (Fischenich, 2001). This restoration may include the return of stream environments to conditions that approximate the most desirable aspects of conditions prior to development.

Remedial action and restoration requires careful planning, supported by careful technical analyses, if the impacts are to be sustainable. The analyses include hydrologic engineering studies in which HEC-HMS may play a critical role.

### Background

#### Goals of Stream Restoration

According to Fischenich (2001), the conversion of forests, farmland, wood lots, wetlands, and pasture to residential areas and commercial and industrial developments directly impacts stream and riparian corridors by:

- Altering stream channels through straightening, lining, or placement of culverts.
- Reducing riparian corridor width through floodplain encroachments.
- Increasing sediment yield during development and increasing pollutant loading following development.
- Displacing native riparian plant communities by invasive non-natives.

Indirect impacts of this urbanization include:

- Greater and more frequent peak storm flows, and longer duration of stream flows capable of altering channel beds and banks.

- Enlargement of the channel through incision and widening processes.
- Decreased recharge of shallow and medium-depth aquifers that sustain base and low flows.
- Increased stream temperatures and higher nutrient and contaminant loading.
- Alteration of the channel substrate.
- Reduction of stream system function.
- Reduction of riparian corridor function.
- Reduction of native wildlife species.

The goal of restoration is to mitigate these impacts by returning the ecosystem to a close approximation of its condition prior to disturbance (NRC, 1992). In the process, ecological damage to the resource is repaired, and ecosystem structure and functions are recreated. A committee of the National Research Council (NRC) has noted that meeting this goal is difficult in urban ecosystems because the basic hydrologic, geomorphic, physical, biological, and biochemical processes have been forever altered. Nevertheless, limited systematic actions can be taken to enhance the system, including:

- Developing buffers that provide protection to existing habitats.
- Enhancing surface water management with facilities specifically designed to reduce adverse hydrologic and geomorphic impacts, to improve water quality, and to protect fish and wildlife habitat.
- Undertaking stream corridor enhancement and restoration activities that will remediate existing problems or prevent future problems.
- Implementing regulations and taking management actions that are aimed at reducing future adverse impacts of development.

These actions follow the ideas proposed in The Nature Conservancy's (TNC) Freshwater Initiative (TNC, 2001).

## **Hydrologic Engineering Study Objectives and Outputs**

Measures of achievement of stream restoration goals are not defined in USACE guidance or authorizing legislation with the same degree of

specificity as the national economic development goal. Consequently, the role of a hydrologic engineering study cannot be defined as concisely. Instead, the information required from a hydrologic engineering study will depend upon the particulars of the actions and measures proposed.

Nevertheless, in a USACE stream restoration effort, the study team should agree upon conditions that are desired and indices for measuring the degree to which these are satisfied by a project. From this set, the hydrologic engineer, working cooperatively with other team members, can identify relevant information required. Typically, the information will be much the same as required for damage-reduction studies: peaks of specified AEP; and volume, duration, depth, and velocity of specified AEP. Water-surface profiles and inundated area maps for specified events may be required also.

For restoration projects in which performance with low-flow conditions is critical, other indices may be of interest. For example, flow, velocity, and depth-duration functions may be desired for assessment of impacts of runoff on habitat development. From these, for example, the likely depths and durations of inundation during prime growing season of grasses can be found.

A hydrologic engineering study for stream restoration planning must assess watershed and channel conditions both with and without proposed changes. This will provide the information necessary to measure the effectiveness of different restoration alternatives.

## **Authority and Procedural Guidance**

The following authorities have been used by USACE offices to restore aquatic habitats and mitigate development impacts:

- WRDA 1986, Section 206. This legislation directs USACE to carry out aquatic ecosystem restoration and protection projects.
- WRDA 1986, Section 1135. This authorizes USACE to modify existing project structures and operations to restore environmental quality. This subsequently was amended to include restoration project areas that are outside USACE project lands, but which were impacted by the project.
- WRDA 1974, Section 22. With this, Congress gave USACE general authority to provide assistance to States and tribal governments with planning for the development, utilization, and conservation of water and related land resources. Recent amendments have expanded this assistance to include ecosystem planning.

- WRDA 1992, Section 204. This authorizes USACE to protect, restore, and create aquatic habitat, including wetlands, in connection with dredging for authorized federal navigation projects.

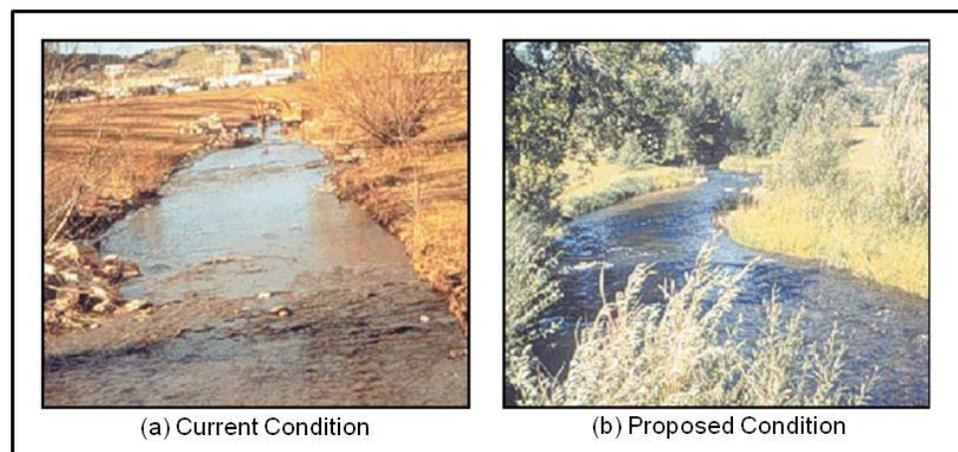
Procedural guidance for conduct of hydrologic engineering studies to support restoration activities is given in USACE EMs (Engineer Manuals) and ERs (Engineer Regulations) cited in earlier chapters of this document.

## Case Study: Channel Maintenance along Stirling Branch

### Watershed Description

Stirling Branch is a tributary of Deer Creek in a developing area in the western United States. The average elevation of the contributing watershed is 1,300 feet. The total watershed area is 1.23 square miles. The watershed has pockets of relatively high-density development. Watershed slopes are relatively steep. The creek channel slopes are on the order of three percent in the upper reaches and about one percent in the lower reaches. The creek crosses several roads in the watershed through culverts, which produce major obstructions to flow.

Development in the watershed has had a significant impact on the stream corridor. As illustrated in [Figure 47a](#), the channel has been straightened, and the riparian corridor has been altered, with native vegetation removed for the sake of hydraulic efficiency of the channel. The channel has been directed through culverts at the road crossings. A maintenance program was established by the local government to maintain these "clean" channels and to ensure that the culverts were clear.



**Figure 47.** Before and After Photos of "Restored" Stream (Fischenich, 1999)

The local government has now reconsidered the wisdom of this channel modification and the maintenance, and has appealed to USACE for

technical assistance in restoring the stream. A local environmental group has supported this, suggesting that the channel should be restored to a state similar to that shown in [Figure 47b](#), with ground-level shrubs and trees of moderate density planted in the stream corridor. USACE environmental specialists have suggested omitting the ground-level shrubs and planting only trees that would be trimmed to keep branches above the 0.01 AEP water surface elevation. However, owners of property adjacent to the stream are not so keen on the restoration. They are concerned that the vegetation in the floodplain will induce flooding, as it obstructs flow. To provide the necessary information to make decisions about the restoration, a hydrologic engineering study with HEC-HMS was undertaken.

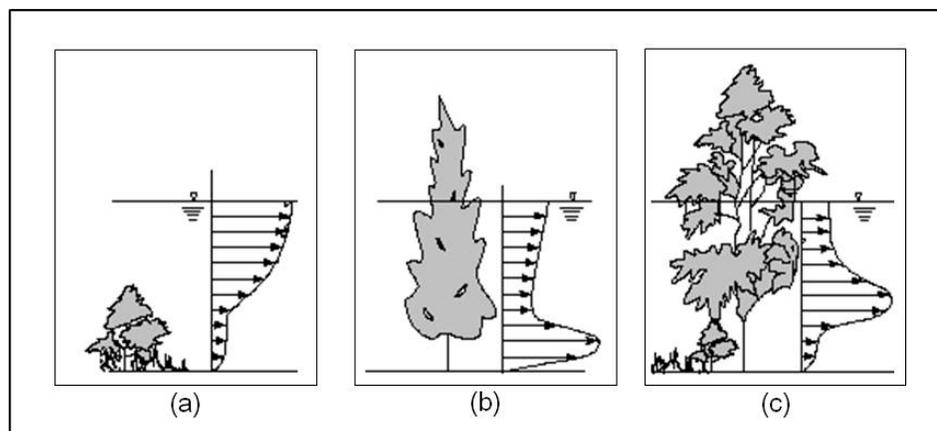
## Decisions Required and Information Necessary for Decision Making

The decision required is this: which, if any, of the proposed stream restoration projects should be selected for implementation? Much information beyond what can be provided by a hydrologic engineering study is necessary to make this decision. However, in this case, the analyst was called upon to provide the flows in and downstream of the restored reach for the 0.50 AEP, 0.10 AEP, and 0.01 AEP events. The 0.50 AEP (2-year) peak flow is a critical parameter for restoration, as this is the bank full or channel-forming flow in the natural stream system; it will be used for sizing many features of the project. The 0.01 AEP event is used for floodplain-use regulation, and thus is a good indicator of any adverse impact of the restoration: If this flow increases significantly, it is an indication that damage may be incurred downstream. The 0.10 AEP event is an intermediate event, and is often used for design of storm water management facilities. Thus, it provides a benchmark for comparison.

## Model Selection

[Figure 48](#) shows velocity profiles for several vegetation types; in this illustration, all are submerged to some extent. In [Figure 48a](#), low vegetation is fully submerged by a higher flow rate. In this case, the velocity is retarded, and the velocity gradient is near zero within the canopy. Above the canopy, the velocity increases approximately logarithmically. In [Figure 48b](#), a tree is partially submerged. Lower branches of this tree are trimmed to have less impact on lower flow rates or on the velocity profile at the lower boundary. In [Figure 48c](#), trees are combined with lower vegetation. In this case, velocity is retarded by both the undergrowth and the tree branches.

These changes in velocity will alter depths in the channel and discharge rates downstream. While a detailed analysis of this requires a detailed open channel model, the simplified routing methods included in HEC-HMS can provide insight to changes in the discharge rates. Thus, in this



**Figure 48.** Illustration of Velocity Profiles with Submerged and Un-submerged Vegetation (Fischenich, 2000)

case, the critical hydrologic engineering component is the channel routing method.

With the relatively steep stream and fast rising hydrographs from the small contributing watershed, the Muskingum-Cunge routing method is an appropriate choice for this analysis. This method can conveniently reflect changes to the channel cross section and changes to the channel roughness due to the vegetation.

Of course, the analyst also needed to develop a basin model to compute the inflow hydrographs - the boundary conditions for the channel routing models. For this, the watershed was subdivided into six subbasins (primarily for convenience of modeling the channels). For each, the SCS CN method was selected for runoff volume computation and the SCS UH method was used for transforming rainfall excess to runoff.

## Model Fitting and Verification

**Subbasin models.** The subbasins were ungaged, so no direct calibration was possible to estimate parameters. Instead, the analyst found the average CN for each subbasin using GIS (geographic information system) tools with coverages of land use and soil type.

The SCS UH lag was estimated for each subbasin as sixty percent of the time of concentration for each subbasin, following SCS recommendations. The time of concentration for each subbasin was estimated with procedures suggested by the SCS (SCS, 1971, 1986). Based upon experience with gaged watersheds in the region, this estimate has been shown to yield reasonable results for those watersheds.

**Routing models.** The restoration alternatives which were proposed correspond to scenarios illustrated in [Figure 48b](#) and "c". To represent

these, the analyst selected the Muskingum-Cunge routing method and entered geometric data appropriate for each case, as shown in Figure 49.

Field	Value
Basin Name	With maintenance
Element Name	Routing through SB5
Length (FT)	700
Slope (FT/FT)	0.01
Manning's n	0.035
Invert (FT)	
Shape	Eight Point
Left Manning's n	0.1
Right Manning's n	0.1
Cross Section	8-Point Cross Section

**Figure 49.** Component Editor for Reach Element Using the Muskingum Cunge Method

The existing, without-project-condition cross sections were surveyed at selected locations. Roughness values for the main channel and both overbanks were estimated for the without-project condition after a field investigation. Photographs of the channel were taken and compared with those in Barnes (1967) to select the Manning's  $n$  values.

Channel cross sections were proposed for the restoration alternatives by the study team's plan formulators. Modified Manning's  $n$  values for the alternatives were estimated using Fischenich's equation (1996) for flow resistance in channels with non-submersed vegetation:

$$n = k_n R^{2/3} \left[ \frac{C_d \text{Veg}_d}{2g} \right]^{1/2} \quad (7)$$

where:

- $C_d$  = coefficient to account for drag characteristics of vegetation;
- $\text{Veg}_d$  = vegetation density; and
- $g$  = gravitational constant

Vegetation density is defined as:

$$\text{Veg}_d = \frac{\sum A_i}{AL} \quad (8)$$

where:

- $A_i$  = area of vegetation below water surface, projected onto a plane perpendicular to direction of flow
- $A$  = cross-sectional area
- $L$  = characteristic length

Flippin-Dudley (1998) suggests that  $C_d$  can be estimated as:

$$C_d = 2.1(VR)^{-1.1} \text{ with debris present and leaves absent from trees and shrubs;} \quad (9)$$

$$C_d = 0.28(VR)^{-1.1} \text{ with debris removed and leaves present.} \quad (10)$$

The analyst used these relationships, finding and using the worst case (greatest  $n$  value) for each stream reach.

## Boundary Conditions and Initial Conditions

Hypothetical frequency-based storms were defined, as illustrated in earlier chapters. The SCS CN method defines, by default, an initial loss that is a function of the CN. This was used here. While the analyst's intuition was that the average initial loss for the 0.50 AEP event would likely be much greater than that for the 0.01 AEP event, the analyst realized that this parameter likely would have little impact on the decision making. The goal of the analysis was to compare the impact of the restoration alternatives, not to complete a detailed flood-damage analysis.

## Application

An HEC-HMS project was developed for the analysis as follows:

1. A single control specification was developed, with a time interval appropriate for the subbasin with the shortest time of concentration. The minimum time of concentration was 28 minutes, so a time interval of four minutes was selected.
2. A meteorologic model was prepared for each of the three hypothetical rainfall events. A 24-hour storm was used. For this small watershed, a shorter duration storm would likely be adequate for definition of the peaks. However, many design studies in the region have been completed with 24-hour storms, so for consistency, that was used here.
3. A basin model was created for each of the three cases of interest: no restoration; restoration with trimmed trees in the floodplain; and restoration with trees and shrubs. The subbasin runoff elements were identical in these three basin models, but the routing models varied in each to represent the differences in channel cross section and vegetation in the floodplain. The basin schematic is shown in [Figure 50](#).
4. The analyst defined nine simulation runs, combining the three meteorologic models with the three basin models. Peak flows at the watershed outlet are shown in [Table 33](#).

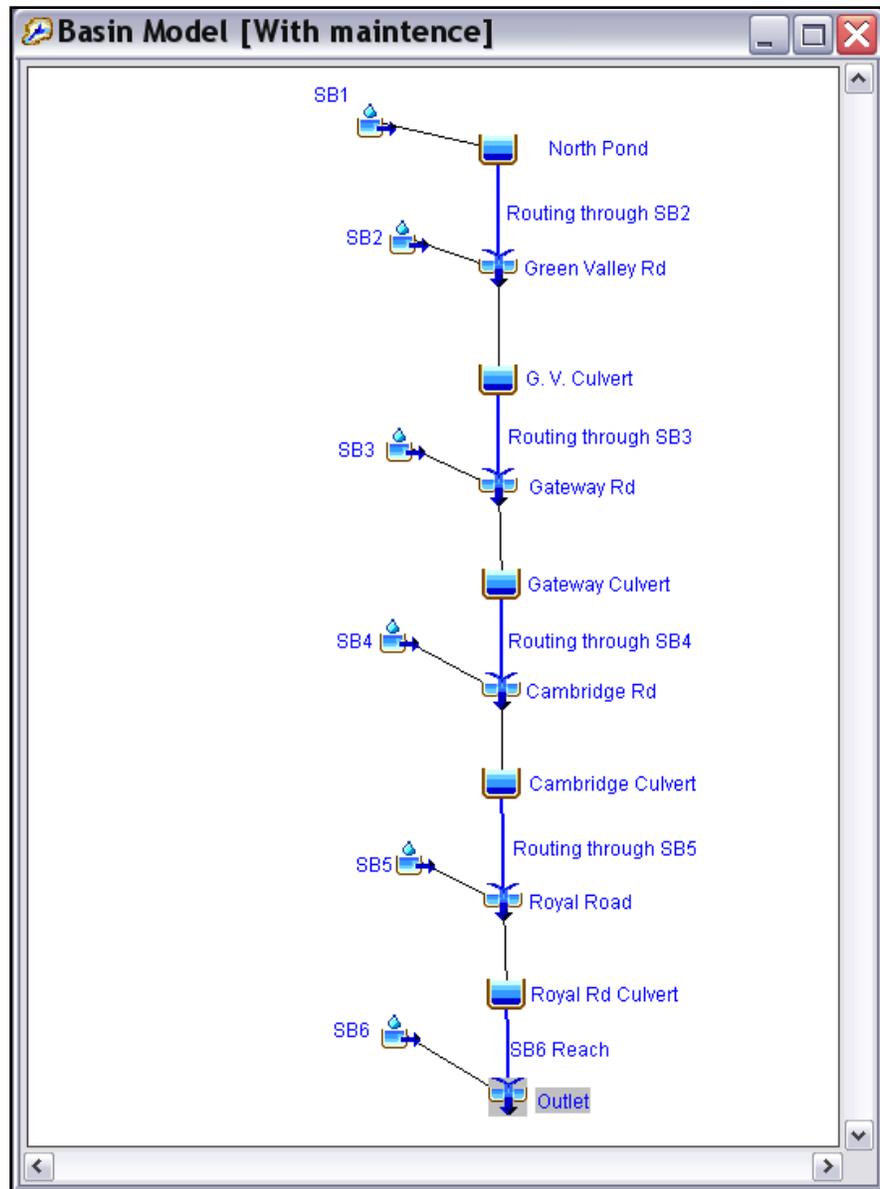


Figure 50. Basin Model Schematic of Stirling Branch

Table 33. Peak Flows of Restoration Alternatives (cfs)

AEP of Hypothetical Event	Without Project Condition	Restoration with Trees Only	Restoration with Trees and Shrubs
0.50	71	65	63
0.10	235	218	208
0.01	534	500	480

Clearly the restoration has an impact on the downstream peaks. The greater the vegetation in the channel, the more the peak flow was attenuated, thus resulting in a lower peak flow. However, the peak stage was likely increased and needs to be investigated with a hydraulic model. HEC-RAS could be used for this analysis.

## Additional Analysis

The analysis described here provides only part of the hydrologic engineering information required for evaluation of the restoration alternatives. It does not provide information about depths of flooding or velocities. That information was developed for this study using HEC-RAS. The HEC-HMS and HEC-RAS models developed by the analyst shared cross-section geometric data and estimates of Manning's  $n$ . Thus, the models provided quick-and-dirty checks of reasonableness: The HEC-RAS model provided velocity estimates, with which travel time could be computed. The computed translation of hydrographs in the channels was compared with this, and some fine-tuning was done.

If a more detailed analysis is warranted, the channel routing in HEC-HMS may be substituted with an unsteady flow hydraulics model such as HEC-RAS. HEC-HMS could be used to compute the local inflows to the channels, and then using DSS, the flows can be input to HEC-RAS. To do this, additional channel geometry and refinements in channel parameters may be needed. During this more detailed analysis, natural channel tendencies, from a geomorphic standpoint, will need to be examined. For example, water velocities may erode the vegetation and tend to straighten the channel.

## Summary

The goal of this study was to determine which stream restoration method should be implemented. HEC-HMS, in conjunction with HEC-RAS, was used to model the watershed and channels to compare restoration alternatives. When altering channel hydraulics for restoration projects, consideration must be given as well to the flood-control purposes of the channel. By increasing the vegetation in the channel, the water surface elevation will likely increase for a given flow.

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# CHAPTER 8

## Surface Erosion and Sediment Routing Studies

This chapter describes how to simulate watershed erosion and route sediment with HEC-HMS. HEC-HMS erosion and sediment transport model results can be a valuable resource in watershed management studies.

### Background

#### Goals of Erosion and Sediment Control

Surface erosion, reservoir sedimentation, and in-stream sediment transport have become increasingly important in watershed management and natural resources conservation planning studies. Sediment processes also affect evaluation and implementation of water quality Best Management Practices (BMPs) and the evaluation of Total Maximum Daily Loads (TMDL's) (Pak, 2010).

The U.S. Environmental Protection Agency (EPA, 2003) has identified several soil erosion and sedimentation consequences:

- Soil loss reduces nutrients and deteriorates soil structure, decreasing the land's productive capacity.
- Suspended solids reduce sunlight available to aquatic plants, cover fish spawning areas and food supplies, smother coral reefs, impede filter feeders, and clog fish gills. Turbidity interferes with the feeding habits of certain fish species.
- Decreased fish population and the unappealing, turbid water limit recreation opportunities. Turbidity also reduces visibility, making swimming more dangerous.
- Deposited sediment reduces the transport capacity of streams, rivers, and navigation channels. Decreased capacity can increase flood frequency. Sediment can also reduce the storage capabilities of reservoirs and lakes increasing flood risk and dredging costs.

- Chemicals including pesticides, phosphorus, and ammonium can be adsorbed and transported with sediment and can eventually be released from the sediment into the water.

The EPA (2003) established management practices that seek to control the delivery of Non-Point Source (NPS) pollutants to receiving water resources by:

- Minimizing available pollutants (source reduction);
- Retarding pollutants transport and/or delivery by either reducing flow, and thus the amount of pollutant transported, or depositing the pollutant; or
- Remediating or intercepting the pollutant through chemical or biological transformation.

Surface erosion and sediment control studies help identify the source of NPS pollutants and provide information beneficial in establishing management practices that serve to reduce or mitigate surface erosion and sedimentation.

## **Surface Erosion and Sediment Routing Study Objectives and Outputs**

While specific erosion and sediment control objectives are site and study specific, surface runoff and erosion models can be used as a tool to model sediment loads from pervious and impervious areas in a watershed and then route the sediment downstream while modeling erosion and deposition within river reaches and reservoirs (Pak, 2010; Gibson, 2010). Models are a useful tool for estimating flow and sedimentation in areas where observed data are unavailable, using data from watersheds with similar characteristics, or in conjunction with sensitivity and uncertainty analyses. Models can be especially useful for interpolating or extrapolating flow and sediment records in watersheds where observed data are available but not at the particular location of interest, or for predicting system response to projected future conditions.

Potential soil erosion and sedimentation study objectives include:

- Estimate sediment time series (sedigraphs) for each grain class at selected locations/elements.
- Estimate the volume of deposition or erosion within a reach element.

- Predict accumulated sediment at the reservoir bottom and accumulated sediment discharge from a reservoir for a given analysis period.
- Estimate changes in volume of eroded soil as a result of alternative land use management practices.
- Compute the quantity and gradation of sediment produced from a watershed to generate a sediment load boundary condition for a more detailed river hydraulics and sedimentation model, such as HEC-RAS (Gibson, 2010).

Carefully constructed models designed to address these objectives can inform soil erosion and sediment management decisions. A calibrated sediment model with accurate field data can predict regional and long term sediment trends.

## **Authority and Procedural Guidance**

USACE soil erosion and sediment transport studies are authorized by:

- The Clean Water Act of 1972. This authority establishes a process to identify impaired waters and set TMDL's in an effort to achieve pollution reduction goals.
- WRDA 1986, Section 1135. This section provides the authority to modify existing USACE projects to restore the environment and construct new projects to restore areas degraded by USACE projects.
- WRDA 1986, Section 204. This section authorizes projects to protect, restore, or create aquatic and ecologically functional habitat including wetlands.

USACE guidance on soil erosion and sediment transport studies includes the following:

- ER 1110-2-8153 (USACE, 1995) prescribes the procedure for conducting sedimentation investigations in support of hydrologic analyses, hydraulic design of civil works projects, and environmental impact analyses.
- EM 1110-2-4000 (USACE, 1989) identifies typical sediment problems encountered in the development of projects in inland waters and presents appropriate procedures to resolve these problems.

- TP-130 (HEC, 1990a) presents a case study of ephemeral channels in Central California and summarizes the procedures used for computing basin wide annual yield and single event sediment production.
- TP-152 (HEC, 1996) provides a procedure for using land surface erosion computations to develop the inflow sediment load for a river sedimentation model.
- PR-13 (HEC, 1990b) provides a procedure for computing basin wide annual yields and single event sediment production for an ephemeral channel.
- IHD-12 (HEC, 1977) addresses the topics of river morphology, data collection and analysis, reservoir sedimentation, and aggradation and degradation in free flowing streams.

## Study Procedures

Surface soil erosion and sediment routing studies typically require the following information: a calibrated precipitation-runoff model, soil data (such as the Soil Survey Geographic (SSURGO) database), land use data, field soil sample data, observed channel and reservoir sediment data, and channel cross section data. This information is utilized in the development, calibration, and validation of the surface soil erosion and sediment routing model.

Surface soil erosion and sediment routing modeling typically includes the following steps:

1. Perform a site investigation and review available data.
2. If data available are insufficient, prepare a field sampling program.
3. Examine published long-term discharge records and sediment gage records.
4. Prepare a calibrated precipitation-runoff model.
5. Select appropriate sediment modeling methods representative of the watershed.
6. Utilize regional studies and equations to estimate parameter values.
7. Calibrate the model to historical sediment data if available.
8. Analyze results to determine desired output values (e.g. mean annual sediment yield).

9. Validate the model if additional historical sediment data are available.
10. (Optional) For alternative analysis: Modify the watershed model to reflect changes in the watershed. Re-exercise the model with the same precipitation events. Compare the results to quantify the impact of the watershed changes.

The development, calibration, and validation of a watershed model used to analyze surface soil erosion and sediment transport is described herein.

## **Case Study: Estimating Sediment Yield in the Upper North Bosque River Watershed (UNBRW)**

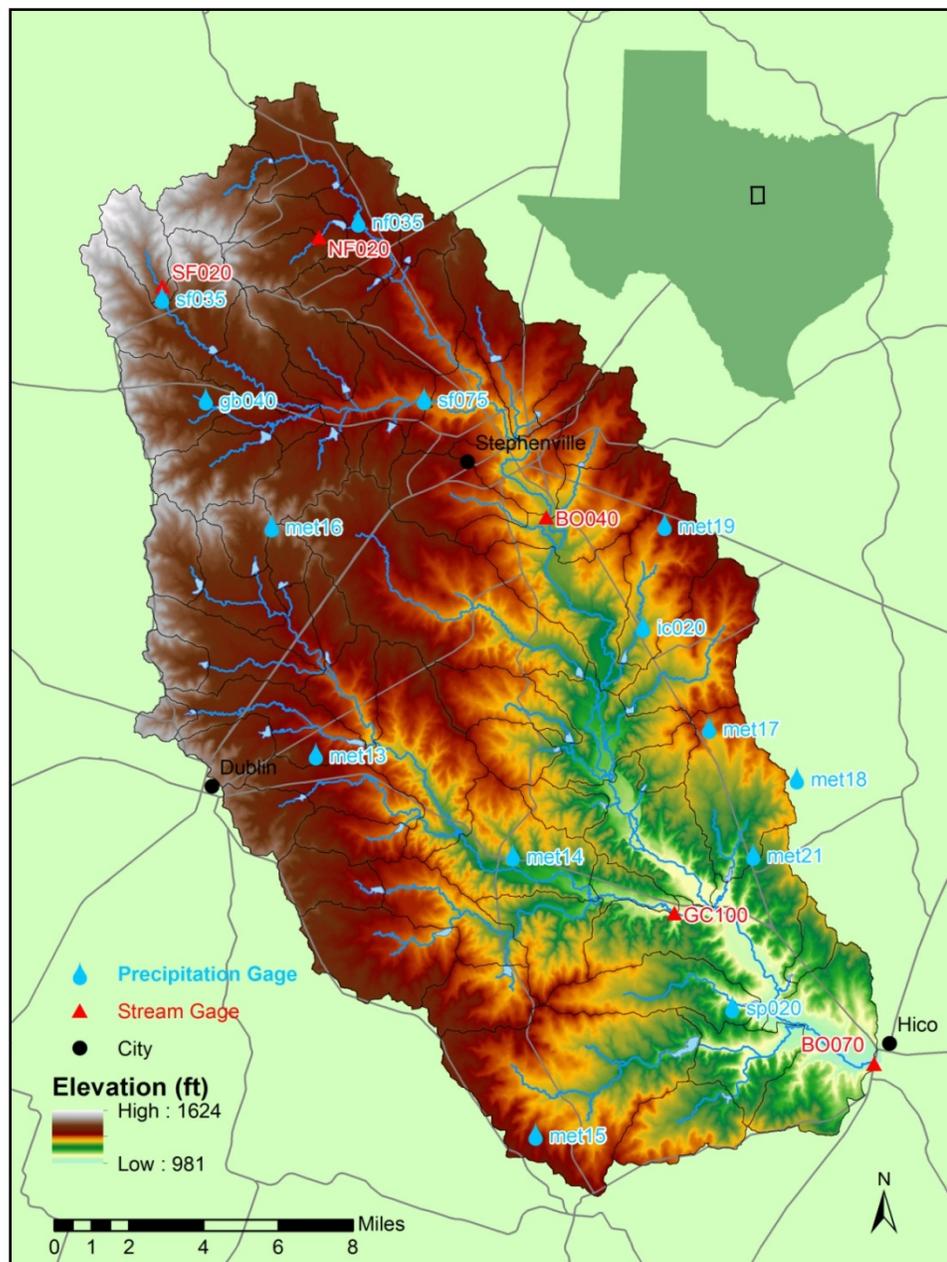
The purpose of this case study is to illustrate surface soil erosion and sediment routing computations through reservoirs and the stream network using HEC-HMS. This case study does not reflect an actual project.

### **Watershed Description**

The surface soil erosion and sediment routing capabilities for HEC-HMS were applied to UNBRW, approximately 359.8 square miles (931.9 km<sup>2</sup>), located in Central Texas as shown in [Figure 51](#). The UNBRW is defined as the contributing area above Hico, Texas (Saleh, 2004).

The UNBRW is 98 percent rural. Primary land uses include rangeland (43 percent), forage fields (23 percent), and dairy waste application fields (seven percent) (McFarland, 1999). Dairy production is the dominant agricultural activity in this watershed. Other agricultural activities include peanut, range-fed cattle, pecan, peach, and forage hay production. Soils in the UNBRW are classified as fine sandy loams with sandy clay subsoil, calcareous clays, and clay loams (Ward, 1992). Elevation in the watershed above Hico (stream gage *BO070*) ranges from 981 feet (299 meters) to 1624 feet (495 meters).

Average annual precipitation is 29.53 inches (750 mm) and the average daily temperature ranges from 42.8 degrees Fahrenheit (6 degrees Celsius) in winter to 82.4 degrees Fahrenheit (28 degrees Celsius) in summer (McFarland, 1999). Continental polar fronts produce low-intensity, long-duration storms in winter and fall and high intensity, short-duration squall-line thunderstorms occur in spring and summer (Saleh, 2004).



**Figure 51.** Project Location Map of UNBRW near Hico, TX

For this study, the watershed model was divided into 68 sub-watersheds upstream of Hico as shown [Figure 51](#). There are fourteen precipitation and five stream gage locations within the watershed which were used for model calibration and validation.

### Decisions Required

In an effort "to restore and maintain the chemical, physical, and biological integrity of the Nation's waters", the Clean Water Act establishes a process to identify impaired waters and set TMDL's to achieve pollution reduction goals (33 U.S.C §1251(a)). The EPA defines TMDL as "a calculation of

the maximum amount of a pollutant that a waterbody can receive and still meet water quality standards". TMDLs are used to identify how a waterbody is impaired, what caused the degradation, and pollutant load reduction necessary to meet water quality standards (EPA, 2003).

Because of intensive agricultural production in the UNBRW, primarily from the dairy industry, state government officials have expressed concerns with the degradation of local water quality (McFarland, 1998). In this case study, suspended sediment load was the target pollutant investigated to inform TMDL decisions in the UNBRW. To establish a TMDL the following decisions must be addressed as they relate to water quality objectives:

- What are the major nonpoint source contributors in the watershed?
- What is the total suspended solids (TSS) load within the watershed over a given analysis period?
- What are the expected siltation rates for reservoirs in the watershed?

### **Information Required**

To address questions listed above, the following information is required:

- The average annual sediment load.
- The average annual siltation amount for each reservoir in the watershed.
- Sediment time series (sedigraph) for each grain class at each analysis point over a given analysis period.

To provide this information, a watershed hydrology and sediment model will be used to compute the average annual sediment yield for a sediment budget analysis. HEC-HMS will be used to model surface erosion and sediment transport. To develop the HEC-HMS sediment model, the following data will be used:

- A calibrated precipitation-runoff model including observed stream flow data.
- Observed TSS data.
- Average monthly evapotranspiration values from Appendix H in EM 1110-2-5027 (USACE, 1987).

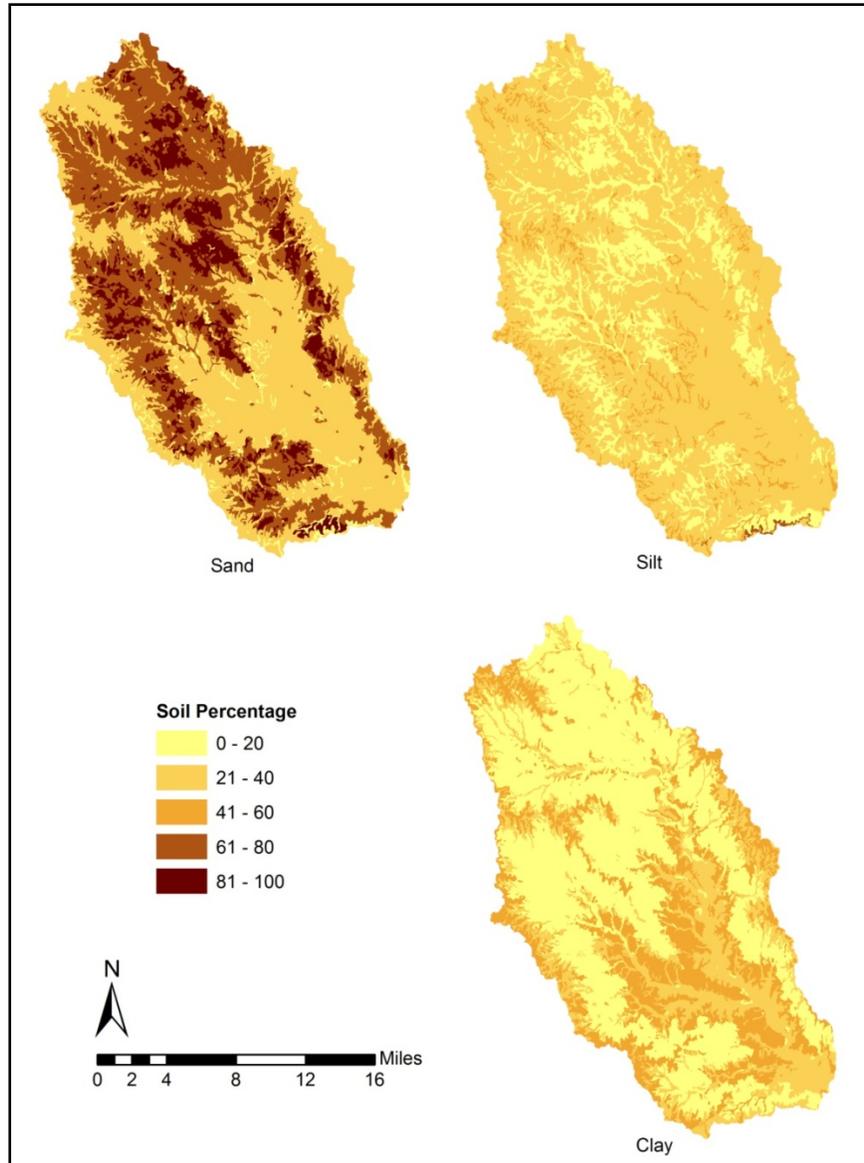
- Precipitation gage data from the Texas Institute for Applied Environmental Research (TIAER) and the NWS.
- Stream cross section data from field surveys.
- Stream bed sediment data from field surveys.
- DEM terrain data (Figure 51).
- SSURGO data from the Natural Resources Conservation Service (NRCS) website:  
<http://www.soils.usda.gov/survey/geography/ssurgo/> (The SSURGO database is used to estimate the soil gradation and erodibility factor in each subbasin using GIS. Figures 52 and 53 show the percentage of surface soil components and the basin average erodibility factors in UNBRW, respectively).
- Land use spatial data for the watershed, as shown in Figure 54 (Land use data are used to estimate percent impervious area and cover and management factors for use in the Modified Universal Soil Loss Equation (MUSLE) equation).

Data is collected, then analyzed and prepared for implementation in HEC-HMS using HEC-DSSVue, HEC-GeoHMS, GIS, and spreadsheet applications.

## Model Selection and Fitting

For this analysis HEC-HMS was used to perform surface soil erosion and sediment routing computations through the subbasins, reservoirs, and stream network of the UNBRW. Figure 55 illustrates the completed basin model schematic of the UNBRW. Watershed data collected were used to develop subbasin, reach, and reservoir elements of the watershed sediment model. The following discussion describes detailed surface erosion and sediment routing processes for the three main elements, subbasin, reach, and reservoir, used in the development of the UNBRW sediment model (Pak, 2008; Pak, 2010). Note: the HEC-HMS User's manual (HEC, 2013) lists all methods available to perform surface soil erosion and sediment routing for subbasin, reach, and reservoir elements.

1. **Subbasin Element:** The subbasin element represents drainage basins, simulating precipitation, infiltration, surface runoff and erosion processes. HEC-GeoHMS delineated the stream and subbasin boundary network based on USGS DEM data, but also forced sub-basins at reservoir and stream gage locations as calibration points. The MUSLE surface erosion method was chosen to simulate sediment yield processes from a pervious land



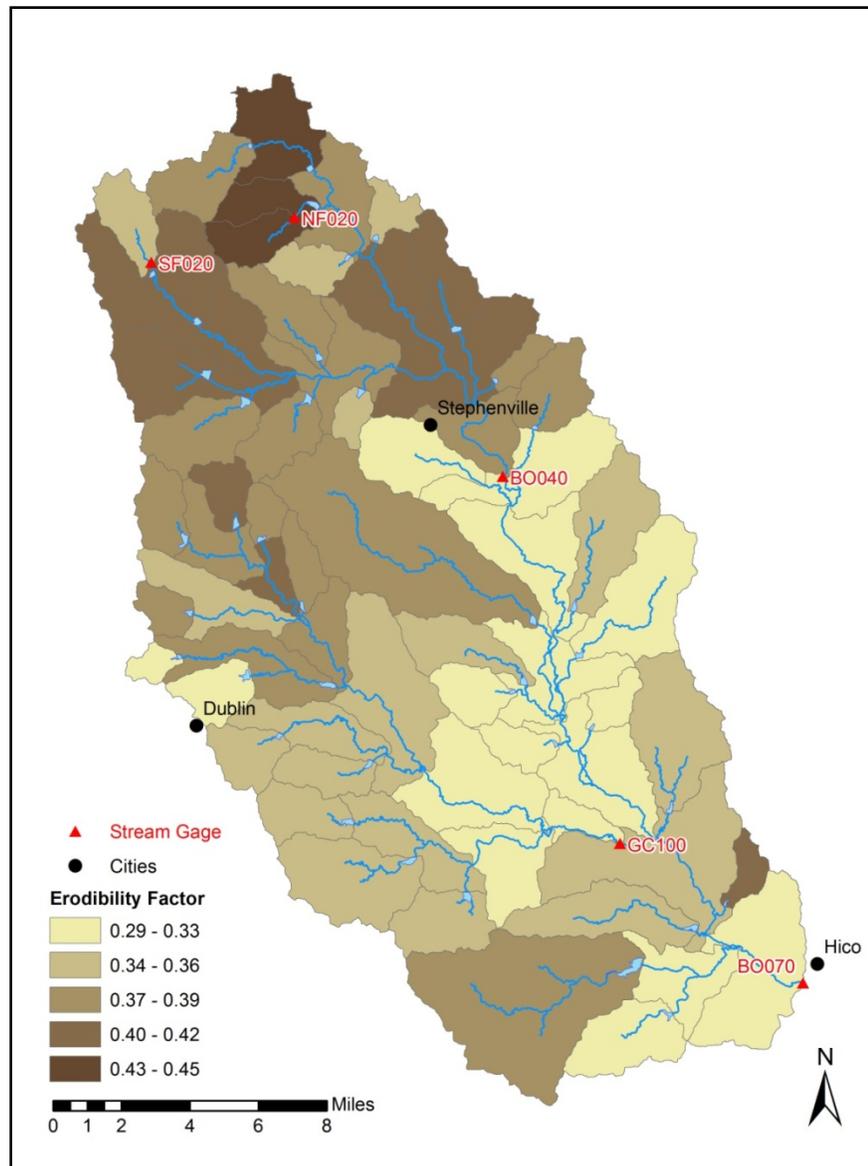
**Figure 52.** Percentage of Surface Soil Components in UNBRW (from the SSURGO database)

segment. This method was selected because it was developed for agricultural applications corresponding to the rural/agricultural nature of the UNBRW. Sediment yield from a pervious land segment using the MUSLE Equation is calculated as shown in Equation 11 (Williams, 1975).

$$Sed = 95 \cdot (Q_{surf} \times q_{peak})^{0.56} \times K \times LS \times C \times P \quad (11)$$

where:

- Sed is the sediment yield for a given event (tons)
- $Q_{surf}$  is the surface runoff volume (cubic feet)
- $Q_{peak}$  is the peak runoff rate (cubic feet per second)

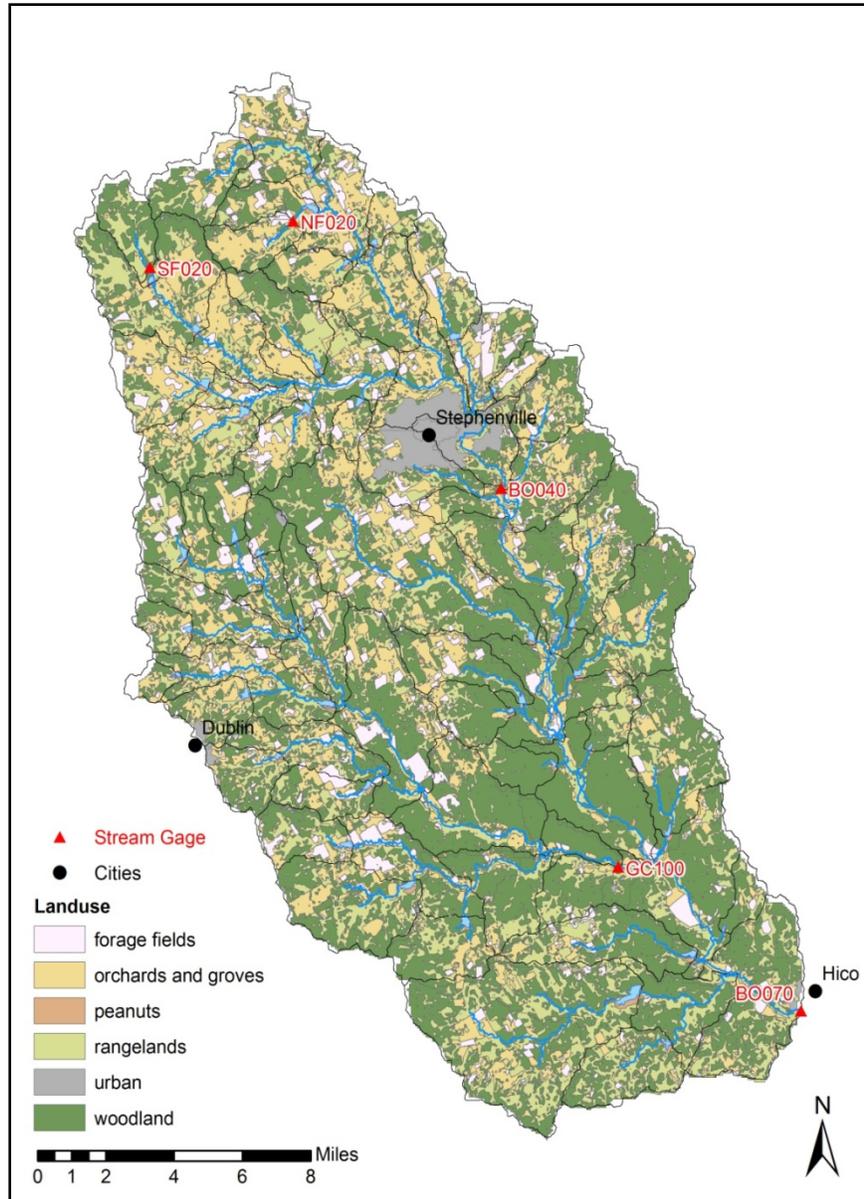


**Figure 53.** Basin Average Erodibility Factors for UNBRW (computed using SSURGO Data)

- K is the soil erodibility factor
- LS is the topographic factor
- C is the cover and management factor, and
- P is the support practice factor

A detailed description of each of the MUSLE parameters is included in [Table 34](#).

HEC-HMS uses the MUSLE method to determine sediment yield over a multiyear simulation. To utilize the MUSLE method for event analyses, HEC-HMS also requires a "Threshold" parameter which defines the flow rate below which no erosion occurs.



**Figure 54.** Land Use Types in UNBRW

The particle size distribution of source material in the subbasin (on the land surface) will vary from the particle size distribution within the stream channel at the basin outlet due to changing hydrodynamic forces as water flows over the land surface and concentrates within streams and channels. The degree of clay and silt enrichment in the suspended sediment is largely the result of preferential deposition of the courser fraction during the transport and delivery of sediment from its source to basin outlet. A quantity known as the enrichment ratio is used to convert the watershed particle-size distribution to an outlet particle-size distribution (Slattery, 1996). The enrichment ratio is calculated as shown in Equation 12.

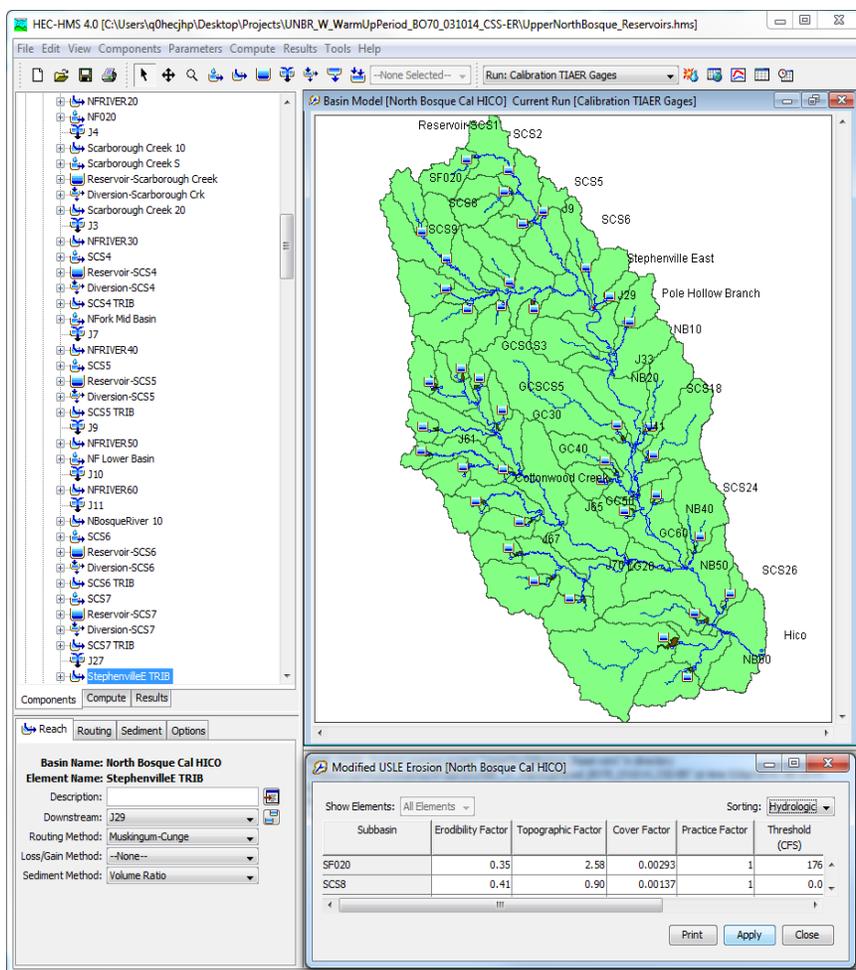


Figure 55. Sediment Model Setup in HEC-HMS

Table 34. Initial Parameters and Methods for Sediment Model

Parameter or Method	Description	Source	Initial Range or Method
<b>Subbasin</b>			
Erosion Method	The erosion method computes the total sediment load transported out of the subbasin. This calculation process is repeated for each event during the simulation time window.	n/a	Modified USLE
Erodibility Factor (K)	The erodibility factor describes the difficulty of eroding the soil. The factor is a function of the soil texture, structure, organic matter content, and permeability. Typical values range from 0.05 for unconsolidated loamy sand to 0.75 for silty and clayey loam soils.	SSURGO data	.29 - .45
Topographic Factor (LS)	The topographic factor describes the surface's susceptibility to erosion due to length and slope. It is based on the observation that long and steep slopes have more erosion than short and flat slopes. Typical values range from 0.1 for short and flat slopes to 10 for long or steep slopes.	DEM data	.78 - 5.61
Practice Factor (P)	The practice factor describes the effect of specific soil conservation practices, sometimes called best management practices. Agricultural practices could include strip cropping, terracing, or contouring. Construction and urban practices could include silt fences, hydro seeding, and settling basins. It is difficult to establish general ranges for these practices as they are usually highly specific.	n/a	1
Cover & Management Factor (C)	The cover factor describes the influence of plant cover on surface erosion. Bare ground is the most susceptible to erosion while a thick vegetation cover significantly reduces erosion. Typical values range from 1.0 for bare ground, to 0.1 for fully mulched or covered soils, to as small as 0.0001 for forest soils with a well developed soil O horizon (surface organic layer) under a dense tree canopy.	Land use data	0.00291 - 0.02796

Parameter or Method	Description	Source	Initial Range or Method
Threshold (CFS)	Only some precipitation events will cause surface erosion. The threshold can be used to set the lower limit for runoff events that cause erosion. Events with a peak flow less than the threshold will have no erosion or sediment yield.	n/a	0
Exponent	The exponent is used to distribute the sediment load into a time-series sedigraph. A small value flattens the sedigraph compared to the hydrograph. A large value heightens the sedigraph compared to the hydrograph.	n/a	0.75
Gradation Curve	The gradation curve defines the distribution of the total sediment load into grain size classes and subclasses at each subbasin outlet. The gradation curve is defined as a diameter-percentage function in the HEC-HMS Paired Data Manager.	Soil sample, SSURGO	n/a
Reach			
Transport Potential Method	Sediment processes within a reach are directly linked to the capacity of the stream flow to carry eroded soil. The transport capacity of the flow can be calculated from the flow parameters and sediment properties. If the stream can transport more sediment than is contained in the inflow, additional sediment will be eroded from the stream bed and entrained in the flow. However, if the flow in the reach cannot transport the sediment of the inflow, sediment will settle and be deposited to the reach bed. The sediment transport capacity is calculated using the transport potential method.	n/a	Laursen-Copeland
Sediment Routing Method	The volume ratio method links the transport of sediment to the transport of flow in the reach using a conceptual approach. For each time interval, sediment from upstream elements is added to the sediment already in the reach. The deposition or erosion of sediment 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.	n/a	Volume Ratio
Initial Bed Curve	The initial bed curve defines the distribution of the bed sediment by grain size at the beginning of the simulation.	Soil sample	n/a
Bed Width (ft)	The width of the sediment bed must be specified. The width should be typical of the reach and is used in computing the volume of the upper and lower layers of the bed model.	Field survey	5 - 70
Bed Depth (ft)	The depth of the bed must be specified. The depth should be typical of the total depth of the upper and lower layers of the bed, representing the maximum depth of mixing over very long time periods.	Field survey	10
Active Bed Factor	The active bed factor is used to calculate the depth of the upper layer of the bed model. At each time interval, the upper layer depth is computed as the d90 of the sediment in the upper layer, multiplied by the active bed factor.	n/a	2
Reservoir			
Trap Efficiency Method	The trap efficiency of a reservoir is the sediment load retained in the reservoir divided by the total sediment load entering the reservoir. The trap efficiency can be estimated by comparing the settling velocity of the sediment to a critical settling velocity (Chen, 1975).	n/a	Chen Sediment Trap
Fall Velocity Method	The deposition of sediment from the water column to the stream bed requires time. The fall velocity of the grain size class provides a physical basis for determining how much time is required for sediment in excess of the transport capacity to settle from the water column. The settling velocity is calculated and multiplied by the time interval to determine the settling distance in one time interval. This settling distance is then compared to the water column depth calculated during reservoir routing in order to determine the fraction of calculated deposition which is actually permitted during a time interval.	n/a	Van Rijn

Source: HEC-HMS User's Manual (HEC, 2013)

$$ER = \frac{\text{percent sediment in a given size class in outlet}}{\text{percent sediment in a given size class in watershed}} \quad (12)$$

where ER is the enrichment ratio. The numerator, percent sediment in a given size class in outlet, can be determined from a suspended sediment sample near the subbasin outlet. The denominator, percent sediment in a given size class in watershed, comes from the SSURGO soil data. Suspended sediment samples near the subbasin outlet are rare for all subbasins in a given study

watershed. For the UNBRW, the enrichment ratio was generated for select subbasins with suspended sediment samples near the basin outlet. These enrichment ratios were used to estimate the enrichment ratio of neighboring subbasins without suspended sediment samples near the basin outlet. For the UNBRW, enrichment ratios were calculated in a spreadsheet and final subbasin outlet grain size distributions were input into HEC-HMS as the gradation curve parameter.

2. **Reach Element:** The reach element connects other reach and subbasin elements, routing flow and sediment downstream. The reach element includes multiple methods for modeling sediment transport and erosion/deposition within a channel. Several sediment transport potential equations and sediment routing methods are available to route sediment through the stream network. For the UNBRW model, Laursen-Copeland (Copeland, 1989) was selected as the transport potential method during the calibration process for its applicability over the range of grain sizes present in the UNBRW and its ability to reproduce measured data well. The volume ratio method was chosen as the sediment routing method because it models relatively large distances between calculation points well. In addition to the transport potential method and sediment routing method, several additional parameters are required in the reach element. These parameters include the initial bed material gradation curve, bed width, bed depth, and active bed factor. For detailed explanation of these parameters see [Table 34](#).
3. **Reservoir Element:** The HEC-HMS reservoir element contains four methods to determine the sediment trap efficiency and compute sediment deposition in the reservoir. Trap efficiency represents the percentage of incoming sediment retained by the reservoir (e.g., (sediment inflow - sediment outflow)/sediment inflow). To model reservoir sedimentation in HEC-HMS, a trap efficiency method and fall velocity method must be selected. [Table 34](#) includes detail on these methods. For UNBRW model, the Chen trap efficiency method with the Van Rijn fall velocity method was selected. Chen's (1975) trap efficiency method was selected because the critical settling velocity, defined based on an ideal rectangular pond, is theoretically appropriate for the HEC-HMS conceptual reservoir system. The Van Rijn (1993) fall velocity method was selected for its theoretical robustness in using Reynolds number and wide use in practice. Chen's (1975) trap efficiency only works with a user-defined elevation-area relationship. HEC-HMS routes water through a reservoir with a storage-discharge curve, computing storage from the elevation-area curve using a conic shape simplification. After routing, HEC-HMS computes the elevation and surface area for each time step.

Based on the calculated discharge, surface area, and settling velocity, the trap efficiency is then calculated. After calculating sediment deposition based on the trap efficiency, the remaining suspended reservoir sediment is discharged through the reservoir based on a volume ratio of the flow discharge.

## **Boundary Conditions and Initial Conditions**

The boundary conditions required by HEC-HMS for this study were generated based on the precipitation data provided by the TIAER. Model initial parameters and initial conditions were adjusted through the calibration process with the warm-up period.

## **Surface Runoff Model Calibration and Verification**

The primary modeling component of HEC-HMS is the hydrology module. The hydrology model simulates precipitation-runoff processes for dendritic watershed systems. Before preparing the surface erosion and sediment transport model, the surface runoff model must be calibrated to available data. The quality of a sediment simulations are contingent on the quality of the hydrology model, so a carefully calibrated flow model is a necessary pre-requisite for an HEC-HMS sediment model. The UNBRW HEC-HMS hydrology model was calibrated using measured five-minute flow data, fifteen-minute rainfall data, and monthly pan evaporation data during the period 1 January 1995 through 31 December 1996. Hydrology model calibration parameters are contained in [Table 35](#). The range of values indicated in the "Range or Method" column of [Table 35](#) is the calibrated range of values specified for all elements in the UNBRW model. The reasonable ranges of calibrated values vary depending on the application. After calibration, the HEC-HMS model was then independently validated with a separate two-year dataset for the period 1 January 1997 through 31 December 1998. Statistics (mean, standard deviation (SD), and mean error (ME)) of daily flow at five sample sites within UNBRW during calibration and validation periods are shown in [Table 36](#). As evident in the results, the standard deviation is much larger than the average flow. This is attributed to flows at the observation points having a few relatively large peak flow values within a record of significantly smaller peak flow values. The relationship between mean and standard deviation can be a function of watershed size, watershed regulation, and the precipitation pattern. This chapter assumes that a calibrated hydrology model for the watershed has been established. Further information on the calibration of a hydrology model can be found in Chapter 6 of the HEC-HMS User's Manual (HEC 2013).

HEC-HMS includes a collection of modeling components to simulate land surface erosion, simplified channel transport, and reservoir routing. After the HEC-HMS model was calibrated to the observed stream flow data,

**Table 35.** Calibrated Parameters and Methods for Hydrology Model

Parameter or Method	Description	Range or Method	Notes
<b>Subbasin</b>			
Canopy Method		Simple Canopy	
Canopy Storage	canopy interception storage (inches)	0.1	
Surface Method		Simple Surface	
Surface Storage	surface depression storage (inches)	0.5 - 0.6	This value was adjusted during calibration.
Loss Method		Soil Moisture Accounting	
Soil Storage	total water storage in rooting zone (inches)	6	
Tension Storage	water held in tension storage in rooting zone (inches)	4	
Maximum Infiltration Rate	maximum infiltration rate from surface into the soil (inches/hour)	0.4 - 1.2	This value was adjusted during calibration.
Soil Percolation	percolation rate from soil layer into Groundwater 1 Layer (inches/hour)	0.3 - 0.5	This value was adjusted during calibration.
Groundwater 1 Storage	water storage in Groundwater 1 Layer (inches)	3	
Groundwater 1 Percolation	percolation rate from Groundwater 1 Layer into Groundwater 2 Layer (inches/hour)	0.3 - 0.5	
Groundwater 1 Coefficient	Groundwater 1 linear reservoir coefficient (hour)	3.7 - 32.4	This value was adjusted during calibration.
Groundwater 2 Storage	water storage in Groundwater 2 Layer (inches)	10	
Groundwater 2 Percolation	percolation rate from Groundwater 2 Layer into deep aquifer (inches/hour)	0.06 - 0.1	This value was adjusted during calibration.
Impervious Area %	percentage of subbasin directly connected to the stream network	0 - 5	
Groundwater 2 Coefficient	Groundwater 2 linear reservoir coefficient (hour)	200 - 400	This value was adjusted during calibration.
Transform Method		Clark Unit Hydrograph	
Time of Concentration	travel time along the longest flow path (hour)	0.9 - 8.1	Values were determined from TR-55 method and adjusted during calibration.
Storage Coefficient	linear reservoir coefficient (hour)	0.9 - 8.1	This value was adjusted during calibration.
Baseflow Method		Linear Reservoir Baseflow	
Groundwater 1 Coefficient	groundwater 1 linear reservoir coefficient (hour)	2	This value was adjusted during calibration.
Groundwater 1 # of linear reservoirs	number of linear reservoirs	1	
Groundwater 2 Coefficient	Groundwater 2 linear reservoir coefficient (hour)	500 - 1,000	This value was adjusted during calibration.
Groundwater 2 number of linear reservoirs	number of linear reservoirs	1	
<b>Reach</b>			
Routing Method		Muskingum-Cunge	
Reach Length	channel reach length (feet)	1,200 - 33,000	
Reach Slope	channel energy slope (feet/feet)	0.0009 - 0.0215	
Channel n	channel Manning's <i>n</i> value	0.035	
Overbank n	left and right Manning's <i>n</i> value	0.05	
Channel Shape (cross section)	eight-point cross section of channel		Field Measurements
<b>Reservoir</b>			
Storage Method:		Elevation-Area-Discharge	
Elevation-Area curves	elevation versus area curve for reservoir pool		
Elevation-Discharge curves	elevation versus. discharge relationship including low level outlets and spillway		

**Table 36.** Statistics (Mean, SD, and ME) of Daily Flows at Five Sampling Sites during Calibration and Validation Periods

Gage Name		Hydrology Analysis: Flow (ft <sup>3</sup> /s)					
		Calibration (01Jan1995 to 31Dec1996)			Validation (01Jan1997 to 31Dec1998)		
		Mean	SD	ME	Mean	SD	ME
NF020	Measured	0.94	5.00	-	1.78	11.14	
	HMS	1.29	6.23	0.35	2.00	8.79	0.22
SF020	Measured	1.35	7.69	-	1.23	7.63	
	HMS	1.66	9.78	0.31	1.76	10.76	0.53
BO40	Measured	33.79	99.11	-	34.65	131.43	
	HMS	32.56	123.94	-1.23	42.50	128.91	7.85
GC100	Measured	30.83	92.80	-	57.21	183.11	
	HMS	31.61	127.48	0.78	40.42	162.30	-16.79
BO070	Measured	102.02	303.04	-	149.96	479.31	
	HMS	107.97	432.24	5.94	142.08	453.57	-7.88

sediment model was initialized in HEC-HMS by activating the sediment feature on the component editor for the basin model. Detailed instruction for sediment model initialization is contained in the HEC-HMS User's Manual, Chapter 16 (HEC, 2013). An overview of sediment model initialization steps includes the following:

- Select sediment methods (transport potential, fall velocity, and grade scale) and enter parameter data (specific gravity and density) that will be applied to all elements within the basin model.
- Select a surface erosion method for each subbasin element.
- Select a channel sediment routing method for each reach element.
- Select a sediment trap efficiency method for each reservoir element.

## Surface Erosion and Sediment Routing Model Calibration and Verification

The subbasin, reach, and reservoir elements were parameterized using available data from the SSURGO soil database, field sediment sample data, and reservoir rating curves. The model was calibrated using measured TSS data. Specific surface erosion, transport potential, sediment routing, trap efficiency, and fall velocity methods were selected among the available options during the calibration process as described in the "Model Selection and Fitting" section above. The UNBRW calibration assumed that TSS included all clay, silt, and sand and excluded gravel. Therefore, TSS data were compared to the sum of the clay, silt, and sand sediment load outputs from HEC-HMS. A summary of calibrated sediment model parameters and methods is included in [Table 37](#). Parameters were adjusted within the applicable range during the sediment model calibration process. The "Calibrated Range or Method" column of [Table 37](#) displays the calibrated range of values used in the UNBRW model. The reasonable

**Table 37.** Calibrated Parameters and Methods for Sediment Model

Parameter or Method	Calibrated Range or Method	Calibration Notes
<b>Subbasin</b>		
MUSLE Method: Cover & Management Factor (C)	0.00137 - 0.03492	The Cover & Management factor was the only MUSLE factor selected for calibration. The Cover & Management factor is the least physically based MUSLE factor. Cover & Management values were adjusted to 47-179% of initial values during calibration.
Threshold (cfs)	0 - 176	Threshold values were calibrated to measured data. A threshold value of 0 was used for all basins with the exception of SF020 for which a value of 176 cfs was used.
<b>Reach</b>		
Transport Potential Method	Laursen-Copeland	The Laursen-Copeland (Copeland, 1989) method was selected for its broad spectrum of gradation applicability (developed for use inclusive of the silt range) and physical basis.
Sediment Routing Method	Volume Ratio	
Bed Gradation Curve	75% - 86%	The percentage of gravel was adjusted during calibration.
<b>Reservoir</b>		
Trap Efficiency Method	Chen Sediment Trap	The Chen (1975) Sediment Trap method was selected for its physical basis.
Fall Velocity Method	Van Rijn	The Van Rijn (1993) method selected for its theoretically robustness and wide use in practice.

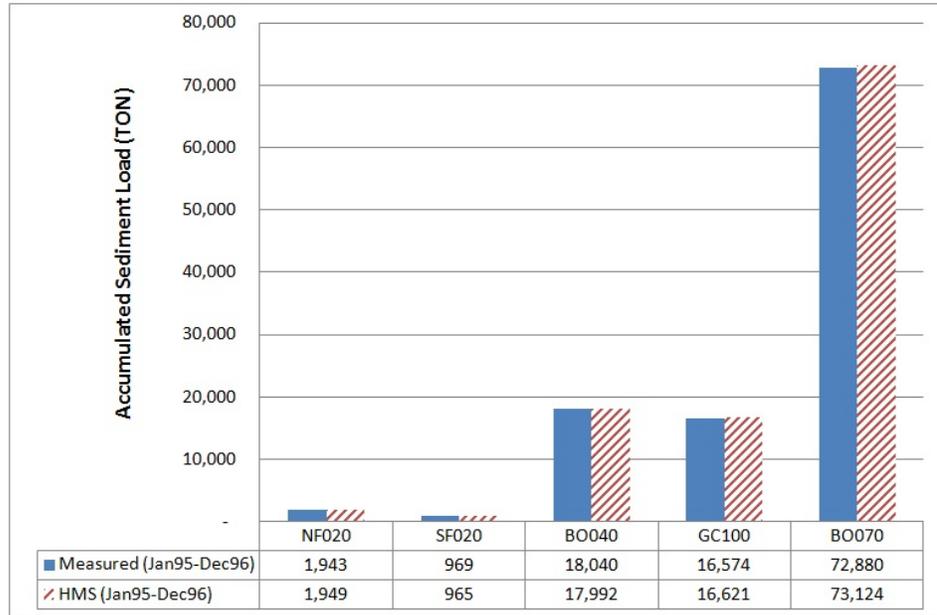
range of values will vary depending on the particular application. The model parameters used during the validation simulation were identical to the parameters determined during model calibration.

The model calibration progressed from small, single sub-basin elements upstream, to larger, more complex, watershed components downstream, with the former informing the later. Single subbasin elements were calibrated to measured TSS data, at stations NF020 and SF020 by adjusting the MUSLE cover & management factor (C) for the two subbasins within 70 to 90 percent of initial values. In addition, the model at SF020 was calibrated by adjusting the threshold flow to 176 cfs. The model at stations *BO040*, *GC100*, and *BO070* were then calibrated based on adjusted parameters from upstream stations (*NF020* and *SF020*) and measured TSS data. The MUSLE cover and management factors were adjusted to 47 to 179 percent of initial values. The bed gradation curves were adjusted to 75 to 86 percent of initial percentage of gravel.

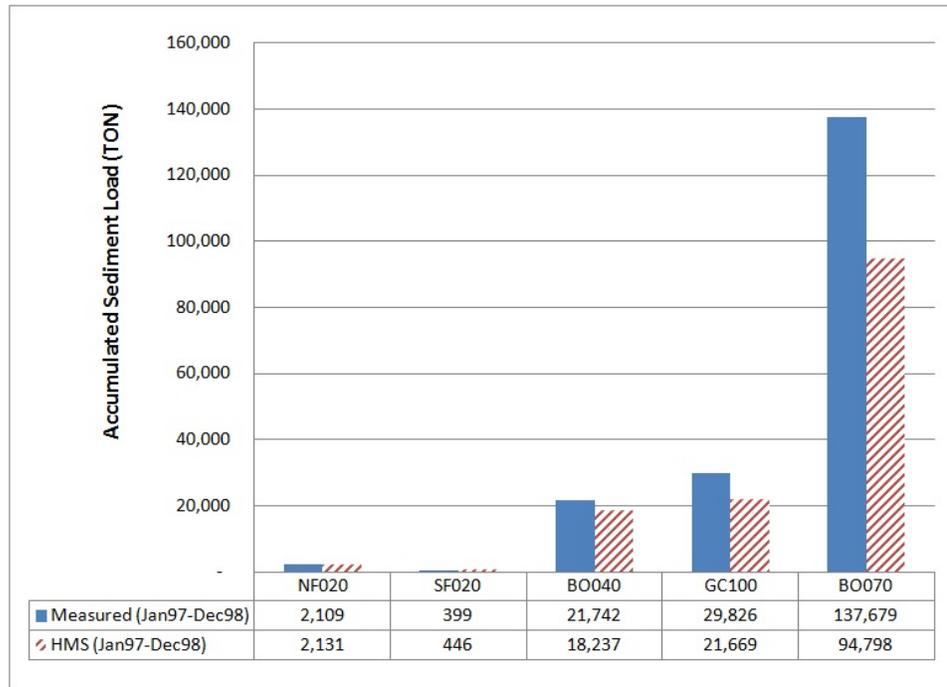
## Results and Discussion

A comparison of accumulated sediment load at each stream gage location over the duration of the calibration period is included in [Figure 56](#). HEC-HMS approximated measured TSS data for the calibration period well.

Accumulated computed sediment load at each stream gage location is compared to the cumulative TSS over the duration of the *validation* period in [Figure 57](#). While the validation results diverge from measured TSS data more than the calibration results, the model still provides a reasonable



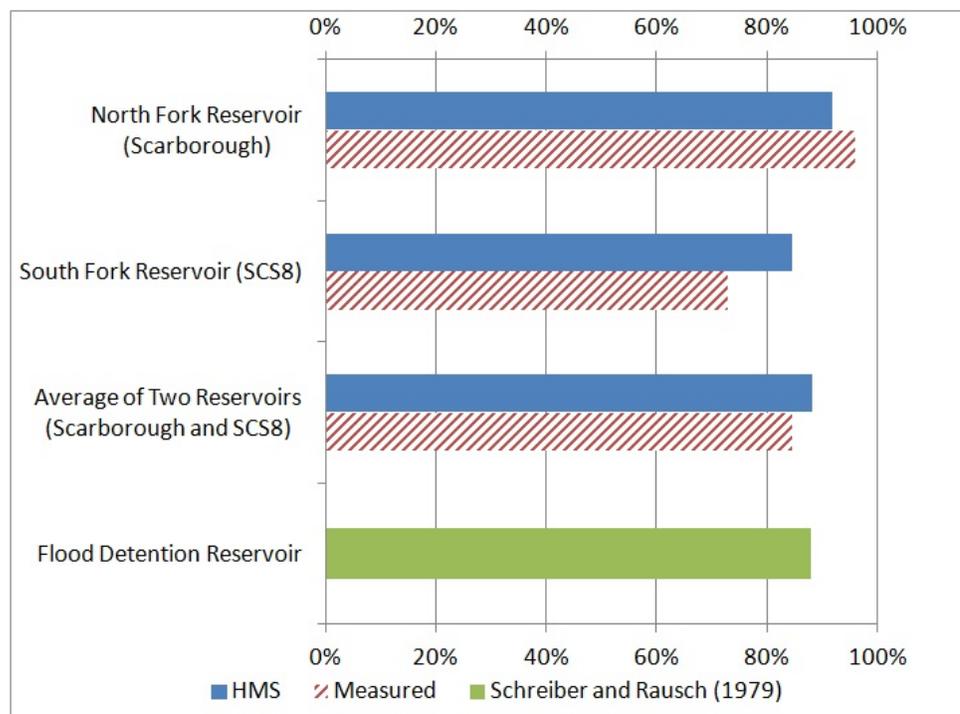
**Figure 56.** Comparison between Measured and Computed Sediment Load during Calibration Period



**Figure 57.** Comparison between Measured and Computed Sediment Load during Validation Period

approximation (-30% ~ 12%) of measured TSS data for the validation period.

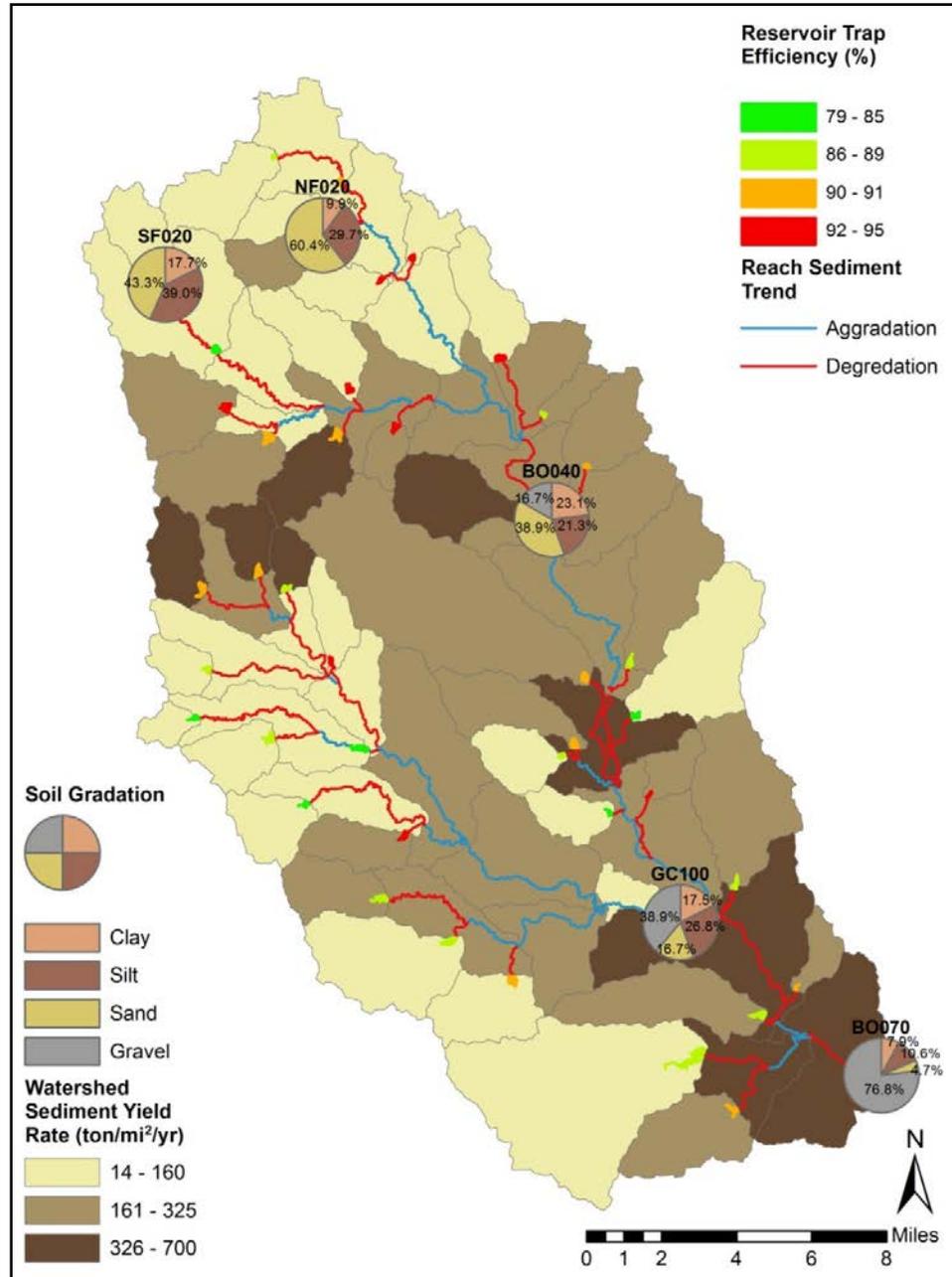
Reservoir trap efficiency rates were also computed and compared to measured values for both North Fork Reservoir (Scarborough) and South Fork Reservoir (SCS8) in [Figure 58](#), the only reservoir locations with



**Figure 58.** Comparison between Measured and Computed Reservoir Trap Efficiency Rate (percent)

measured data in the UNBRW. An HEC-HMS computed average reservoir trap efficiency of 88 percent for the two reservoirs was very close to the observed value of 85 percent. These results are consistent with a Schreiber and Rausch study (1979) which concluded that the average trap efficiency was also 88 percent for a flood detention reservoir located in an agricultural area near Columbia, Missouri (4,167 feet long by 230 feet wide with a mean depth of seven feet and a maximum depth of sixteen feet).

GIS was utilized to visualize sediment results of the UNBRW modeling as shown in Figure 59, displaying HEC-HMS sediment results for each subbasin, reservoir, and reach element. The 68 subbasin elements are displayed with three categories for the estimated unit sediment yield rate (ton/mi<sup>2</sup>/yr). The 84 reach elements are displayed with two categories to indicate predictions of aggradation and degradation in channels. The forty reservoir elements are displayed with four categories of reservoir trap efficiency rates. The five stream gage locations are displayed with a pie chart representing soil gradation of cumulated sediment loads for four grain classes (clay, silt, sand, and gravel) at each gage location. These data were generated from the HEC-HMS model and retrieved using HEC-DSSVue. Subbasin sediment yield rates provide an indication to watershed management officials of major non-point sediment source contributors in the watershed.



**Figure 59.** Calibration Results for Subbasin Surface Erosion Rates (ton/mi<sup>2</sup>/yr), Reservoir Trap Efficiency Rates (percent), Reach Erosion Trend (aggradation/degradation), and Soil Gradation (percent) at Stream Gage Locations during Calibration Period (1 January 1995 - 31 December 1996)

Reach sediment trends (aggradation/degradation) indicate that reach bed sediment is building up (aggradation) or eroded away (degradation). Reservoir trap efficiency rates help estimate reservoir life expectancy and plan reservoir maintenance.

These results indicate major non-point source contributors, long term TSS trends, and reservoir sedimentation rates that can inform watershed

managers and local government decisions, basing TMDL and water quality standards on quantifiable data.

### **Additional analysis**

In general, if sediment aggradation/degradation trends for a reach are desired, then HEC-HMS is a sufficient model. If quantity and location of sediment deposition within the reach is desired, then HEC-RAS is a more applicable model. If a more detailed channel sediment routing analysis is warranted HEC-HMS can be used to compute local sediment loads to the main channels, then using HEC-DSS, local sedigraphs for all grain classes (clay, silt, sand, and gravel) can be supplied as input boundary conditions to the HEC-RAS sediment model.

## **Summary**

The objective of this study was to provide watershed sedimentation information to decision makers to inform management decisions related to soil surface erosion, water quality, and reservoir capacity. An HEC-HMS model of the UNBRW was created with representation of physical basin characteristics based on watershed soil, terrain, land use, and precipitation data. The HEC-HMS model was calibrated and validated based on observed data collected within the watershed. Model results produced valuable sedimentation information including the following:

- The sediment yield rate for each subbasin giving indication of the most erodible regions in the watershed.
- Sediment time series (sedigraph) for each grain class to provide boundary conditions to hydraulic sediment models such as HEC-RAS.
- Long-term aggradation and degradation trends for each reach to indicate reaches with higher erosion rates. This information could be used to assist in formulating possible corrective measures to enhance fish and wildlife habitat, water quality, flood control, and bank stability.
- Average trap efficiency rate for each reservoir to calculate the reasonable life expectancy of each reservoir.

This information can be used to assist management officials in determining non-point source contributors in the watershed and TSS loading within the watershed, as well as establishing TMDLs to achieve water quality standards.

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# CHAPTER 9

## Flood Forecasting Application

### Background

Streamflow forecasts are fundamental to the effective operation of flood control reservoirs and levee systems. Streamflow forecasts provide hydrographs (time of flood flows and volume) and peak flow estimates to inform reservoir operation. Forecasts may also support emergency operations by providing estimates of the timing and extent of expected hazardous or damaging flood conditions in the form of inundation mapping. Streamflow forecasts can be developed using a precipitation-runoff model of the watershed. The forecast simulations are based on recent meteorologic and hydrologic conditions in the watershed (including snow pack and soil moisture conditions). The forecast simulations may also incorporate predicted future meteorologic conditions. HEC-HMS includes a forecast alternative simulation type with features that were specifically added for flow forecasting. The forecast alternative simulation type differs from the basic simulation run in that it is meant for real-time forecasting with quick calibration methods built in. Forecast uncertainty is assessed by changing parameter initial states and meteorologic boundary conditions.

### General Process

The objective of precipitation-runoff modeling within a flood forecast is to provide reliable hydrograph estimates in a time-sensitive manner. The modeling effort must produce accurate results, which are used to provide information about flooding magnitude and are used to inform future operation of reservoirs and other flood risk management projects. The main components of either developing an HEC-HMS precipitation-runoff model or applying the HEC-HMS model to a flood forecast are listed below.

1. Develop an HEC-HMS model for the watershed and calibrate the model to a range of precipitation-runoff events. Model development includes delineation of subbasins and reaches, selection of a meteorological model type that will be used for model calibration, validation, and forecast alternatives (gridded versus rain gage approaches), and initial parameterization of the model. The calibration events should include the type of events that the model will be applied to when used in forecasting mode.

Additional events should be simulated to verify the model and selected parameter sets. Finally, document model development, model parameter information, and modeling assumptions that would be useful for others when applying the model to a flood forecast.

2. Once the basic HEC-HMS model has been configured, then it can be customized for performing forecast simulations. Part of the process for configuring a forecast alternative simulation is to create zone configurations and select basin model elements within each zone for parameter adjustments. During the development of a forecast alternative, it is common to group hydrologically similar subbasins or reaches together so that parameter adjustments can be applied uniformly across all elements in the group. HEC-HMS includes zone configurations to group similar features together. A parameter adjustment can be entered once and applied to all of the subbasins/reaches within the zone. A local element override can be used if a subbasin/reach within a zone diverges in behavior from the rest of the subbasins/reaches in that same zone. More information on creating zones is described in Chapter 6 of the HEC-HMS User's Manual (HEC, 2016).
3. Choose the relevant basin model (e.g., dry, average, wet, summer, winter, etc.) and meteorologic model (e.g. rainfall only, rainfall with snowmelt, no future precipitation, with future precipitation, etc.) to be used in the forecast alternative. A forecast alternative is constructed using the same initial conditions and parameters found in the selected basin model. Parameters can be adjusted using zonal editors, blending, and baseflow and reservoir initialization editors.
4. Choose a time interval to compute the forecast alternative. The time interval must adequately capture the hydrograph peak, and in some cases the total runoff volume is needed for reservoir operations.
5. Select appropriate zonal configurations for loss rate parameters, transform parameters, baseflow parameters, and/or routing parameters. Zones may be configured based on existing stream gage locations, soil types, etc. that group elements which will have similar parameters.
6. Determine a time window for the forecast alternative. The time window includes a start date and time, an end date and time, and a time of forecast. Typically, the time of forecast represents the last available time for meteorologic observations (future meteorologic conditions can be included within the forecast alternative). The simulation can be started hours, days, or weeks before the time of

forecast, depending on the watershed and/or meteorologic conditions. The difference between the start time and the time of forecast is called the look back period. The look back period can be used to compare observed data with the computed results to help with real-time model calibration. Model calibration is where model parameters (including initial conditions) are adjusted to improve model results. The end date is typically only a few days into the future. The length of the look back period and forecast period depends on the response time of the watershed. Small watersheds may only need a two to three day look back period for a one day forecast. A larger watershed may need a seven day look back period for a three to four day forecast period. Some special simulations may use a forecast period exceeding a week in duration, like a snowmelt simulations in larger watersheds.

7. Set boundary conditions for a flood forecast. The meteorologic model defines the boundary conditions for the simulation. Boundary conditions include precipitation, evapotranspiration, and snowmelt. Observed flow data can also act as a boundary condition if blending is used, or used as flow at a source element or at a reservoir element with the specified release option turned on.
8. Initialize conditions for a flood forecast, including baseflow and reservoir storage. Information on using the Forecast Initial Baseflow and the Forecast Reservoirs editors can be found in Chapter 14 of the HEC-HMS User's Manual (HEC, 2016).

Baseflow usually dominates flow composition at the beginning of a forecast simulation. Observed flow provides one good source for calculating the baseflow initial condition. When there are multiple subbasins above the observed flow location, a ratio can be applied to the observed flow to calculate the baseflow initial condition for each subbasin. The Forecast Initial Baseflow editor includes methods for initializing subbasin flow contributions, and it uses the baseflow zone configurations to assist in organizing data entry.

Pool storage is a key factor in reservoir discharge. The initial condition must be specified for each reservoir at the start of the forecast simulation. When using forecast alternatives, initial elevation can be selected as the initial condition for each reservoir in the basin model. The Forecast Reservoirs editor provides a choice of methods to estimate discharge data for the forecast period. Reservoir storage can also be initialized using observed elevation data with this editor. The Forecast Reservoirs editor also contains an option for updating the reservoir elevation at the time of forecast using an observed elevation.

9. Calibrate the real-time model efficiently. Subbasins and reach parameters are initialized to default values using parameter values from the existing basin model. Parameter values in the elements may be adjusted for the forecast without altering the original parameter values in the basin model using the features described below. The order of parameter adjustment within a forecast alternative is 1) parameter values are taken from the basin model, 2) adjustments are made based on information defined within the zonal editors, 3) individual element overrides are applied, and 4) initialization from observed data (initial baseflow and reservoir stage) are applied. Information about forecast parameter adjustments, slider adjustments, and blending can be found in Chapter 14 of the HEC-HMS User’s Manual (HEC, 2016).

- a. **Forecast Parameter Adjustments.** Zone parameter adjustments are entered using the parameter editors for the forecast alternative. A separate parameter editor is available for each loss rate, transform, baseflow, and routing method used in the basin model. Figure 60 shows the Forecast Deficit and Constant Loss parameter adjustment editor. Adjustments are set for each zone using the upper half of the editor, while individual element parameter overrides can be applied using the bottom half of the editor.

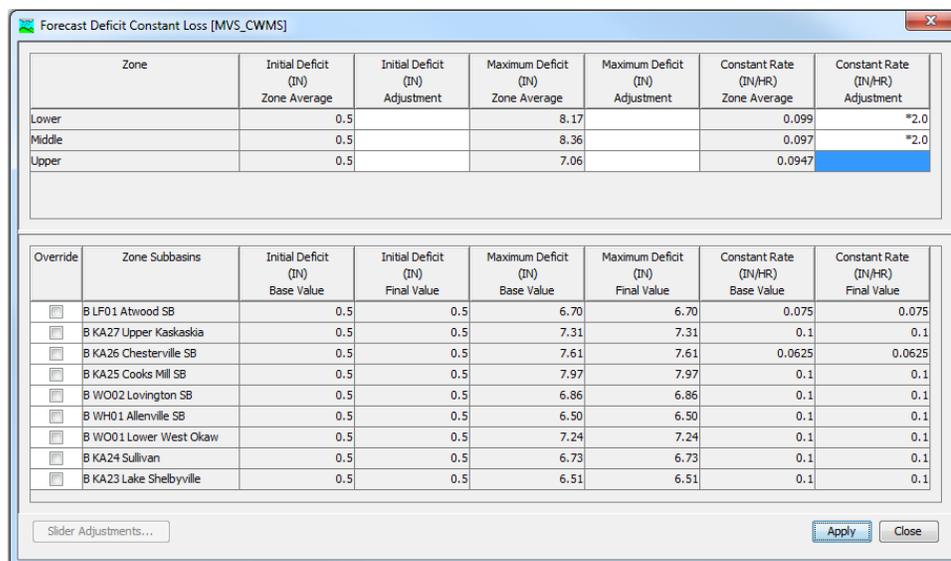


Figure 60. Forecast Deficit and Constant Loss Parameter Adjustment Editor

- b. **Slider Adjustments.** Slider adjustments are an alternate way to view and change a zone parameter adjustment or an element parameter override. Instead of typing a new value for the adjustment, a slider can be moved to quickly obtain a new adjustment factor.

- c. **Forecast Blending.** Blending provides a method for systematically using observed flow to adjust computed streamflow. If blending is used, observed flow replaces computed streamflow for the look back period, and a transition from observed flow to computed streamflow is applied starting at the forecast time. Three different options are available for performing the transition between observed flow and computed streamflow: step, taper, or none. More information on the forecast blending options can be found in Chapter 14 of the HEC-HMS User’s Manual (HEC, 2016).
  - d. **Computing to a Computation Point.** Any hydrologic element in a basin model can be designated as a computation point. A forecast alternative can be computed to a computation point rather than computing the entire model each time a change is made. One approach to calibration is to start at the most-upstream element with observed flow; after adjusting parameters above the upstream point, calibration efforts are focused on the drainage between the upstream point and the next point downstream. The computing to a point feature in HEC-HMS supports this calibration approach.
  - e. **Model Computational Efficiency.** The HEC-HMS modeling software is designed to be computationally efficient. Only components with data changes since the last compute will be recomputed. This design provides greater efficiency as changes are made to elements further downstream in the basin model, since changes made to upstream elements affect results in each element downstream. The forecaster may choose to force all components to be recomputed if it is desired (holding down the control key when pressing the compute button forces all elements to be computed).
10. View and analyze results; extract results for further analysis. The output from HEC-HMS gives the forecaster stage and flow hydrographs from each hydrologic element. These results can be imported into additional modeling software such as HEC-ResSim for reservoir operations and HEC-RAS for inundation mapping.

## Authority and Procedural Guidance

The following are USACE guidance on flood forecasting:

- ER 1110-2-240 (USACE, 2016) states that flood forecasts may be used for planning future operations; however, actual releases should be determined following operation in the water control manual and based on observations, to the extent practical. Flow

forecast alternatives can be used to determine impacts of different scenarios on reservoir operations and streamflow.

- EM 1110-2-3600 (USACE, 1987) discusses methods of hydrologic analysis which are applicable to management of water control systems. Forecasts “simulate the continuous natural response of hydrologic and river systems, combined with the effects of project regulation on conditions of streamflow and river stages”. Forecasts can evaluate different alternatives for streamflow and project regulation, and forecasts can reduce damage in unprotected areas, making them an important element in the management of water during floods.

## Forecasting Procedures

To meet the objectives of a flood forecast analysis, peak flow, total runoff volume, hydrograph timing, and peak reservoir stage are often required. These values are calculated for a look back period and then a forecast period (typically 3-5 days into the future). In general, the modeling process includes two parts: model development and flood forecasting. Model development typically utilizes the simulation run compute type where the time window is set to a historic flood event and the model is calibrated to observations from the historic event. A separate basin model is used for each calibration event (each basin model has a unique set of parameters). Flood forecasting uses the forecast alternative compute type which includes forecast specific parameter editors that efficiently estimate initial condition and allows for quick adjustments to the base model. The base model is used as a starting point for real-time calibration.

Model development steps include:

1. Select appropriate methods to represent watershed,
2. Collect watershed data and characteristics,
3. Utilize regional studies and equations to estimate parameter values,
4. Calibrate the model using historical data,
5. Validate the model using historical event data, and
6. Develop model parameter sets for different watershed conditions (such as dry, average, and wet conditions). These parameter sets can be used to initialize the flood forecasting model and will then be refined for the flood event being modeled.

Steps for flood forecasting include:

1. Configure the forecast alternative,
2. Calibrate the model to observed data in the look back period by adjusting model parameters,
3. Import future precipitation, and
4. Run the calibrated model to develop a flow forecast.
5. Analyze and view the results. Model results can be extracted for additional model applications such as HEC-ResSim and HEC-RAS.

## **Case Study: Flood Forecasting in the Kaskaskia Watershed**

A case study is presented to illustrate the steps for developing an HEC-HMS forecast alternative and applying the forecast alternative to generate multiple forecast scenarios. The case study includes a description of the decisions required when configuring the model and the types of watershed information needed to build the model. Selection of methods for modeling hydrologic processes is included as well as model calibration and validation. Finally, the case study shows how to create a forecast alternative simulation and steps for applying the forecast alternative for hydrologic simulation.

### **Watershed Description**

The Kaskaskia River watershed is located in central Illinois with headwaters near Champaign, Illinois and the outlet to the Mississippi River just upstream of Chester, Illinois. The watershed is approximately 5,790 square miles in total drainage area. Two reservoirs are operated by USACE on the Kaskaskia, Lake Shelbyville and Carlyle Lake. Several other smaller reservoirs that are not operated by USACE were not included in the Kaskaskia HEC-HMS model. The total drainage area upstream of Shelbyville Dam is 1,054 square miles, and total area draining to Carlyle Dam is 1,663 square miles. The Kaskaskia Lock and Dam is located just upstream of the outlet and provides a minimum depth for navigation. The Kaskaskia River watershed, including the location of dams and tributaries, is shown in [Figure 61](#). Slopes in the watershed are typically gentle, except for a few portions of steeper slopes. Soil types vary throughout the watershed, but, generally, the upper portions are composed of soils with higher infiltration rates, while middle and lower portions are composed of soils with lower infiltration rates (USACE, 2013).

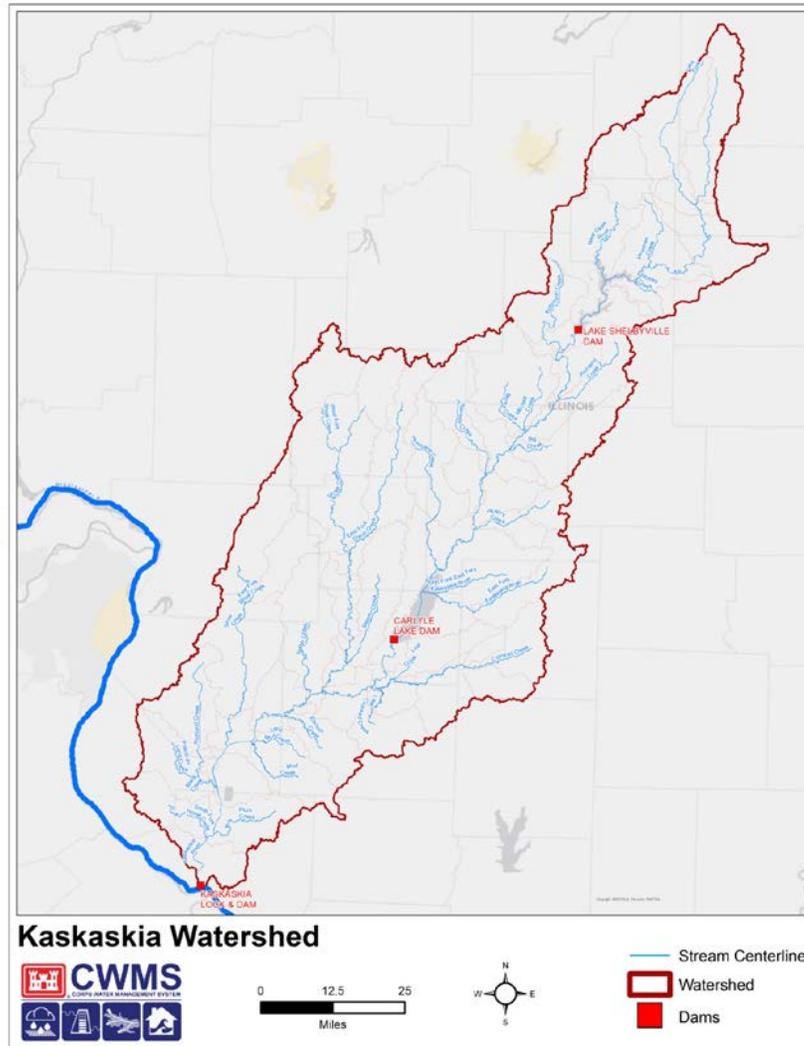


Figure 61. Kaskaskia Watershed

## Decisions Required

For the development of an HEC-HMS model to be applied to flood forecasting, the following questions must be answered:

- What is the appropriate level of subbasin delineation for a forecast model? (Typically, a flow forecast will need to be completed in a timely manner and a complex or detailed model is not required to determine adequate results.)
- What are the appropriate modeling methods for a forecast model? (Some locations will need a continuous simulation model where evapotranspiration and soil moisture need to be included in the model. In addition, previous studies or regional preferences for specific modeling methods can help determine which modeling methods to select).
- Which calibration events should be chosen to give an adequate range of parameter values? (Select as many calibration, and

validation, events as possible to build multiple parameter sets that could be used for dry, average, wet, and seasonal conditions.)

Once a model has been created and calibrated to historic events, the forecast alternative compute type must be configured before the forecast alternative can be used for efficient forecast simulations. The following question must be answered when configuring the forecast alternative:

- How should zones be defined for subbasins or reaches? (Zones should be defined in a way to expedite calibrating the model in real-time. Grouping subbasins and reaches into zones of homogenous watershed characteristics, or by elements within major tributaries or upstream of an observed flow gage could be helpful.)

Once a forecast model has been created, the modeler must run a forecast. To achieve the most accurate forecast results, the following questions must be answered:

- What are the current watershed conditions? Which basin model should be used in the forecast alternative configuration? (Understanding the state of the watershed at the beginning of the forecast simulation will help identify which parameter set to use. Some additional model calibration will be necessary, but starting with the right parameter set will reduce the amount of time needed to develop a well calibrated model.)
- What are the appropriate values with which to initialize the model baseflow and reservoir stage? (Observed information should be used to the extent possible to initialize baseflow and reservoir stage.)
- How should zonal adjustments be used to calibrate the model? (Zonal adjustments should be used to adjust the base model's parameter set so the model can reproduce the observed flow within the look back period. Once the model is calibrated, it can be used to predict precipitation-runoff in the forecast period.)

## Information Required

To answer the questions above, the forecaster will use an HEC-HMS model to compute the flood hydrograph for the different watershed conditions. To develop an HEC-HMS model, the following information is required:

- Basin data: Soil types and infiltration rates, land use characteristics and percent impervious, physical geometry including lengths and slopes, local precipitation patterns, drainage patterns, channel geometry and conditions.

- Reservoir and stream gage locations and watershed characteristics are considered for subbasin delineation. Flows at forecast locations are required for a flood forecast model.
- Physical characteristics for reservoirs, such as elevation-storage curves, outlet dimensions, etc.
- Observed precipitation and flow data for a range of calibration events.
- A reasonable range of model parameters should be developed by synthesizing calibration results.

For flood forecasting,

- Future meteorologic conditions, precipitation (QPF) and temperature, for the forecast period.
- Understanding of other models and their data needs (reservoir simulation, hydraulic, and consequence models will use flow forecasting results from HEC-HMS), as well as the data needs of emergency management and the reservoir regulators.

For the Kaskaskia basin model, the information required was found using national databases, such as the Gridded Soil Survey Geographic (gSSURGO) Database from the NRCS, data records from gages obtained through the Geostationary Operational Environmental Satellites (GOES) networks and other national data sources, existing models, and regional studies.

## **Spatial and Temporal Extent**

The study area has been defined as the Kaskaskia River watershed. The next step in model development is to divide the study area into subbasins.

If a detailed DEM is available, HEC-GeoHMS tools can be used to delineate the subbasins, establish the flow paths, and calculate physical parameters of the watershed (such as length, centroid location, and average slope). However, if the best DEM available for the watershed is a 30-meter DEM, the resolution of the terrain data may not be sufficient to gather accurate information from the HEC-GeoHMS tools. Additional information, like the published stream and subbasin boundaries, can be used to improve the GIS based delineation. For the Kaskaskia watershed, a 10-meter DEM was available from the USGS NED as well as LiDAR grids for some counties within the basin. The subbasin delineation from the 10-meter DEM was verified against published drainage areas, at gage locations, and published delineations from the National Hydrography Dataset.

Because gage data from historical events were available, subbasins were delineated such that outlet points were at existing stream gages where

possible. These data were used in the calibration of the subbasins in the watershed model.

The temporal resolution of the model also needs to be selected. Some methods in the basin model have parameters that are sensitive to time interval, so those parameters have to be estimated with the knowledge of what the time interval will be when the model is run. The forecaster must ensure the peak of the hydrograph is adequately captured when selecting a time step. For the Kaskaskia watershed, a time interval of one-hour was selected.

## Model Method Selection

Creating the Kaskaskia River HEC-HMS model included selecting modeling methods for rainfall loss, runoff transform, baseflow, and channel routing. Modeling method selection is often made based on familiarity of the modeling method, availability of existing models to aid in model parameterization, and whether the modeling method is appropriate for the intended application. Modeling methods and applicable starting parameter estimates are described below.

- Rainfall loss. The deficit and constant loss method was selected. The deficit and constant loss method is one of two loss methods that are appropriate for continuous simulation (the deficit and constant and soil moisture accounting loss methods simulate wetting and drying of the soil due to evapotranspiration). Two watershed conditions, wet/typical and dry, were modeled for the Kaskaskia basin. The initial deficit starting value was estimated as half the available water storage for the wet/typical condition and equal to the available water storage for the dry condition.
- Runoff transform. The Modified Clark method was selected to model runoff transform. The Modified Clark method utilizes the spatial variability contained within gridded radar rainfall (the other transform methods require gridded precipitation to be averaged over the subbasin). The Modified Clark method requires time of concentration and storage coefficient parameters. The TR-55 method was used to determine a starting value for time of concentration. The storage coefficient was estimated based on previous models of the Kaskaskia.
- Baseflow. The recession method was used to model baseflow. Parameters used to model baseflow were estimated using available gage data, historic flow records, and engineering judgement.
- Channel Routing. A combination of Modified Puls and Muskingum channel routing methods were selected. An HEC-RAS model was available to develop the storage-discharge curves needed for the reaches using the Modified Puls routing method. Initial parameter estimates for the reaches using the Muskingum routing method were estimated using channel properties.

## Model Calibration and Verification to Historic Flood Events

Kaskaskia model calibration includes adjusting parameters associated with the described modeling methods. The model was calibrated to multiple watershed conditions, wet/typical, wet/snowmelt, and dry. During model calibration, the modeler adjusted the following parameters based on the selected modeling methods.

- Initial and maximum deficit and constant loss rate for the deficit and constant rainfall loss method.
- Time of concentration and storage coefficient for the ModClark runoff transform.
- Initial discharge, recession constant, and ratio to peak parameters for the recession baseflow method.
- Number of subreaches for the Modified Puls and Muskingum channel routing methods, and the Muskingum K and X parameters.

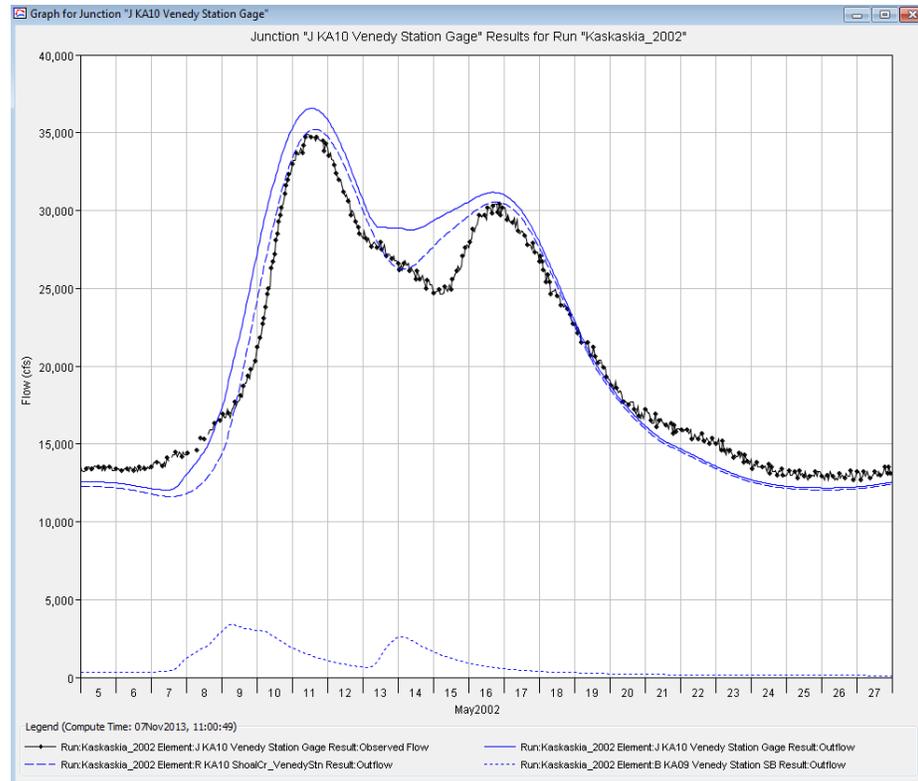
Model parameters were adjusted to improve the simulated flow as compared to observed flow hydrographs. (When located in a colder climate where snow accumulation and melt are important processes to include, the snow model parameters would need to be adjusted during model calibration to match historic estimates of snow water equivalent.)

In order to develop parameter sets for the wet/typical and dry watershed conditions, the modeler calibrated to events representing both conditions. The wet/typical condition represents average watershed conditions, while the dry condition represents the watershed with little or no precipitation for an extended amount of time. Two wet/typical events, one wet/snowmelt event, and one dry event were used to calibrate the model. Details of each calibration event are shown in [Table 38](#).

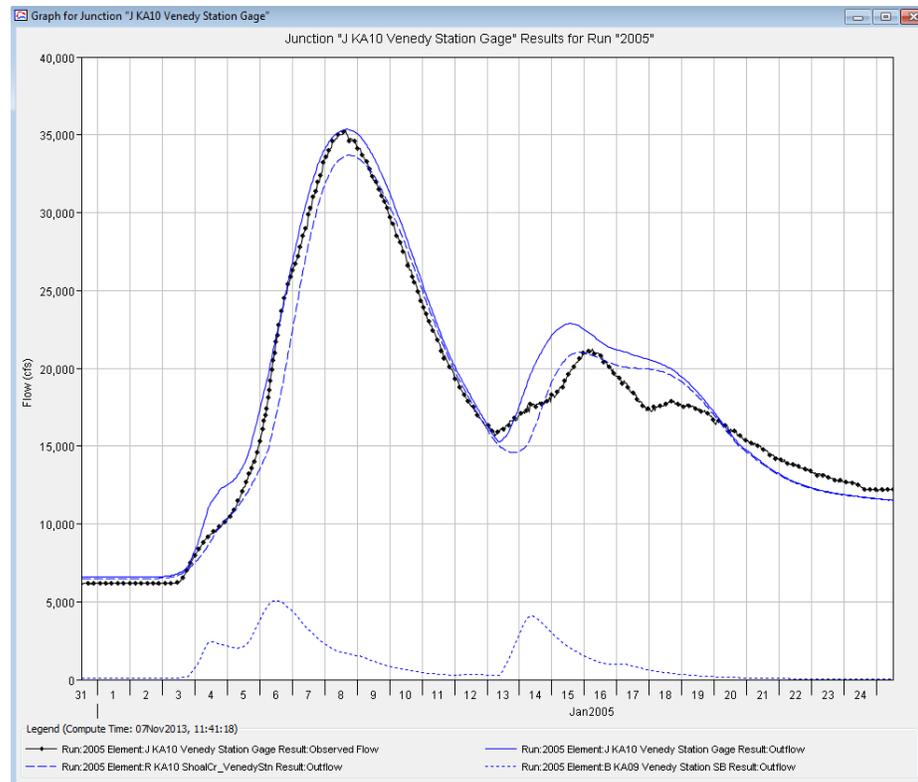
**Table 38.** Calibration Events for Various Watershed Conditions

Watershed Condition	Start Date	End Date	Description
Wet/Typical	05May2002	28May2002	Event of record for the Kaskaskia basin, rainfall totals of 6-10 inches in May 2002
Wet/Snowmelt	31Dec2004	24Jan2005	Snowfall in late December followed by a rain and ice storm in early January in combination with above normal temperatures
Wet/Typical	17Apr2011	10May2011	April 2011 set a record for average statewide precipitation with persistent rainfall continuing through May,
Dry	30Aug2012	13Sep2012	Significant drought in summer of 2012, heavy rains from Tropical Storm Isaac brought 6-8 inches of rain in 7 days

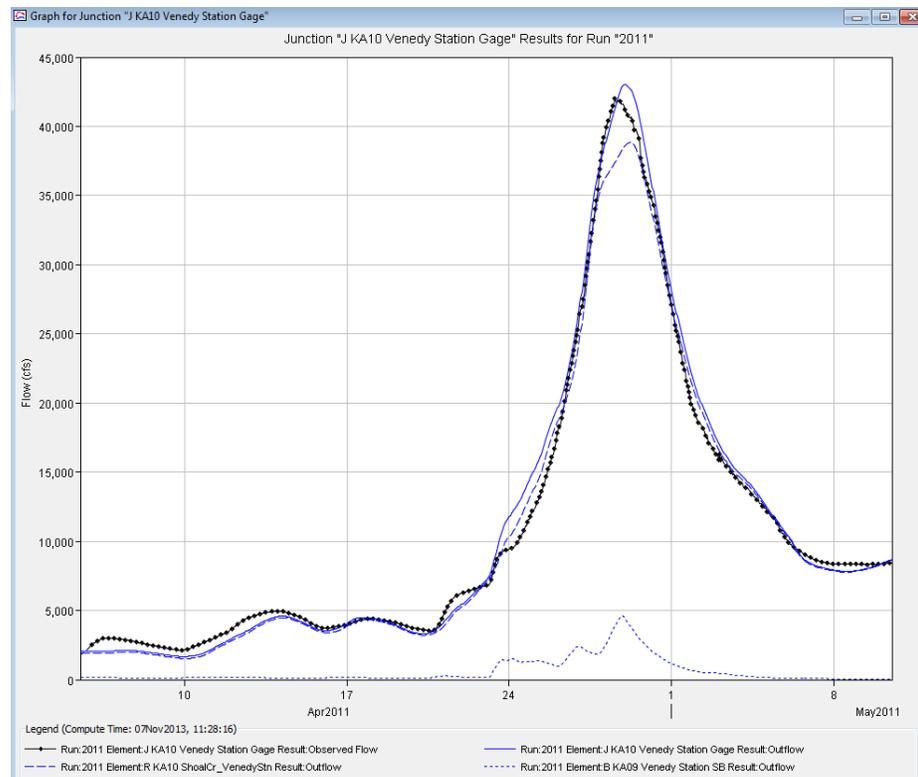
The modeler calibrated the Kaskaskia model to match peak flows, hydrograph shape, and hydrograph volume. Refer to EM 1110-2-1417 or a discussion on hydrologic model calibration (USACE, 1994). Simply stated, model calibration is the process of adjusting model parameters, within a reasonable range, in order for the model to reproduce observed information (observed flow, reservoir stage, and snow water equivalent are a few observed data sources used to calibrate precipitation-runoff model). For the Kaskaskia model, peak flow and hydrograph shape were given priority over total runoff volume. Figure 62 through Figure 65 show the computed versus observed hydrographs from one of the gage locations in the lower portion of the Kaskaskia watershed for each of the calibration events (the dotted black line references the observed flow hydrograph and the solid blue line references the total compute flow hydrograph at this location).



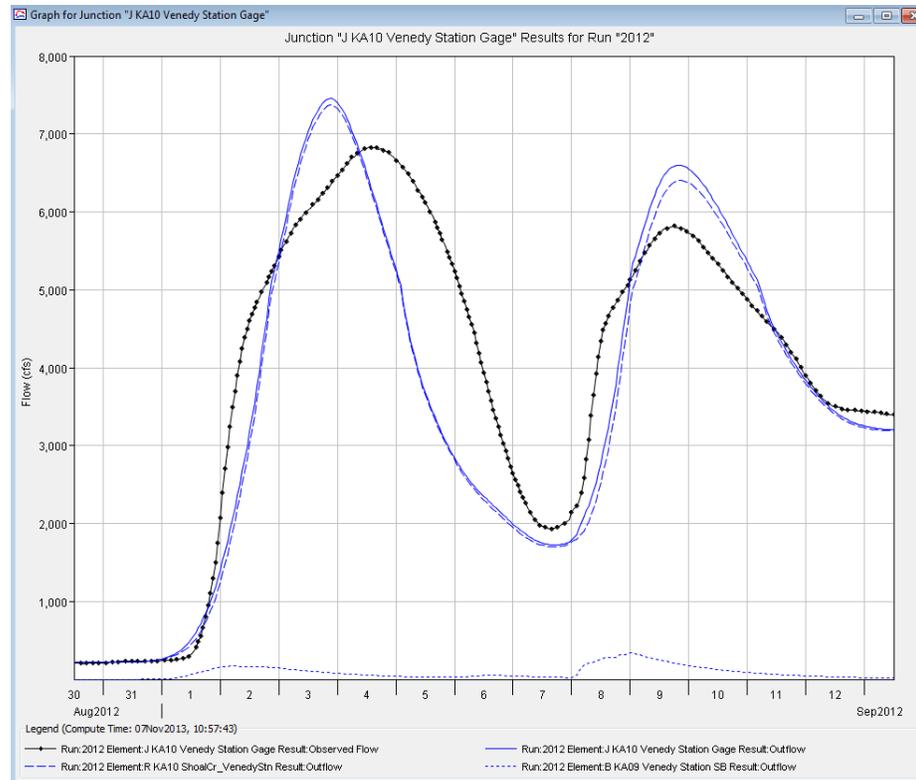
**Figure 62.** 2002 Wet/Typical Calibrated Hydrographs at the Venedy Gage



**Figure 63.** 2005 Wet/Snowmelt Calibrated Hydrographs at the Venedy Gage



**Figure 64.** 2011 Wet/Typical Calibrated Hydrographs at the Venedy Gage



**Figure 65.** 2012 Dry Calibrated Hydrographs at the Venedy gage

After calibrating the model to each event, the modeler determined final parameters for the wet/typical, wet/snowmelt, and dry watershed conditions. Final wet/typical parameters were averaged for the two chosen calibration events. Final wet/snowmelt and dry parameters were taken directly from the calibration events. Separate basin models were created for the three watershed conditions and populated with the final parameter values.

After model calibration, the next step is verifying the model. Final model parameters determined from the calibration simulations are applied to additional flood events. Parameters representing initial conditions can be adjusted for the validation events to correctly represent the state of the watershed at the beginning of the simulation.

## Application to Flood Forecasting

Flood forecasting involves configuring the forecast alternative and efficient real-time adjustments to the calibrated and validated model. Typical adjustments would include updating the soil moisture for current conditions, changes to infiltration rates, scaling unit hydrograph response times, modifying baseflow parameters, and scaling reach routing parameters. The process of flood forecasting in the Kaskaskia HEC-HMS model is described below. The steps included begin with the calibrated and validated HEC-HMS model. Specific information about completing

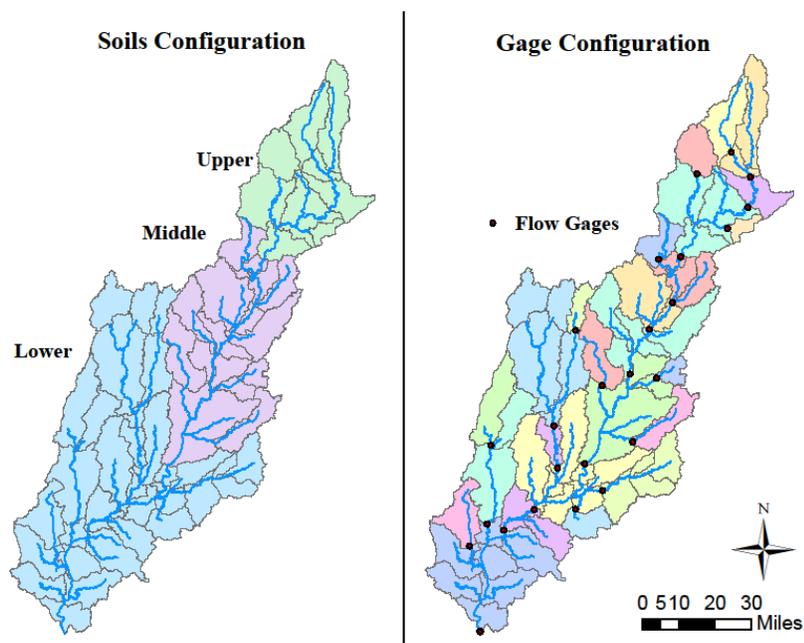
the flood forecasting steps in HEC-HMS is described in greater detail in Chapter 14 of the HEC-HMS User's Manual (HEC, 2016).

### *Preparing the Calibrated Model for a Forecasting Simulation*

Once the forecast alternative simulation is created, it must be configured in order to use the forecast alternative specific editors. As mentioned, the forecast alternative editors do not edit parameter values in the basin model. The base model (calibrated model) is used as a starting point for estimating runoff for a forecast simulation. Some adjustments will be needed to the base model due to current conditions in the watershed and possible errors in observed data.

The steps to configuring a forecast alternative in HEC-HMS are:

1. Set up zone configurations and elements within each zone for parameter adjustments. The forecaster chose to divide subbasin zones in the Kaskaskia basin into two configurations: zones by stream gage locations, referred to as the “gage” configuration, and zones by soil types, referred to as the “soils” configuration. The gage configuration allows the grouping of subbasins by location so the user can quickly compare results of parameter adjustments to observed flows. The gage configuration includes 25 zones representing the basin's 25 stream gages. The soils configuration is composed of 3 zones representing areas of different soil types. Both configurations are shown in [Figure 66](#).



**Figure 66.** Zone Configurations

Reaches can also be divided into zones for use in calibration. A reach zone configuration was not developed for the Kaskaskia

because the forecaster determined that the channel routing parameters would not be adjusted during real-time forecasting simulations.

2. Choose the relevant basin model and meteorologic model to be used in the forecast. The forecast alternative is constructed using the same initial conditions and parameters found in the selected basin model representing current watershed conditions. The forecaster chose the wet/typical watershed condition for this Kaskaskia forecast model as the conditions in the watershed were more aligned with the parameters in the wet/typical basin model.

The meteorologic model represents observed and forecasted future precipitation for each alternative. The first alternative forecasts zero future precipitation, and the second forecasts 5 inches of precipitation in the upper portion of the watershed at the time of forecast. Assumptions were made about the hourly distribution of the precipitation forecast. The precipitation was assumed to fall uniformly over the upper portion of the watershed.

3. Choose a time interval in which to compute the forecast. Based on available data and in order to capture the peak of the hydrograph, the forecaster chose a one-hour time step.
4. Select appropriate zonal configurations for loss rate parameters, transform parameters, and baseflow parameters. The forecaster selected the soils zone configuration for the loss parameters because loss parameters are usually directly linked to soil types. The gage configuration was selected for the transform and baseflow parameters to quickly scale unit hydrograph timing and recession baseflow to improve model performance at flow gage locations.

### ***Efficient Real-Time Adjustments to the Calibrated Model***

Forecast alternatives allow the user to develop various forecasts showing the results of different inputs (meteorologic boundary conditions) in the model. In this case study, the forecaster chose to analyze two different precipitation forecasts. Each forecast alternative was configured the same except for the meteorologic model.

5. Determine the time window for the forecast alternative. The forecaster chose a time of forecast date and time which was near the time of the last available observed data. The simulation start time was four days before the forecast start time, and the end time was selected as five days past the forecast start time, for a total simulation of nine days. The look back period was used to compare observed and computed results to help with calibration.

6. Set boundary conditions for a flood forecast. Boundary conditions acting on the Kaskaskia include precipitation and evapotranspiration, both accounted for in the meteorologic model. Observed flow is also a boundary condition because the forecaster used forecast blending.
7. Initialize conditions for a flood forecast, including baseflow and reservoir storage.

Initializing baseflow sets the flow condition for subbasins at the beginning of a forecast simulation. Observed flow provides a good source for calculating the baseflow initial condition. In the Forecast Initial Baseflow editor, the subbasins are divided by the zone configuration – for the Kaskaskia watershed, the gage zone configuration was selected. Figure 67 shows the Forecast Initial Baseflow editor for the Venedy Station zone. The forecaster selected an observed flow gage to determine flow at the start of the simulation. The ratio calculates the baseflow initial condition for each subbasin. This value was initially estimated as the ratio of each subbasin’s area to the total drainage area of the selected gage. The forecaster then adjusted the ratios until the observed and computed initial baseflow values were similar, as is shown in Figure 68 (Figure 68 shows only part of the results plot, the plot is zoomed into the look back period when observed flow data was available for real-time calibration). Minimum, maximum, and default values were not selected in this case because the gage data was scanned for missing or incorrect values before it was imported into the model. Developing appropriate ratios or ratio ranges can be completed when calibrating the model or prior to running the model in a true flood forecast mode. The ratio should be somewhat consistent; only minor adjustments might be needed from one simulation to the next.

Subbasins	Discharge Gage	Observed (CFS)	Minimum (CFS)	Maximum (CFS)	Default (CFS)	Ratio	Estimated Value (CFS)	Final Value (CFS)
B KA11 Posey SB	KA Venedy Station	378.0				0.018	6.8	6.8
B CR03 Lost Creek	KA Venedy Station	378.0				0.062	23.4	23.4
B CR01 Lower Crooked Creek	KA Venedy Station	378.0				0.048	18.1	18.1
B SH02 Beaver Creek	KA Venedy Station	378.0				0.097	36.7	36.7
B SH01 Lower Shoal Creek	KA Venedy Station	378.0				0.019	7.2	7.2
B KA10 Okawville	KA Venedy Station	378.0				0.058	21.9	21.9
B KA09 Venedy Station SB	KA Venedy Station	378.0				0.132	49.9	49.9

Figure 67. Forecast Initial Baseflow Editor for the Venedy Station Zone

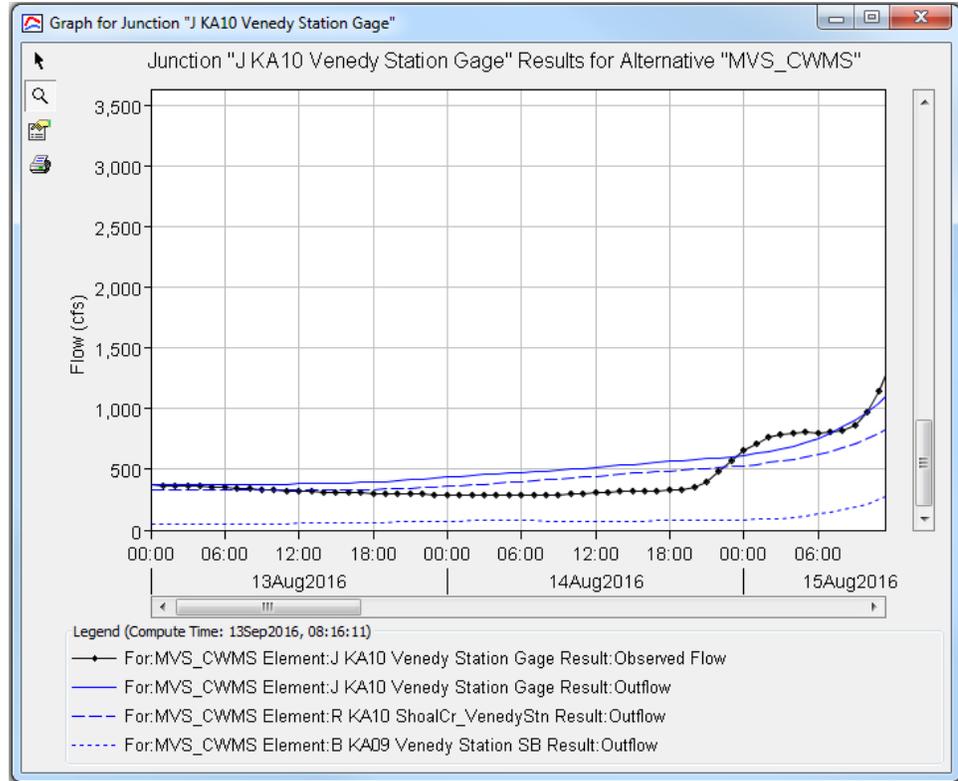


Figure 68. Initial Baseflow at the Venedy Station Gage

Reservoir initialization sets the initial pool storage based on elevation from observed stage gages and the input storage-elevation tables in the HEC-HMS model. In the Forecast Reservoirs editor, the forecaster selected the applicable stage gage for each of the reservoirs at the start time and the forecast time. Minimum, maximum, and default values were not selected in this case because the gage data was scanned for missing or incorrect values before it was imported into the model. The Forecast Reservoirs editor at the start time is shown in Figure 69. Figure 70 shows the pool elevation at Carlyle Lake at the start of the simulation.

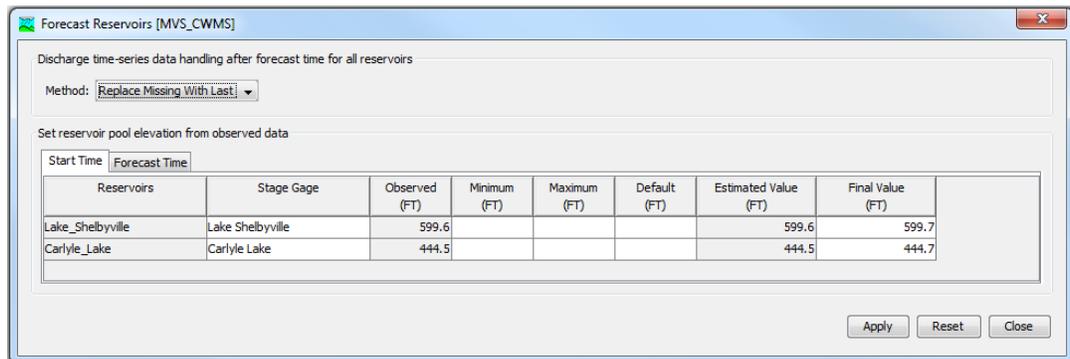
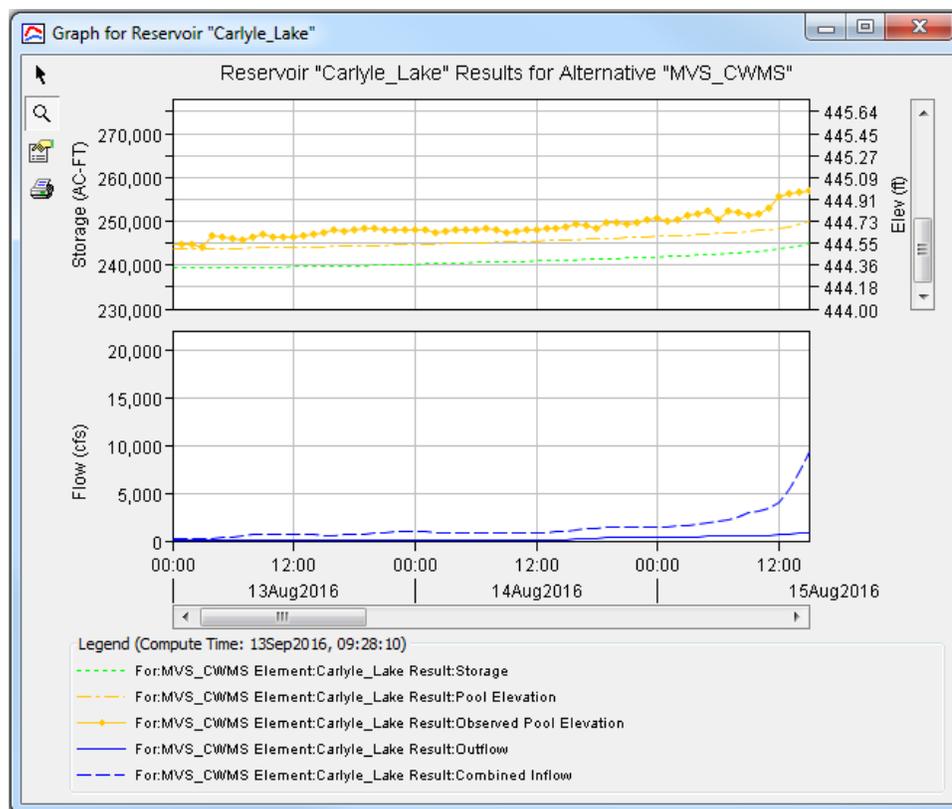


Figure 69. Forecast Reservoirs editor at the start time of the forecast



**Figure 70.** Carlyle Lake Reservoir Initialization

### ***Calibration and Application of the HEC-HMS Forecast Alternative***

While calibration of the HEC-HMS forecast alternative includes each zone designated in the forecast alternative, this description of real-time calibration focuses on the Venedy Station gage zone, middle soils zone, Venedy stream gage, and associated subbasin elements.

8. Calibrate the model efficiently. Parameter values for subbasins and reaches were initialized to default from the wet/typical basin model. When parameter values were adjusted in the forecast alternative, no adjustments were made to the original wet/typical parameter values contained in the basin model. The parameter adjustments described below were made in forecast alternative specific editors.
  - a. **Forecast Parameter Adjustments.** Parameters for loss, transform, and baseflow were edited in the Forecast Parameter Adjustments editors. [Figure 71](#) shows the deficit and constant loss forecast parameters editor. During model calibration, the forecaster multiplied the constant rate of each subbasin in the lower and middle zones by 2 in the top half of the editor. The bottom half of the editor is used for overriding zonal edits by editing individual element parameters.

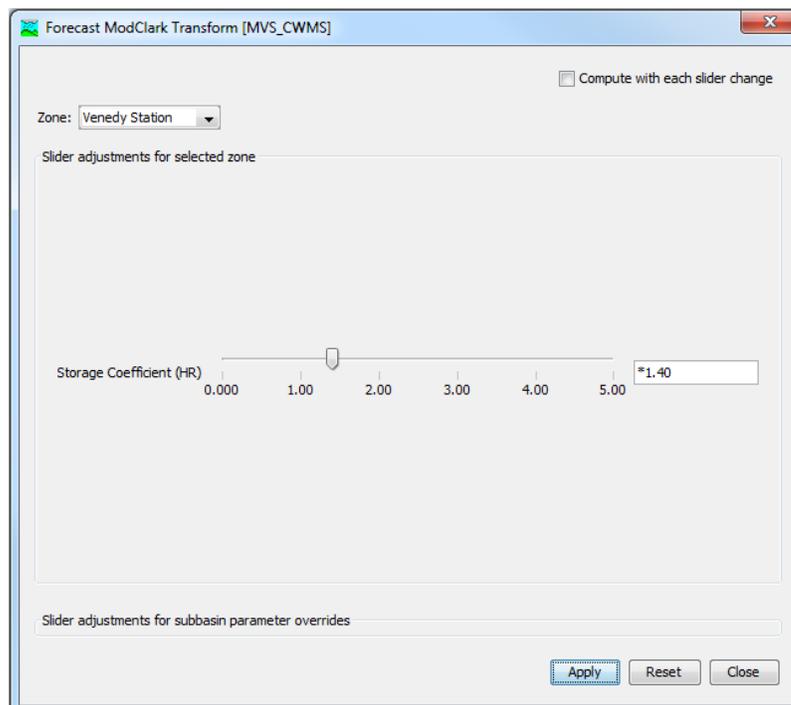
Zone	Initial Deficit (IN) Zone Average	Initial Deficit (IN) Adjustment	Maximum Deficit (IN) Zone Average	Maximum Deficit (IN) Adjustment	Constant Rate (IN/HR) Zone Average	Constant Rate (IN/HR) Adjustment
Lower	0.5		8.17		0.099	*2.0
Middle	0.5		8.36		0.097	*2.0
Upper	0.5		7.06		0.0947	

Override	Zone Subbasins	Initial Deficit (IN) Base Value	Initial Deficit (IN) Final Value	Maximum Deficit (IN) Base Value	Maximum Deficit (IN) Final Value	Constant Rate (IN/HR) Base Value	Constant Rate (IN/HR) Final Value
<input type="checkbox"/>	B LF01 Atwood SB	0.5	0.5	6.70	6.70	0.075	0.075
<input type="checkbox"/>	B KA27 Upper Kaskaskia	0.5	0.5	7.31	7.31	0.1	0.1
<input type="checkbox"/>	B KA26 Chesterville SB	0.5	0.5	7.61	7.61	0.0625	0.0625
<input type="checkbox"/>	B KA25 Cooks Mill SB	0.5	0.5	7.97	7.97	0.1	0.1
<input type="checkbox"/>	B WO02 Lovington SB	0.5	0.5	6.86	6.86	0.1	0.1
<input type="checkbox"/>	B WH01 Allenville SB	0.5	0.5	6.50	6.50	0.1	0.1
<input type="checkbox"/>	B WO01 Lower West Okaw	0.5	0.5	7.24	7.24	0.1	0.1
<input type="checkbox"/>	B KA24 Sullivan	0.5	0.5	6.73	6.73	0.1	0.1
<input type="checkbox"/>	B KA23 Lake Shelbyville	0.5	0.5	6.51	6.51	0.1	0.1

**Figure 71.** Forecast Parameter Adjustments Editor for Deficit and Constant Loss Parameters

- b. **Slider Adjustments.** Instead of using the Forecast Parameter Adjustments editor, the forecaster used the Slider Adjustment tools to adjust Modified Clark transform parameters. The slider adjustment tool for the Venedy Station zone is shown in Figure 72. The forecaster chose not to compute with each slider change, and instead manually computed the simulation (by clicking the compute button) after making each change.



**Figure 72.** Forecast ModClark Transform Slider Bar Adjustment Editor

- c. **Forecast Blending.** Forecast blending provides a method for systematically using observed flow to adjust computed streamflow. The forecaster selected the taper method for forecast blending and a taper duration of 72 hours.
- d. **Computing to a Computation Point.** The Venedy station gage was designated as a computation point. Therefore, after making parameter adjustments to the Venedy Station zone, the forecaster selected to compute to just the Venedy Station computation point to speed calibration. Only model elements upstream of the Venedy Station gage were computed when the compute to point option was used.
- e. **Model Computational Efficiency.** When parameters of the Venedy Station zone were adjusted and the model was recomputed, only components with data changes were recomputed. This feature provides another way to allow for efficient calibration of the forecast alternative.

### ***Viewing and Analyzing Results***

- 9. View and analyze results, and extract results for further analysis. [Figure 73](#) shows the results of the first forecast alternative in which no precipitation was forecasted. [Figure 74](#) shows the results of the second alternative forecasting five inches of rain in the upper portion of the watershed at the time of forecast. The forecaster easily and efficiently compared the results of the two alternatives noting how the change in precipitation forecast affected the runoff response from the watershed. These results could then be used in reservoir flood operations or imported into additional model software like HEC-ResSim or HEC-RAS for reservoir operations or inundation mapping.

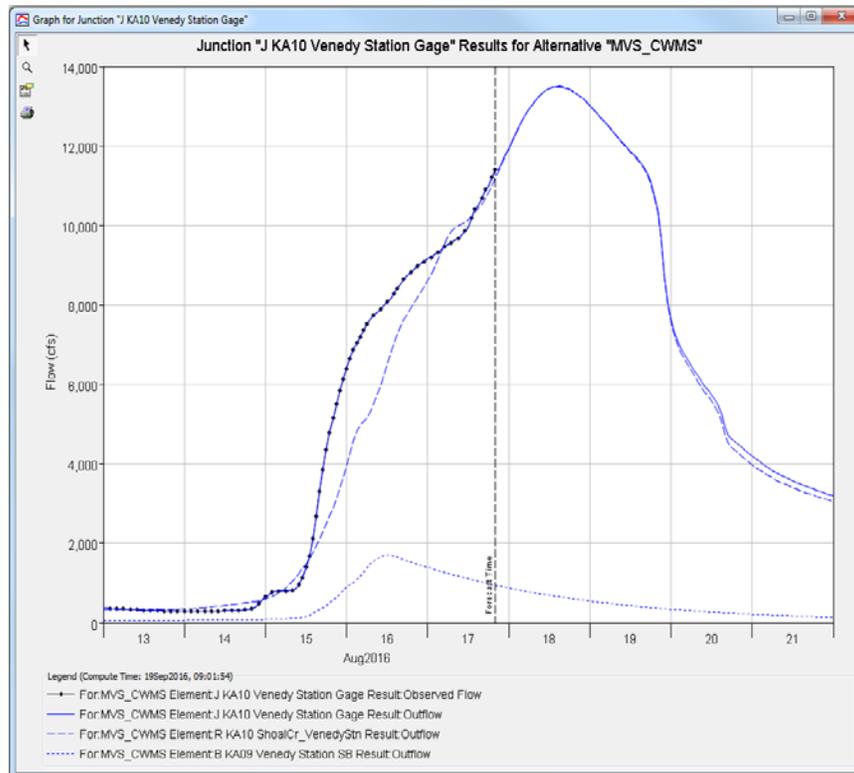


Figure 73. Venedy Station Gage Results Assuming no Forecasted Precipitation

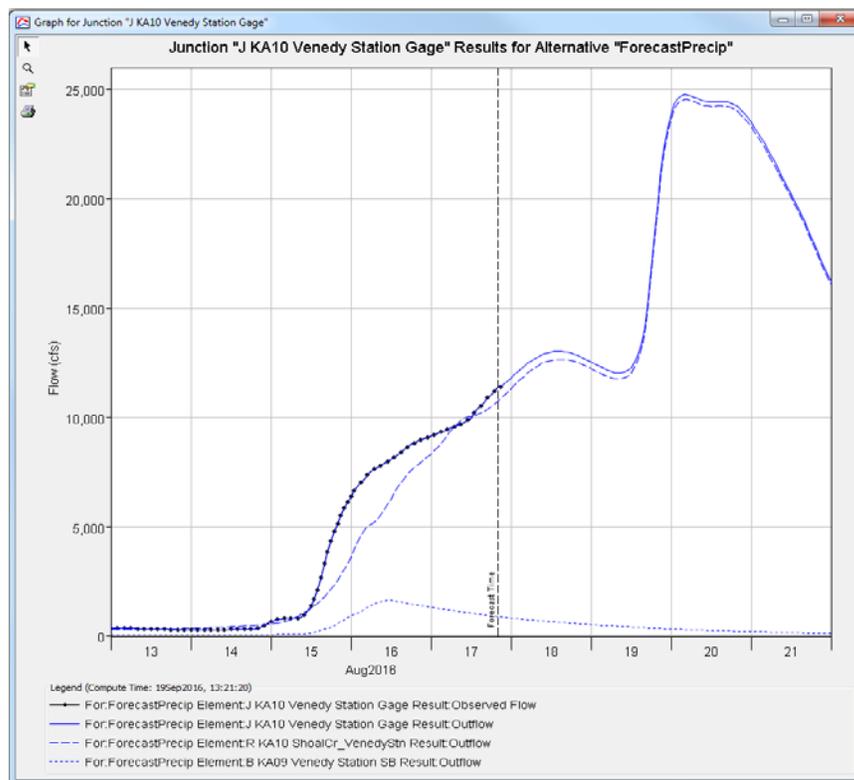


Figure 74. Venedy Station Gage Results Assuming Forecasted Precipitation

## Summary

The goal of this study was to develop an HEC-HMS model of the Kaskaskia basin and create forecast alternatives to view and analyze the results of two precipitation forecasts. Model development included developing a basin model of the Kaskaskia River watershed. Multiple parameter sets were developed to represent both a wet/typical and a dry condition. Both watershed conditions were calibrated to historic events to develop final parameters.

Forecast alternatives allowed the forecaster to analyze two precipitation forecasts and quickly and easily compare the results. The forecaster developed two zone configurations; the gage configuration was assigned to transform and baseflow parameters, and the soils configuration was assigned to loss parameters. The forecaster initialized baseflow and reservoir elevations using observed stream gage and stage gage data. Loss and transform parameters were efficiently calibrated using the Forecast Parameter Adjustment editor and the Slider Bar editor. Computed flows were compared to observed flows by systematically calibrating the model from upstream to downstream utilizing the compute to point compute option. The built-in model computational efficiency measures in HEC-HMS limited the simulation run time by only computing results for those elements that required results to be updated. Forecast blending was used to replace computed flows in the look back period with observed flows. The results of each forecast alternative were quickly compared and were ready for use as inflows for reservoir operations and inundation mapping.

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