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Hydrologic Modeling System
HEC-HMS

Applications Guide

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14. ABSTRACT
The document illustrates application of program HEC-HMS to typical studies of those undertaken by Corps' offices, including (1) urban flooding studies; (2) flood-frequency studies; (3) flood-loss reduction studies; (4) flood-warning system planning studies; (5) reservoir design studies; and (6) environmental studies. For each study category, this document identifies common objectives of the study and the authority under which the study would be undertaken. It then identifies the hydrologic engineering information that is required for decision making and the methods that are available in HEC-HMS for developing the information. The manual illustrates how the methods could be configured, including how boundary conditions would be selected and configure. For each category of study, this guide presents and example, using HEC-HMS to develop the required information.

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Hydrologic Modeling System HEC-HMS

Applications Guide

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Hydrologic Modeling System HEC-HMS, Applications Guide

PREFACE

This manual illustrates application of computer program HEC-HMS in studies typical of those undertaken by hydrologic engineers of the U.S. Army Corps of Engineers, including (1) urban flooding studies; (2) flood-frequency studies; (3) flood-loss reduction studies; (4) flood-warning system planning studies; (5) reservoir design studies; and (6) environmental studies.

HEC-HMS is the Corps' next generation computer program for watershed modeling. It is a product of the Corps' Civil Works hydrologic engineering research and development program. The program was developed by the staff of the Hydrologic Engineering Center (HEC), Davis, CA, and by consultants to HEC. Arlen Feldman managed HEC-HMS development, and Darryl Davis was Director of HEC during program development.

David Ford Consulting Engineers prepared this manual. William Scharffenberg of HEC managed preparation.

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.

This manual was prepared using version 2.1.2 of computer program HEC-HMS.

EXECUTIVE SUMMARY

Hydrologic engineers in Corps of Engineers' offices nationwide support Corps' 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, of peak flow rates, and of timing of flows. 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 Corps' offices, including:

- Urban flooding studies.

- Flood-frequency studies.

- Flood-loss reduction studies.

- Flood-warning system planning studies.

- Reservoir design studies.

- Environmental studies.

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.

C H A P T E R   1

Introduction

The mission of the Corps of Engineers 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 the Corps undertake that require watershed and channel information?

Study classification

Hydrologic engineers in the U.S. Army Corps of Engineers 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**. The Corps 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, Corps 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 the Corps' 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 Corps' 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 the Corps 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, *Hydrologic engineering studies design*, 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.

Table 1.   Description of project phases

Project phases	Description
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). 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 Corp's approach to flood studies is to follow a process that involves planning, design, construction, and operation. The sequential phases are described in Table 1. An initial HEMP is prepared at the end of the reconnaissance phase; this defines

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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. 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) 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.

Category	Method
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 U.S.
	Soil Conservation Service (SCS) hypothetical storm
Runoff-volume	Initial and constant-rate
	SCS curve number (CN)
	Gridded SCS CN
	Green and Ampt
	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
Routing	Kinematic wave
	Lag
	Modified Puls
	Muskingum
	Muskingum-Cunge standard section
	Muskingum-Cunge 8-point section
	Confluence
	Bifurcation
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* (USACE, 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* (USACE, 2001) provides instructions for executing 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 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 subwatersheds that are hundreds of square miles, by computing runoff from the individual subwatersheds, and by combining the resulting hydrographs. A time step of 6 hours might be appropriate for such an application. However, the methods in HEC-HMS can also be used to compute runoff from a 2 or 3 square mile urban watershed, using a 5-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 and Freeze (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 Corps' 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

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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 program user's manual.

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 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 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 ungaged 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.

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,

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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, the Hydrologic Engineering Center has developed a suite of other programs that provide additional capabilities, such as those listed in Table 4. These programs are integrated through databases or interfaces with HEC-HMS. For example, a discharge hydrograph computed with program HEC-HMS can be used directly as the upstream boundary condition for HEC-RAS or as the reservoir inflow boundary condition for HEC-ResSim. Similarly, a discharge-frequency function computed with HEC-HMS (as illustrated in Chapter 3 of this report) can be typed in the HEC-FDA interface and used subsequently to compute EAD.

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.

Table 4.   Other HEC programs that are integrated with HEC-HMS

Program Name	Description of Capabilities	Reference
HEC-RAS	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.	USACE (2002)
HEC-FDA	Computes expected annual damage (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 (1998)
HEC-FIA	Computes post flood urban and agricultural flood damage, based upon continuous evaluation with flow or stage time series.	
HEC-ResSim	Simulates reservoir system operation, given description of reservoirs and interconnecting channels, reservoir inflow and local flow hydrographs, and reservoir operation rules.	

References

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 (1994a). *Hydrologic analysis of watershed runoff, ER 1110-2-1464*. Office of Chief of Engineers, Washington, DC.

USACE (1994b) *Hydrologic engineering studies design, EP 1110-2-9*. Office of Chief of Engineers, Washington, DC.

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

USACE (2000). *HEC-HMS technical reference manual.* Hydrologic Engineering Center, Davis, CA.

USACE (2001). *HEC-HMS hydrologic modeling system user's manual*. Hydrologic Engineering Center, Davis, CA.

USACE (2002). *HEC-RAS User's manual*. Hydrologic Engineering Center, Davis, CA.

C H A P T E R   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

Corps of Engineers 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 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 the Corps to take action to reduce the hazards and risk associated with floods.

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

The following Corps guidance on urban flooding studies includes:

- ER 1105-2-100 *Planning guidance notebook*. This provides guidance and describes procedures for all civil works planning studies.

- ER 1165-2-21 *Flood damage reduction measures in urban areas*. This defines the Corps involvement in urban flood studies. A Federal interest exists for the

portion of the watershed where the channel flow exceeds 800 cfs for the 10 percent chance flood (0.10 annual exceedance probability). 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 *Hydrologic analysis of interior areas.* This describes general considerations when evaluating interior areas, commonly found in urban watersheds protected by levees from large bodies of water.

- EM 1110-2-1417 *Flood-runoff analysis.* This describes methods, procedures, and general guidance for hydrologic analysis including rainfall, snowmelt, infiltration, transformation, baseflow, and stream routing.

- EP 1110-2-9 *Hydrologic engineering study design*. This describes the components needed to develop the hydrologic engineering management plan (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 include 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.

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

Strong Ranch Slough and Sierra Branch portion of the watershed is 7.1 square miles and the Chicken Ranch Slough 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.
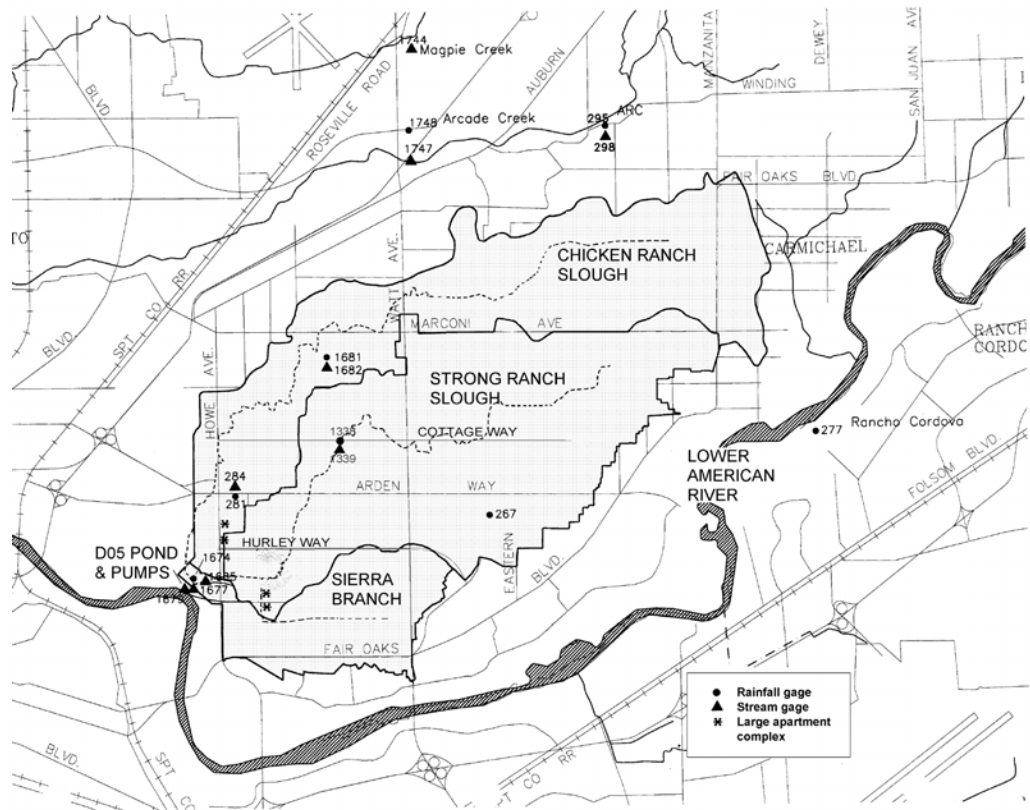
Figure 1.   Chicken Ranch Slough and Strong Ranch Slough watershed

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-feet 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 15 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 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, the Corps has now moved on to the feasibility phase. In this phase, the Corps has been tasked with answering the questions:

- Will the development of the open area increase the peak runoff in the Chicken Ranch Slough watershed for the 0.01 annual exceedance probability (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, the Corps will use a watershed model to compute the peak flow for the different watershed conditions. Computer program 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 Chicken Ranch Slough (CRS) only. So, the portion of the watershed that contributes flow to Strong Ranch Slough 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 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 (sq mile)	S (ft/mile)	L (mile)	L_c (mile)
CRS u/s of Corabel gage	COR	4.22	15.84	4.43	2.24
CRS d/s of Corabel gage, u/s of Fulton	FUL	0.30	12.67	0.61	0.07
CRS d/s of Fulton, u/s 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 8 lists the estimates of percent of directly connected impervious area for CRS watershed.

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.

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

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:

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

in which 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; and n = basin n coefficient. The basin n coefficient is a function of the percent imperviousness and the land use of the watershed. Table 7 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 dimensionless unit hydrograph for urban basins (Cudwoth, 1989). Using Equation 1, the lag was estimated; values are shown in Table 8.

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Table 7.   Basin n values for Equation 1

Basin land use	Percent directly Impervious	Channelization description	
		Developed pile or channel	Undeveloped natural
Commercial, offices	90	0.031	0.070
Residential: 4-6 du/acre	40	0.042	0.084
Residential: 3-4 du/acre	30	0.046	0.088

Table 8.   Unit hydrograph lag and percent impervious estimates

Description	Identifier	Estimated lag (hr)	Percent directly impervious
CRS u/s of Corabel gage	COR	1.78	50
CRS d/s of Corabel gage, u/s of Fulton	FUL	0.22	60
CRS d/s of Fulton, u/s of Arden Way	ARD	0.75	50

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 8-point 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 model 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 1-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 5 to 10 points on the rising limb of the unit hydrograph for each subbasin is usually adequate. Using the approximate relationship that the lag time equals 60% of the time of concentration from EM 1110-2-1417 (USACE,

1994), 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 10 points. This yielded a minimum approximate time step of 2 minutes, as follows:

(0.22 hrs x 60 min/hr) / (0.6 x 10 points on rising limb) = 2 min time step   (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 U.S. are greater than 12 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 is takes to satisfy the initial loss and the time of concentration. A general estimate for this time is 4 times the time of concentration of the watershed. Using this estimate yielded a 12-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.
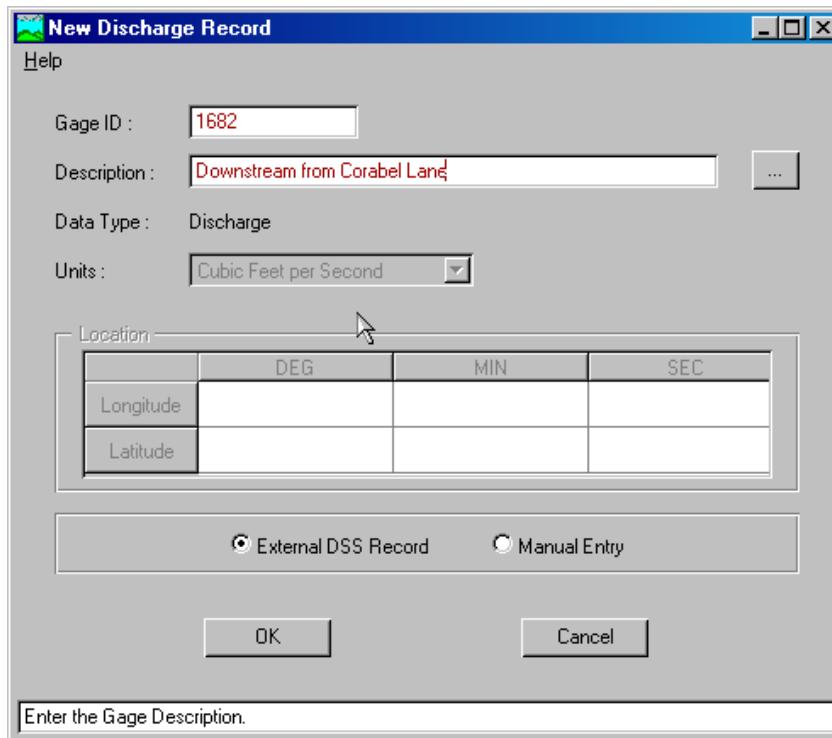
The lag time and percent impervious were estimated as described above. The initial loss, constant loss rate, and peaking coefficient will be estimated using calibration. The initial estimate for the constant loss rate is based upon regional relationships. It is 0.07 in/hour. Because the watershed is 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 *Technical reference manual* (USACE, 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 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 basin to select the appropriate methods for **Loss Rate**, **Transform**, and **Baseflow Method**, and entered the initial estimates for each method.

3. Added the observed flow as a gage by selecting **Data**, **Discharge Gages.** From the **Discharge Gage Manager,** the analyst then selected **Edit**, **Add Gage**. In this case, the observed values were in HEC-DSS format, so **External DSS Record** was selected, as shown in Figure 2, and the HEC-DSS pathname was entered on the following screen. Additional instructions on adding a gage are included in the HEC-HMS *User's manual* (USACE, 2001).

Figure 2.   Creating a discharge gage with historical data

4. Associated the observed flow gage with the subbasin by right-clicking on the **Subbasin** and selecting **Observed Flow**. The appropriate gage was then selected. This is shown in Figure 3.
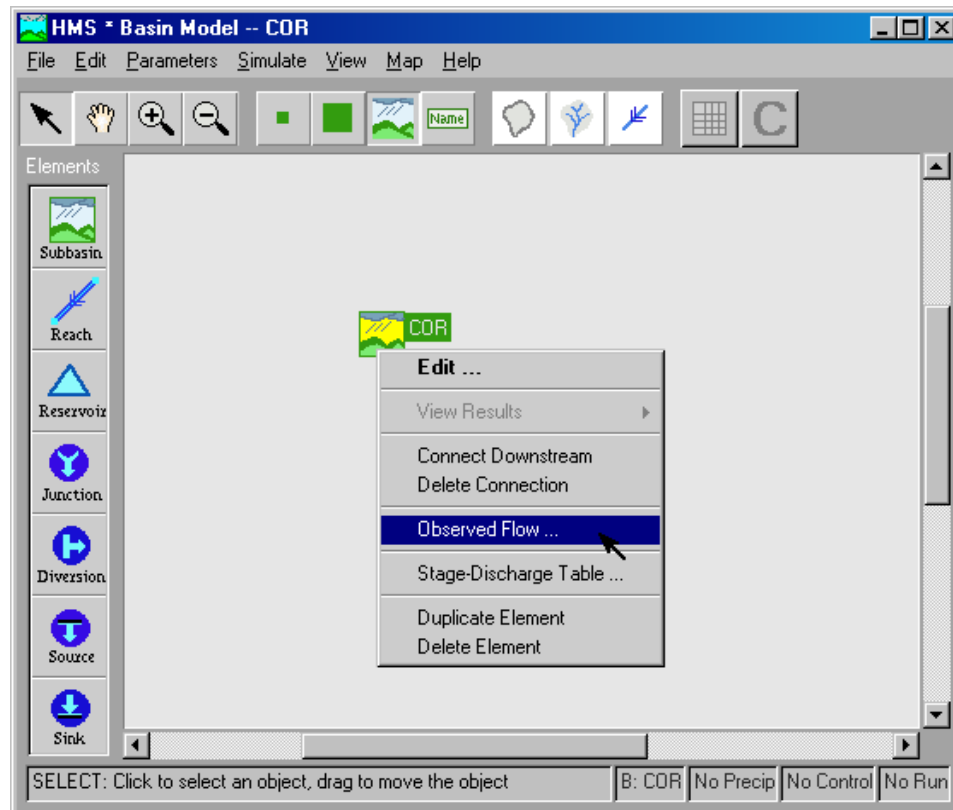
Figure 3.   Adding observed flow for calibration

5. Created a new **Meteorologic Model** for the historical event. To do so, gaged precipitation data were entered through the **Data** menu. 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 *Technical reference manual* for instructions on creating a **Meteorologic Model**.

6. 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 *User's manual* for instructions on creating control specifications.

7. Created a new **Optimization Run** by selecting **Tools**, **Optimization Run Configuration** from the project window.

8. Navigated to the **Optimization Manager** by selecting **Tools, Optimization Manager** from the project window. Parameter values to be included in the January 10, 1995 calibration event were identified, as shown in Figure 4. This was accomplished by selecting the appropriate **Hydrologic Element** and **Parameter** using the drop-down lists. Additional instructions are included in the *User's manual*.
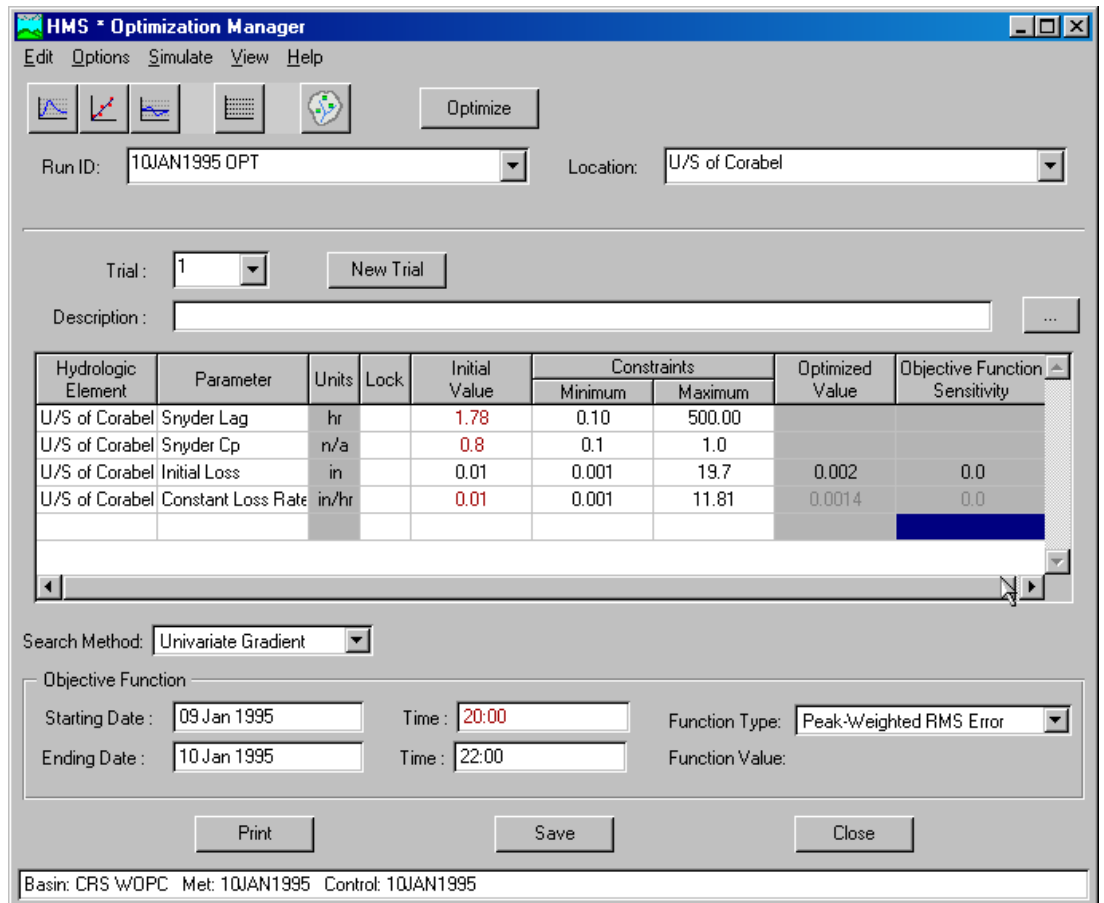
Figure 4.   Calibration results for the January 10, 1995 event

9. In the **Optimization Manager**, the analyst set the time window to correspond with the rising and falling limb of the primary flow peak of the event. This allowed the program to calibrate to the peak events and focus the optimization function on matching the peak flow.

10. Once the initial parameter values were specified, clicked the **Optimize** button to begin the computations.

11. Studied the plots of the results, revised initial estimates as needed, and repeated step 10. 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 10% less than the observed peak flow.

This process was repeated for the January 22, 1997 event. The calibration results for the January 22, 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.
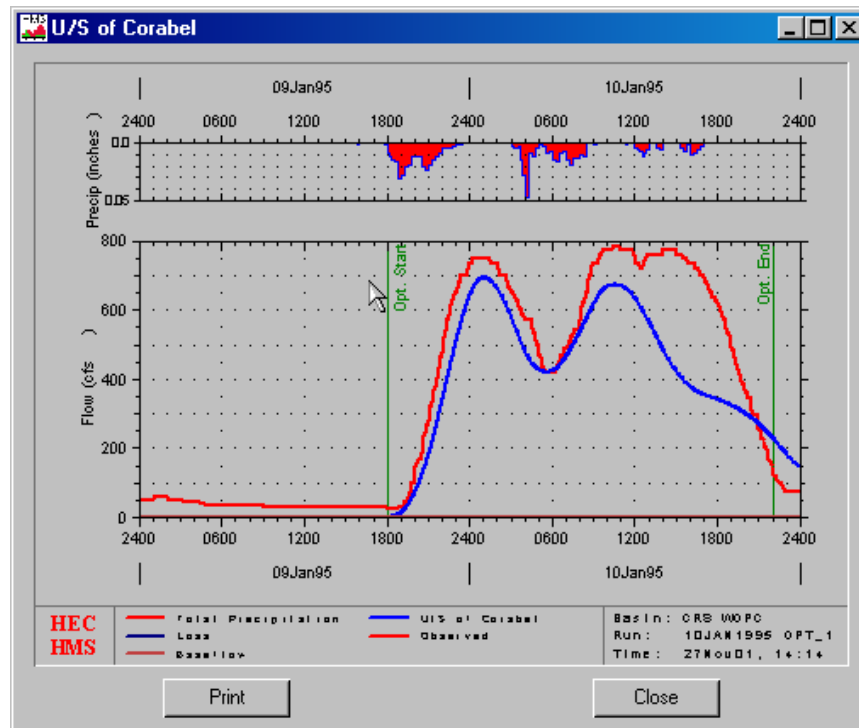
Figure 5.   Calibration results for the January 10, 1995 event
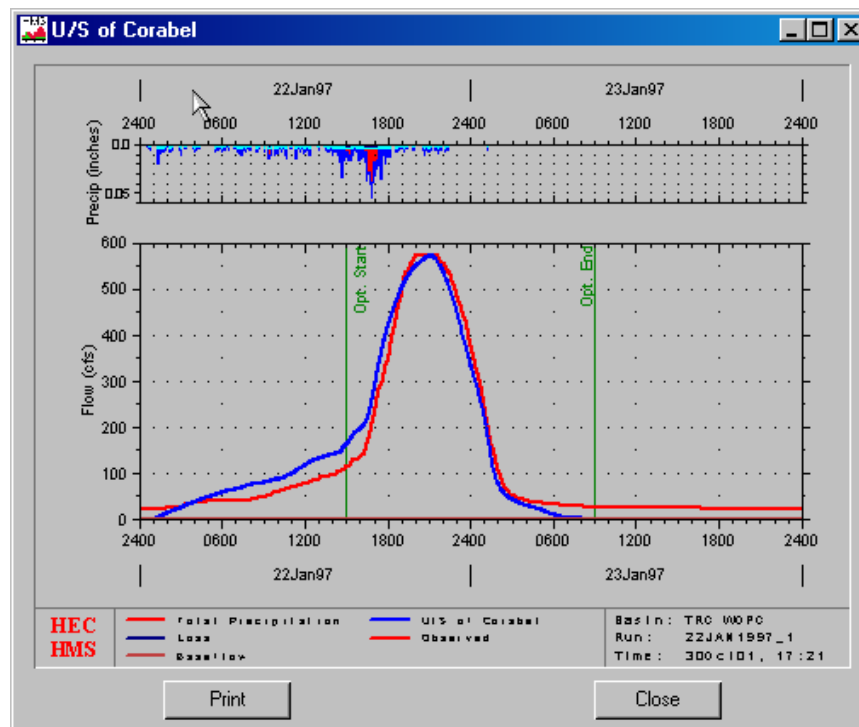
Figure 6.   Calibration results for the January 22, 1997 event

The parameter estimates resulting from the calibration to the January 10, 1995 event and the January 22, 1997 event are summarized in Table 9. The values were averaged and verified using the observed precipitation and flow data for the January

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26, 1997 event. To do so, the average lag and peaking value were input to the **Basin Model** and the **Optimization Manager** was exercised again. This time, only the loss rate parameters were adjusted as specified in step 8 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 too great. The lag value was added to the **Parameter** list in the **Optimization Manager** for the January 26, 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 January 22, 1997 event.
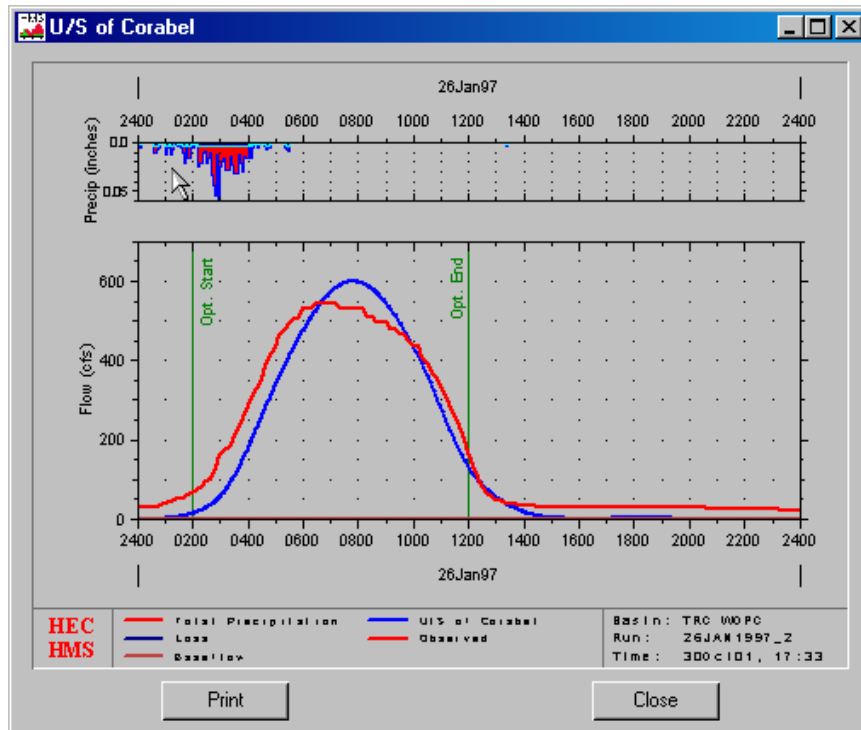
Figure 7.   *Verification of estimated parameters with January 26, 1997 event*

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.

Table 9.   *Parameter estimates from calibration for the headwater basin*

Calibration event	Snyder lag (hr)	Snyder C_p
January 10, 1995	5.42	0.68
January 22, 1997	4.68	1.00
January 26, 1997	4.55	1.00
Final average value	**4.88**	**0.89**

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 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)	Adjusted Snyder 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 8-point 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 Ave to Arden Way.

Figure 8.   Routing reach parameters

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 annual exceedance probability (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.

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.

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.

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 *User's manual*. New **Meteorologic Models** and **Control Specifications** were added.

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 exercised the model and calculated 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 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

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
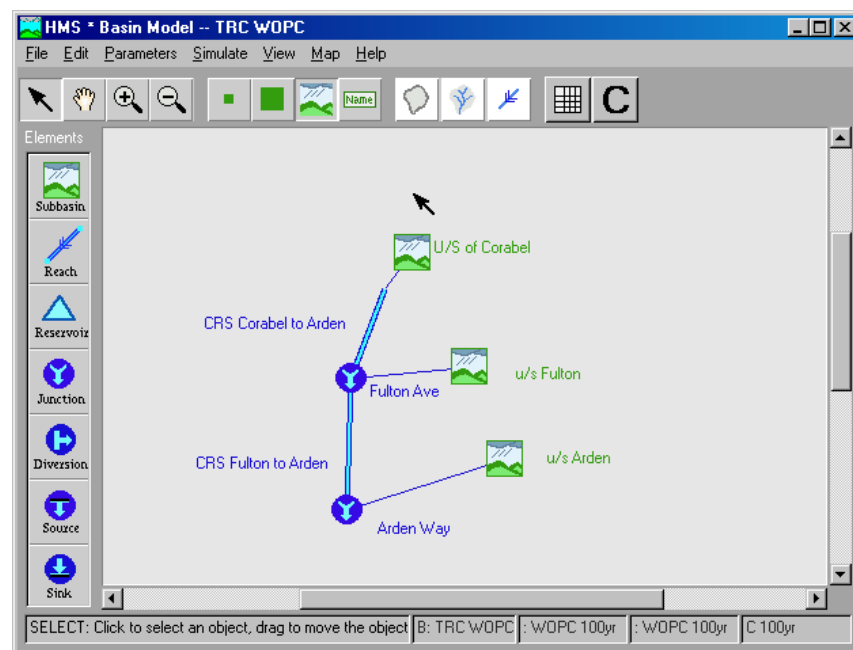
Figure 9.   CRS basin schematic

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Figure 10.   Meteorologic Model for the 0.01-AEP event

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%.

Intuitively, the analyst expected that the unit hydrograph lag would decrease and the peaking coefficient, C_p, would increase. Using relationships from the Denver lag equation (EMSI), an increase from 50% to 55% impervious area would increase the C_p value by 8%. This results in a modified value of 0.96 for the COR subbasin. Using Equation 1 (the regional lag equation), an increase of 5% of impervious area decreases the lag by 4%. 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 Snyder's 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.

Table 13.   *HEC-HMS results at Arden Way for 0.01 AEP event*

Condition	Peak flow (cfs)	Runoff volume (ac-ft)
Existing (without development)	941	794.2
Future (with development)	1,003	828.8
Percent increase	**6.6%**	**4.4%**

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 U.S. For example, equations related to the state of Washington are published in *Magnitude and frequency of floods in Washington* (Sumioka, Kresch, and Kasnick, 1998).

The regression equations for California are published in *Magnitude and frequency of floods in California* (Waananen and Crippen, 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:

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

where $Q_{1\%}$ = the flow for the 0.01 AEP event, in cfs; A = the drainage area, in square miles; P = the mean annual precipitation, in inches; and H = the average main channel elevation at 10 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 7 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 (1977), Sauer, *et al.* (1983), and Jennings, *et al.* (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 18 inches, the analyst computed a 0.01 AEP peak flow of 1,113 cfs. If the urbanization factor by Rantz 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 by Thomas, *et al.* (2001) in *Evaluation of flood frequency estimates for ungaged* watersheds. Using the functions developed by Rantz, an increase in developed watershed of 5% will reasonably increase the peak runoff of the 0.01 AEP event by 6%.

Processing of results

To determine how significant this increase in flow is to the peak stage in Chicken Ranch Slough, 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 Chicken Ranch Slough 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.

References

Barnes, H.H. (1967). *Roughness characteristics of natural channels. Water-supply paper 1849.* U.S. Geological Survey, Washington, D.C.

Cudworth, A.G., Jr. (1989). *Flood Hydrology Manual.* U.S. Department of the Interior, Bureau of Reclamation, Denver, CO.

Environmental Modeling Systems, Incorporated. (http://www.ems-i.com/wmshelp/WMSHELP.htm) *WMS help, Denver tag time equation*. San Jordan, UT.

Jennings, M.E., W.O. Thomas, Jr. and H.C. Riggs (1994). *Nationwide summary of U.S. Geological Survey regional regression equations for estimating magnitude and frequency of floods for ungaged sites.* Water-resources Investigation 94-4002. U.S. Geological Survey, Washington, DC.

National Weather Service (1972). *National Weather Service river forecast system river forecast procedures, NOAA Technical Memo NWS-HYDRO-14.* Hydrologic Research Laboratory, Silver Spring, MD.

Rantz, S. E. (1971). *Suggested criteria for hydrologic design of storm-drainage facilities in the San Francisco Bay region, California*. Open file report, U.S. Geological Survey, Menlo Park, CA.

Sauer, V.B., W.O. Thomas, V.B. Stricker, and K.V. Wilson (1983). *Flood characteristics of urban watersheds in the United States.* Water-supply Paper 2207. U.S. Geological Survey, Washington, DC.

Sumioka, S.S., D.L. Kresch, and K.D. Kasnick (1998). *Magnitude and Frequency of Floods in Washington.* Water-resources Investigation 97-4277. U.S. Geological Survey, Washington, DC.

Thomas, W.O., Baker, M., Grimm, M.M., and McCuen, R.H. (2001). *Evaluation of flood frequency estimates for ungaged watersheds.* Unpublished manuscript.

U.S. Army Corps of Engineers [USACE] (1980). *Flood damage reduction measures in urban areas, ER 1165-2-21*. Office of Chief of Engineers, Washington, DC.

USACE (1994). *Flood-runoff analysis, EM 1110-2-1417.* Office of Chief of Engineers, Washington, DC.

USACE (1987). *Hydrologic analysis of interior areas*, *EM 1110-2-1413*. Office of Chief of Engineers, Washington, DC.

USACE (2000). *HEC-HMS technical reference manual.* Hydrologic Engineering Center, Davis, CA.

USACE (2001). *HEC-HMS hydrologic modeling system user's manual*. Hydrologic Engineering Center, Davis, CA.

Waananen, A.O. and J.R. Crippen (1977) *Magnitude and frequency of floods in California.* Water-resources Investigation 77-21. U.S. Geological Survey, Washington, DC.

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

Corps flood frequency studies are authorized generally 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.

- 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 the Corps to take action to reduce the hazards and risk associated with floods.

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

- ER 1110-2-1450 *Hydrologic frequency estimates*. This describes the scope and general requirements for flood frequency studies.

- EM 1110-2-1415 *Hydrologic frequency analysis.* This describes the procedure and computational guidelines for flood frequency studies. The procedures generally follow *Bulletin 17B* (Interagency advisory committee on water data, 1982) recommendations.

- EM 1110-2-1417 *Flood-runoff analysis.* This 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 and EM 1110-2-1415 present procedures for such analysis, and the HEC-FFA computer program implements those procedures.

The USGS (Sauer, *et al.*, 1983 and Jennings, *et al.,* 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 and Cordery (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 depth-duration-frequency (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 the National Weather Service or by a local government agency. For example, *NOAA Atlas 2* presents isohyetal maps for 6-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 U.S. (Miller, *et al*., 1973). This document also presents methods for deriving depths for other durations. For the central and eastern U.S., National Weather Service *TP-40* (Herschfield, 1961), *TP-49* (Miller, 1964), and *HYDRO-35* (Fredrick, *et al*., 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 the Corps study team used HEC-HMS to evaluate the impact of development of 320 acres of open land in the Chicken Ranch Slough and Strong Ranch Slough (CRS/SRS) watershed. The CRS/SRS watershed is an urban watershed of approximately 15 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% (from 941 to 1,003 cfs) for the 0.01 annual exceedance probability (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, the Corps' analyst has been tasked with answering the question: What is the increase in flood damage in Chicken Ranch Slough 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 direct-runoff transform method.

- Baseflow: None used.

- Routing: Muskingum-Cunge 8-point channel routing method.

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

A 2-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 annual exceedance probability (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 7 other events; Table 15 shows these. HEC-HMS has 8 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 8 of the precipitation frequencies listed were used as a boundary condition for the watershed model, thus yielding 8 quantiles for the frequency functions.

Table 15.   Depth-duration-frequency 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

Meteorologic Models for the 8 frequency events were added to the HEC-HMS input. (Because the 0.01-AEP precipitation input was completed in Chapter 2, only 7 additional models need be created.) To create a **Meteorologic Model**, the analyst:

1. Selected **Component**, **Meteorologic Model, New** from the main project definition window.

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 **OK**.

   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 HEC-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 subbasins in the study area by selecting the basin model and clicking the **Add** button as shown in Figure 11. Clicked **Apply** and **OK**.

4. Specified the **Precipitation Method** by selecting **Frequency Storm** from the list, as shown in Figure 12. When HEC-HMS asked "Change precipitation method to Frequency-Based Hypothetical? All data will be lost for User-Specified Hyetograph," clicked **OK**.
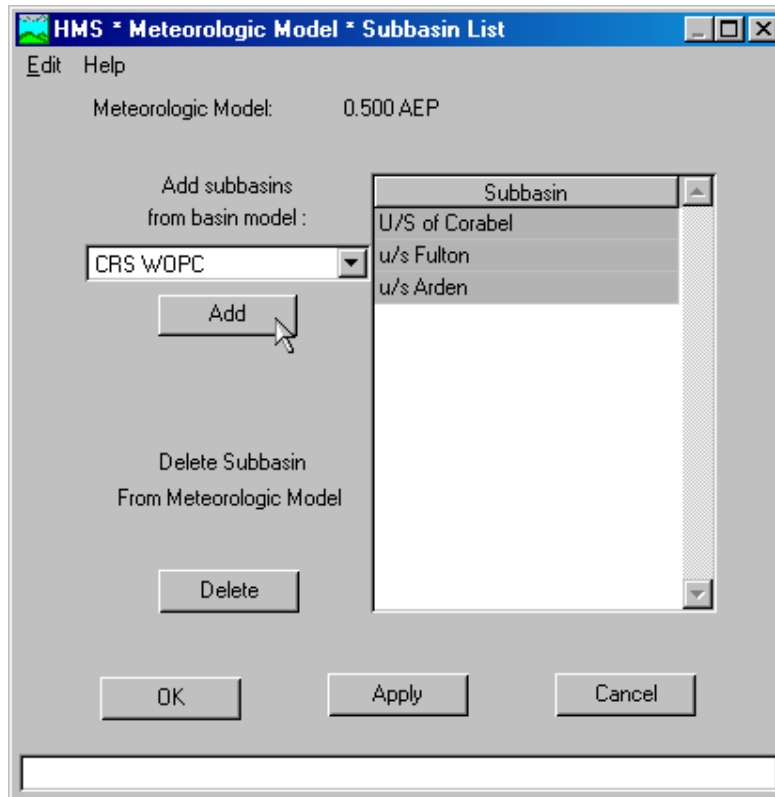
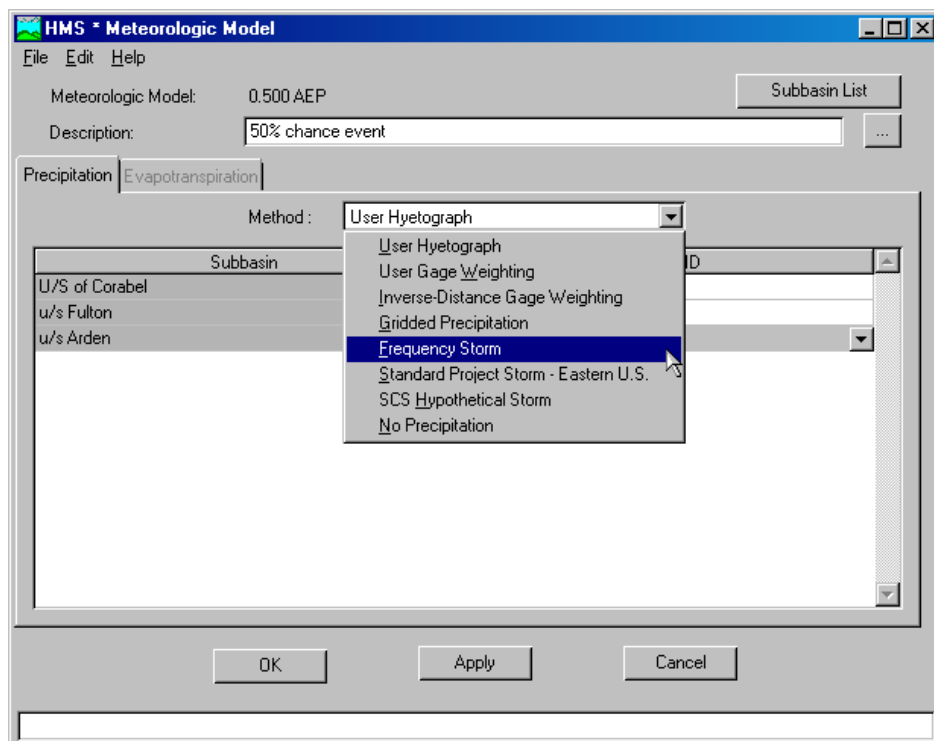*Figure 11.*　　　Connected **Meteorologic Model** to subbasins

Figure 12.　　　Selected appropriate **Precipitation Method**

5. Entered the precipitation depth-duration data from Table 15, as illustrated in Figure 13. Once the input was complete, the analyst clicked **Apply** and **OK**.

Here, the actual **Exceedance Probability** is selected for each of the 8 boundary conditions. (Note that the options shown in the HEC-HMS form are actually exceedance probability multiplied by 100, expressed as a percentage). The **Series Type** identifies the series type of the depth-duration input data. The depth-duration input data is assumed to be partial duration since all *TP-40* data are partial duration. If **Partial Duration** is selected, no changes are made to the specified values. If **Annual** is selected, HEC-HMS converts the specified values to partial duration series depths for 0.50-, 0.20-, and 0.10-AEP events. Multipliers from *TP-40* are used for this. The **Max Intensity Duration** controls how HEC-HMS defines the frequency storm; it uses the procedures described in the *Technical reference manual* (USACE, 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 50% is selected for **Peak Center**, the peak intensity is at 50% of the duration—at 12 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 *Technical reference manual* (USACE, 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; the values used are the point rainfall values entered.

Figure 13.   *Input for 0.500-AEP precipitation event*

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Steps 1–5 were repeated to develop the 8 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 8 copies of the current development condition **Basin Model** (developed in Chapter 2) and 8 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.
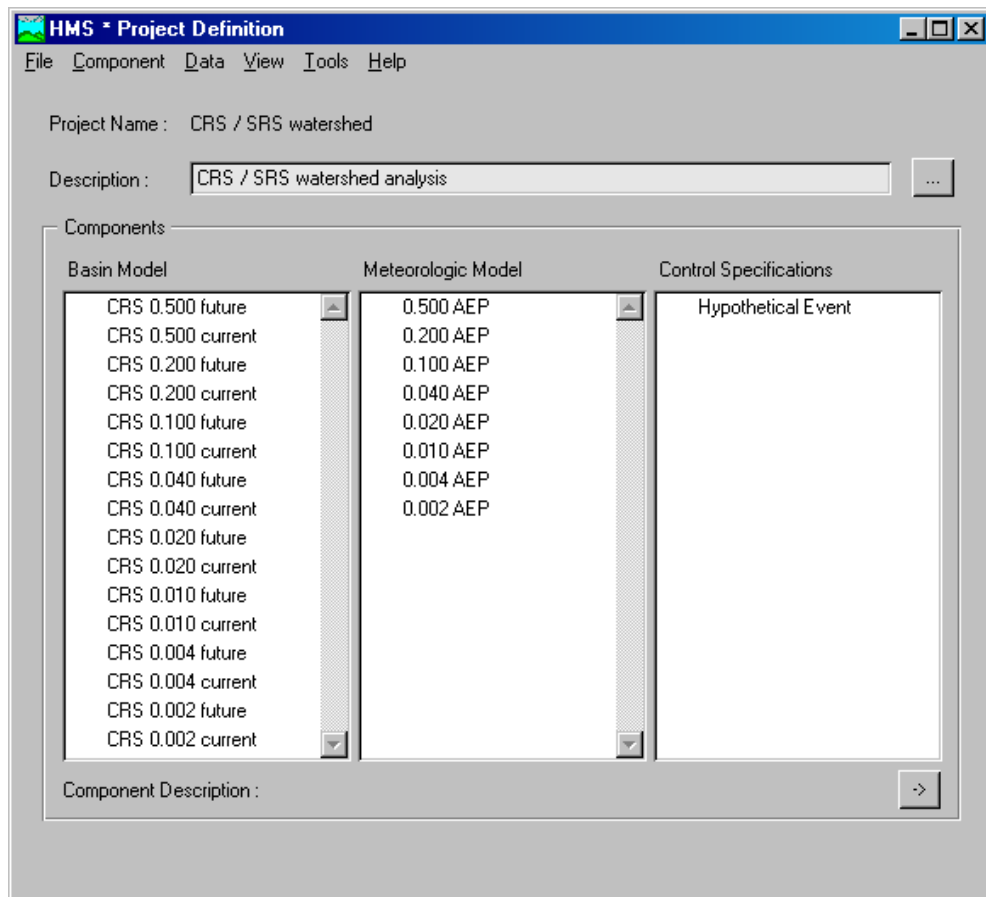
Figure 14.   **Project Definition** *window with required models for frequency analysis*

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

1. Clicked **Component**, **Control Specifications**, **New** from the project definition window.

2. Entered a name for the **Control Specs**, gave a brief **Description**, and clicked **OK**.

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

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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 12 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 input window.
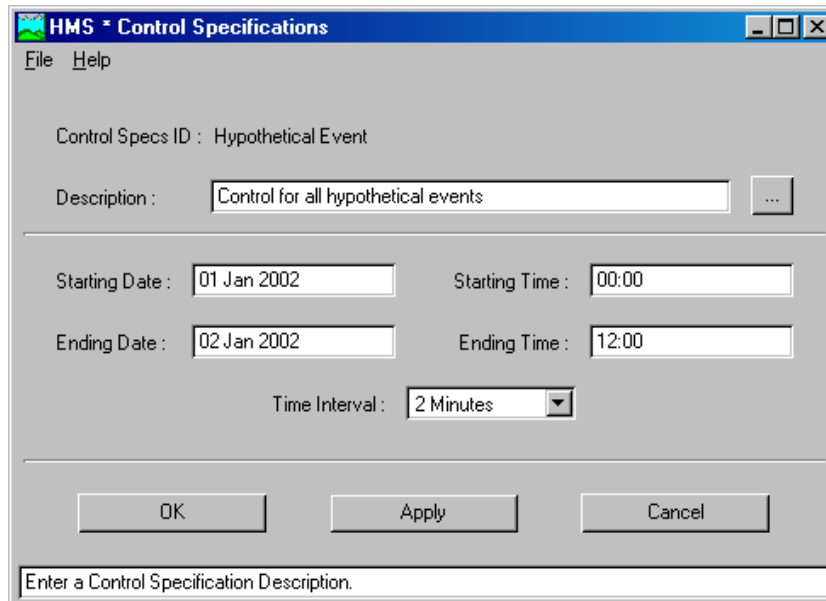
Figure 15.   **Control Specifications** *for hypothetical events*

The analyst exercised HEC-HMS with the current development condition **Basin Model**, 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 8 frequency events at Arden Way is shown in Table 16.

Table 16.   *Without-project, current development peak flow at Arden Way*

AEP	Peak flow for current development condition (cfs)
0.500	285
0.200	445
0.100	569
0.040	713
0.020	824
0.010	941
0.004	1,057
0.002	1,226

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 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* (USACE, 1980).

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 and Crippen, 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 US (Sauer, *et al.*, 1983 and Jennings*, et al.*, 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-FFA (USACE, 1992). From this, both quantiles and confidence limits can be defined.

If gage records are good and exist for a relatively long period, say longer than 30 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 recommend avoiding statistical analysis for short samples of data. Here, for illustration though, the analyst used the 10 years of annual maximum flow at the Corabel gage and developed a flow-frequency function using computer program HEC-FFA (USACE, 1992). 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-FFA, respectively. The 0.50-AEP value compares well. However, the 0.01-AEP flow from HEC-HMS is about ½ the value predicted by fitting a distribution to the data.

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% confidence bands from HEC-FFA. These confidence bands describe the uncertainty about the fitted function. For example, the confidence bands for the 0.01-AEP event shows 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.

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-FFA quantile		
			Expected (cfs)	95% confidence (cfs)	5% confidence (cfs)
0.500	285	140	282	195	410
0.100	569	564	718	440	1,278
0.010	941	1,500	1,808	741	3,482

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:

1.   Calibrate the peak flow from the hydrologic model to match the desired frequency.

2.   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

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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 17 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 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-FFA. 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 16.

Table 19.   Without-project, future development peak flow at Arden Way

AEP	Peak flow for future development condition (cfs)
0.500	309
0.200	483
0.100	610
0.040	762
0.020	879
0.010	1,003
0.004	1,124
0.002	1,301
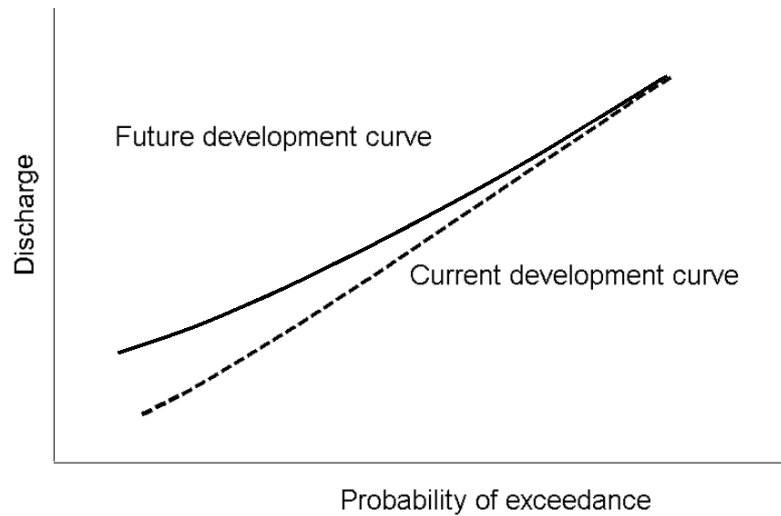
Figure 16.   Comparison of without-project flow-frequency curves

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.
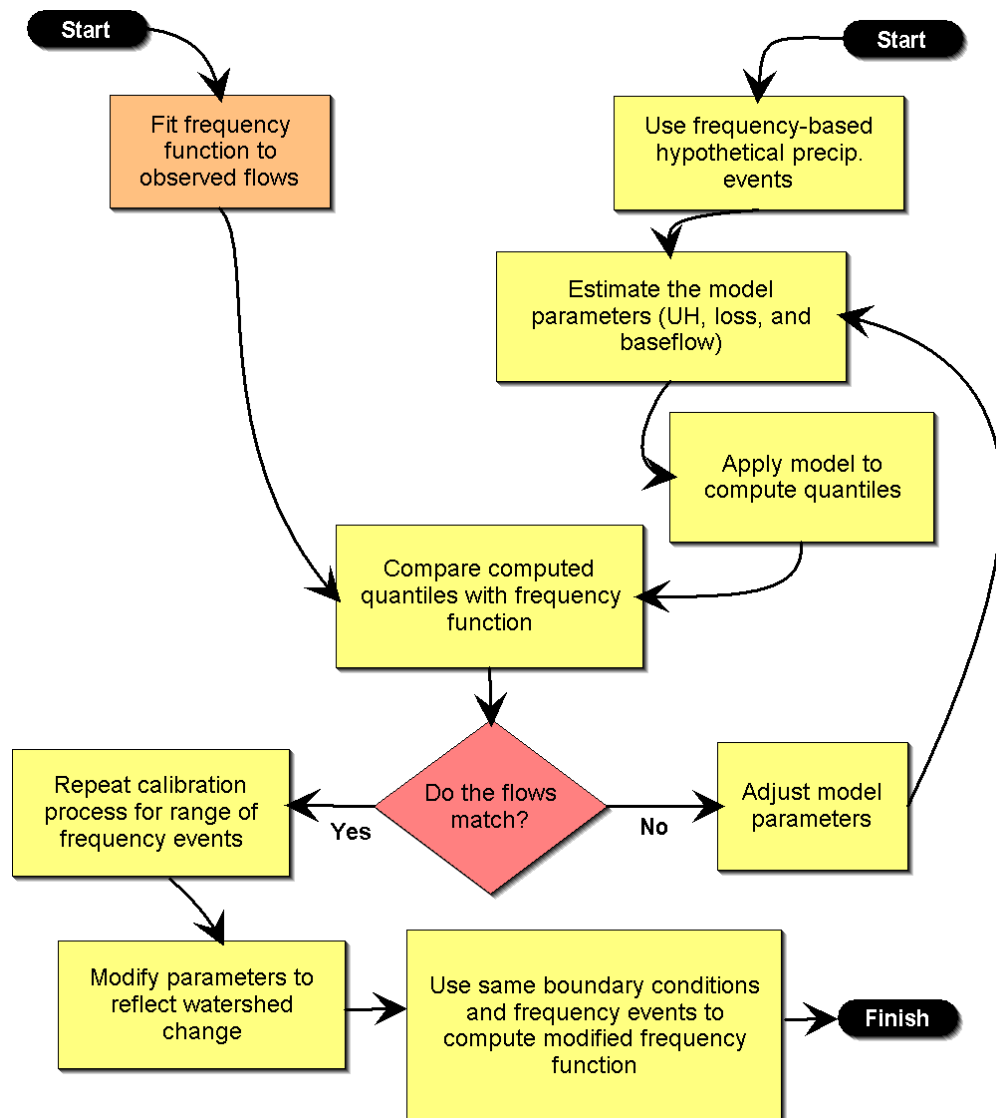
Figure 17.   Method 1 steps to calibrate models to flow-frequency function

Method 2. An alternative method to develop a modified frequency curve based upon an adopted frequency curve is schematically shown in Figure 19 and is as follows:

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

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 18, the 0.01-AEP precipitation event may yield the 0.015-AEP peak flow.

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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).
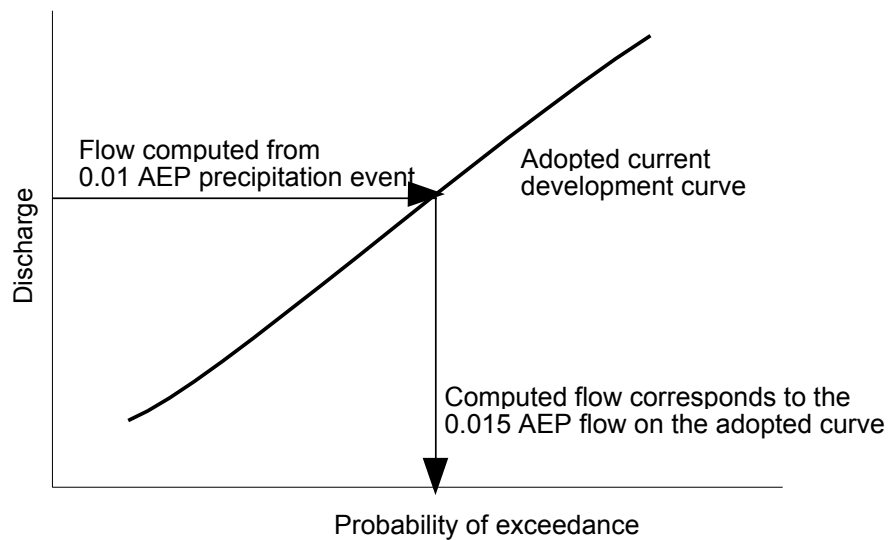
Figure 18. Example of using adopted frequency curve to assign AEP
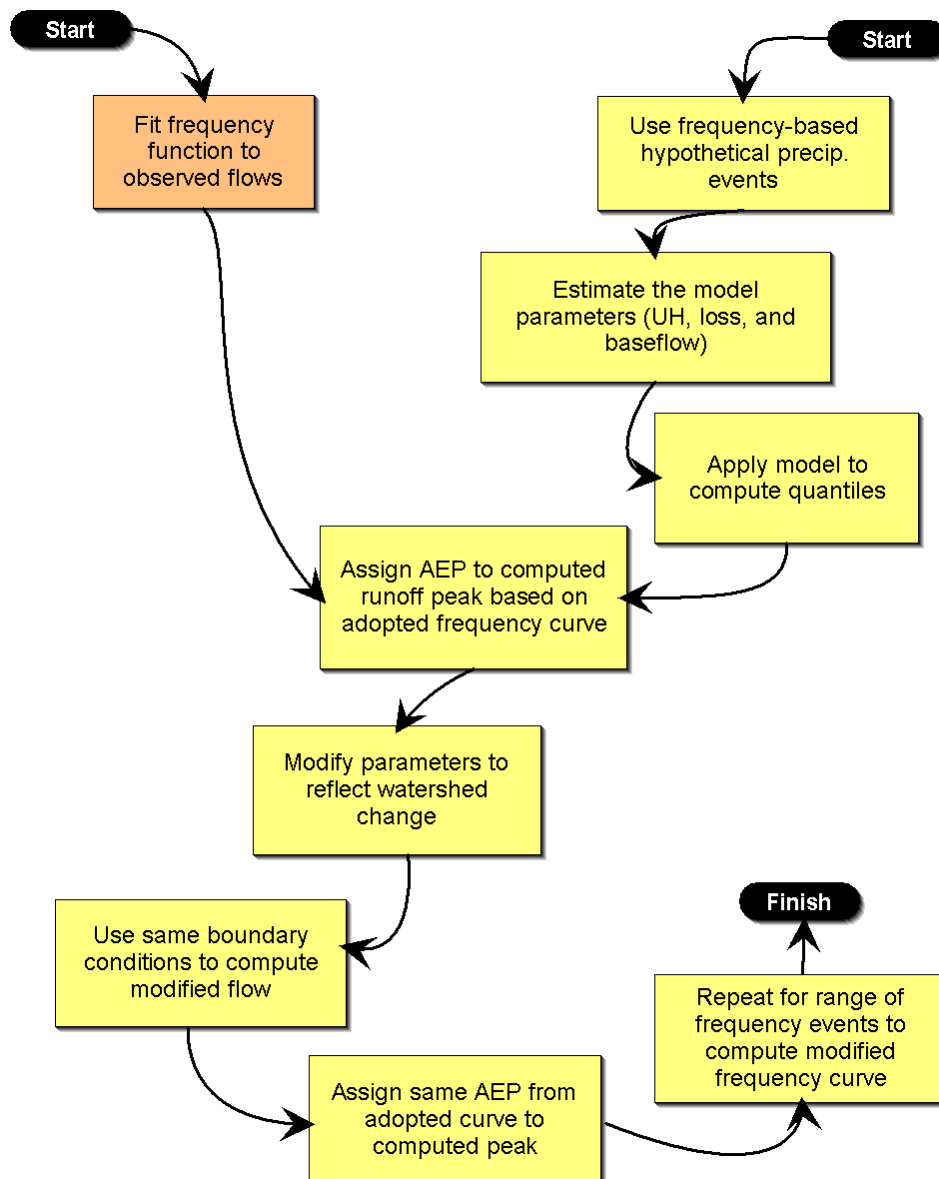
Figure 19.   Method 2 steps to calibrate models to flow-frequency function

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 Chicken Ranch Slough 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.

References

Frederick, R.H., Myers, V.A., and Auciello, E.P. (1977). *Five- to – 60-minute precipitation frequency for the eastern and central United States, HYDRO-35*. National Weather Service, Silver Springs, MD.

Herschfield, D.M. (1961). *Rainfall frequency atlas of the United States for durations from 30 minutes to 24 hours and return periods from 1 to 100 years, TP 40*. Weather Bureau, U.S. Department of Commerce, Washington, DC.

Interagency advisory committee on water data (1982). *Guidelines for determining flood flow frequency, Bulletin 17B*. Hydrologic subcommittee, USGS, Reston, VA.

Jennings, M.E., W.O. Thomas, Jr., and H.C. Riggs (1994). *Nationwide summary of U.S. Geological Survey regional regression equations for estimating magnitude and frequency of floods for ungaged sites, 1993, USGS Water Resources Investigations Report 94-4002*, Reston, VA.

Miller, J.F. (1964). *Two- to ten-day precipitation for return periods of 2 to 100 years in the contiguous United States, TP 49*. Weather Bureau, U.S. Department of Commerce, Washington, DC.

Miller, J.F., Frederick, R.H., and Tracey, R.J. (1973). *Precipitation-frequency atlas of the western United States, NOAA Atlas 2*. National Weather Service, Silver Spring, MD.

Pilgrim, D.H., and Cordery, I. (1975). "Rainfall temporal patterns for design floods." *Journal of the Hydraulics Division*, ASCE, 101(HY1), 81-85.

Sauer, V.B., W.O. Thomas, V.A. Stricker, and K.V. Wilson (1983). *Flood Characteristics of Urban Watersheds in the United States, USGS Water Supply Paper 2207*, Reston, VA.

US Army Corps of Engineers [USACE] (1980). *Adoption of flood flow frequency estimates at ungaged locations, TD-11*. Hydrologic Engineering Center, Davis, CA.

USACE (1992). *HEC-FFA user's manual*. Hydrologic Engineering Center, Davis, CA.

USACE (2000). *HEC-HMS technical reference manual*. Hydrologic Engineering Center, Davis, CA.

Waananen, A.O. and J.R. Crippen (1977) *Magnitude and frequency of floods in California.* Water-resources Investigation 77-21. U.S. Geological Survey, Washington, DC.

C H A P T E R   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

Corps 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 Corps guidance for hydrologic engineering analyses in flood-damage reduction studies includes:

- EP 1110-2-110 *Hydrologic engineering analysis concepts for cost shared flood damage reduction studies*. This document 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 *Hydrologic engineering requirements for flood damage reduction studies*. This identifies possible damage reduction measures and summarizes typical hydrologic engineering studies required for formulation and evaluation of each.

- EM 1110-2-1619 *Risk-based analysis of flood damage-reduction studies*. This engineering manual 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

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plan that yields the maximum net benefit. As described in EM 1110-2-1419, net benefit, *NB*, of a proposed plan is computed as:

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

in which B_L = annual equivalent location benefit of the plan; B_I = annual equivalent intensification benefit of the plan; $E[D_{without}]$ = expected annual damage (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 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 Corps 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 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 programs 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 (USACE, 1998) 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.

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 Chicken Ranch Slough / Strong Ranch Slough 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 Corps 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 Chicken Ranch Slough into Strong Ranch Slough at Fulton Avenue.

Alternative 4: Floodwall along Chicken Ranch Slough, 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 4-5 of EM 1110-2-1619.

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.
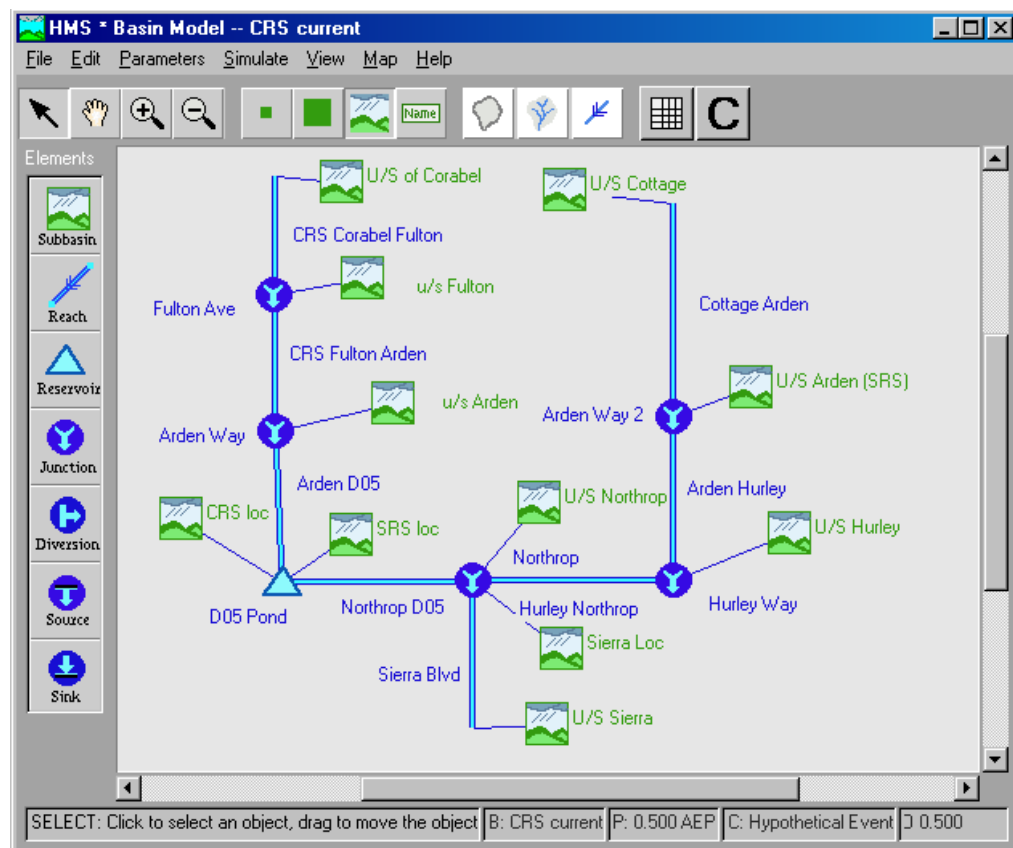
Figure 20.   Complete basin model of CRS/SRS watershed

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 must define a storage-outflow relationship for the pond and outlet works. Similarly, HEC-HMS includes a diversion model; all properties of the diversion are described with a function that predicts flow into the diversion channel, given the flow in the channel at the point of diversion.

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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 8-point channel routing model; 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 8-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. The routing models in HEC-HMS do not provide estimates of stage or of a rating function. 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 8 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	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

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 40-acre detention pond to these basin runoff models. To do so, the detention element was dragged from the toolbar and dropped onto the schematic diagram in the appropriate location, as shown in Figure 21. To specify the properties of the detention, the analyst double-clicked the detention icon and completed the form that appeared. As shown in Figure 22, three options are available for specifying the properties; the analyst here selected **Elevation-Area-Outflow**. In columns 1 and 2 of the grid on the form, the analyst specified pond pool elevations and the corresponding areas that would be used in the pond. 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. In column 3 of the grid, the analyst specified the total outflow from the pond for the corresponding elevations from column 1. 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. A number of commercial off-the-shelf computer programs are available for computing culvert flow for these cases, including HEC-RAS. However, in this case, the analyst used nomographs to develop the rating.
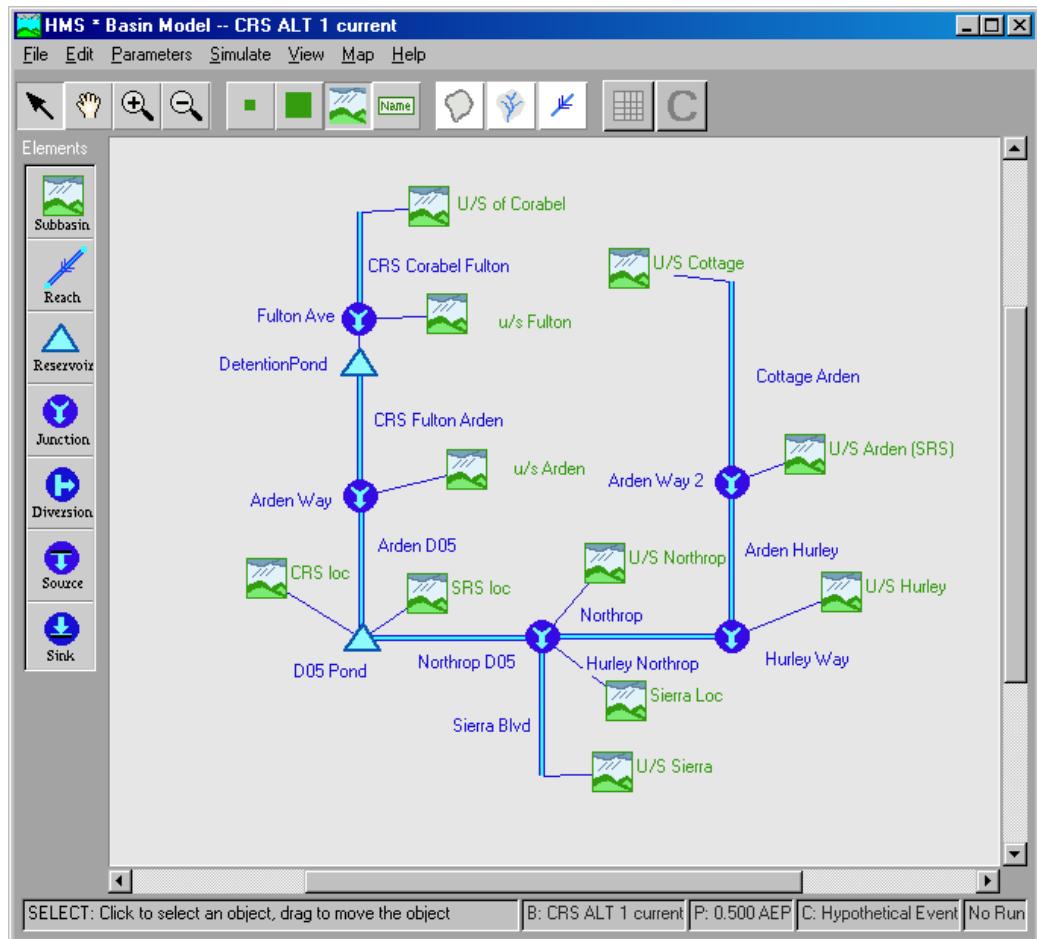
Figure 21.   Basin model for alternative 1

Figure 22.   *Form for specifying properties of detention pond*

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
    8 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
    editor. To do so, from the **Basin Model** screen, the analyst selected
    Parameters, **Loss Rate**, **Initial/Constant…** The values were then set in the
    resulting window, as shown in Figure 23.
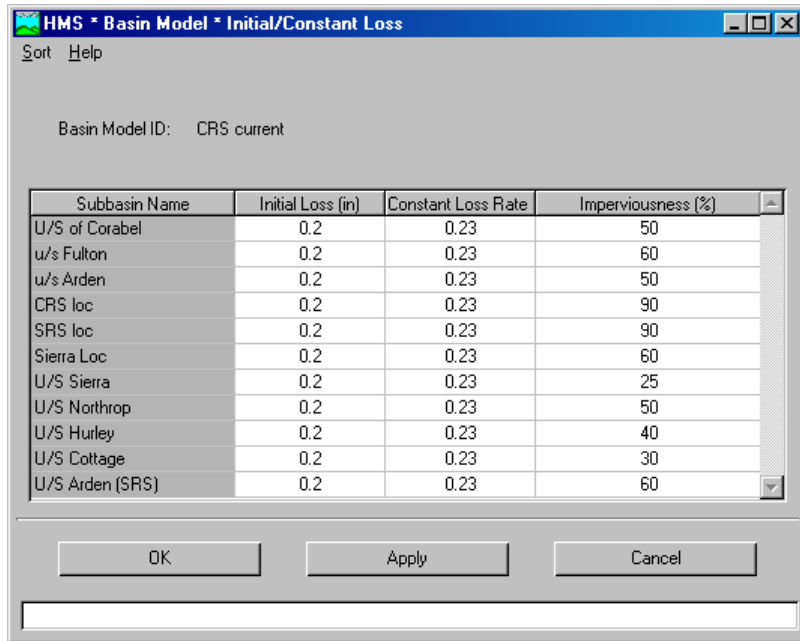
Figure 23.   Alter parameter values using global editor

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

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

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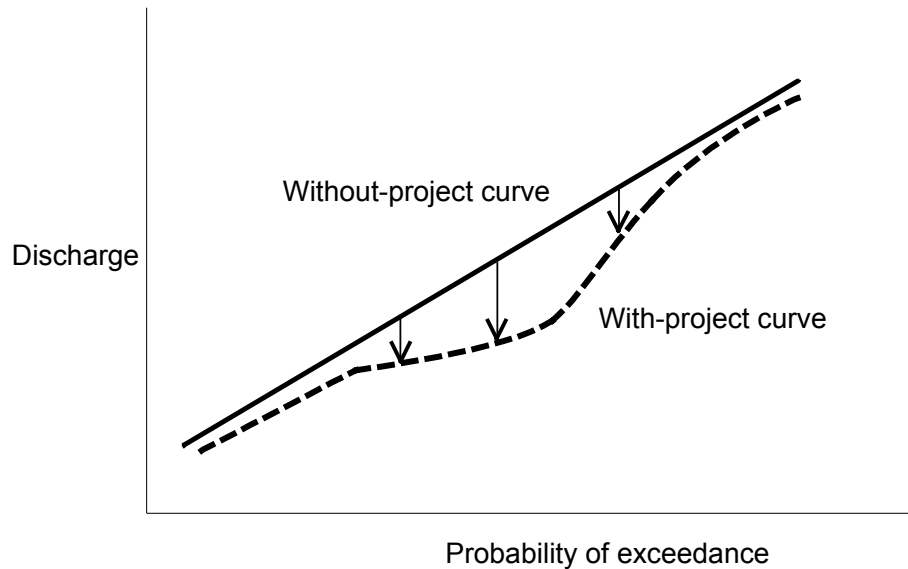
Figure 24.   Conceptual effect of alternative 1 on the flow-frequency curve

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 40 acres to 20 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 20-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.
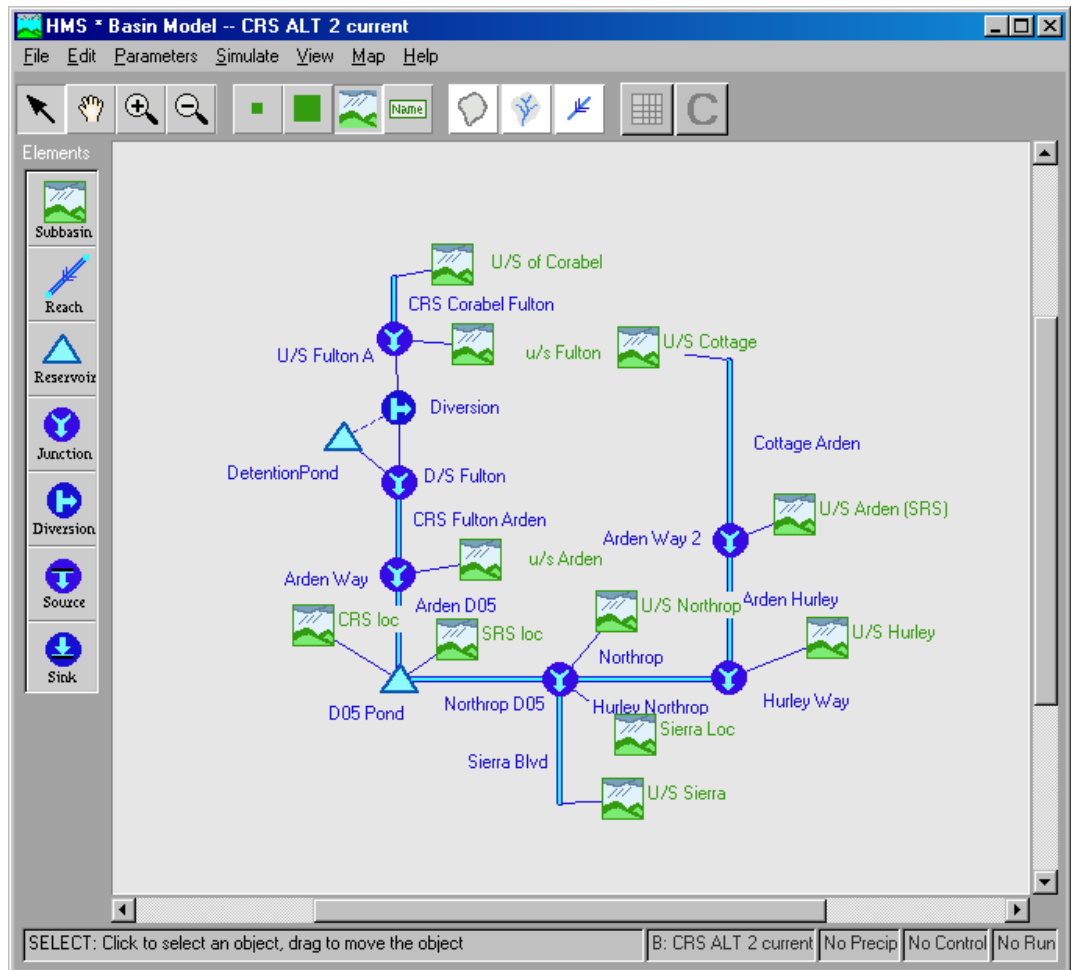
Figure 25.   Basin model for alternative 2

To describe the diversion, a performance function must be entered in the form 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.

Figure 26.   *Diversion properties form*

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 8 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

8. **Basin model for alternative 3**. Figure 27 is a sketch of the proposed inflow to the diversion channel for alternative 3. Water will overflow a side-channel spillway into the diversion channel, which will be connected to Strong Ranch Slough, where the likelihood of flood damage is less.
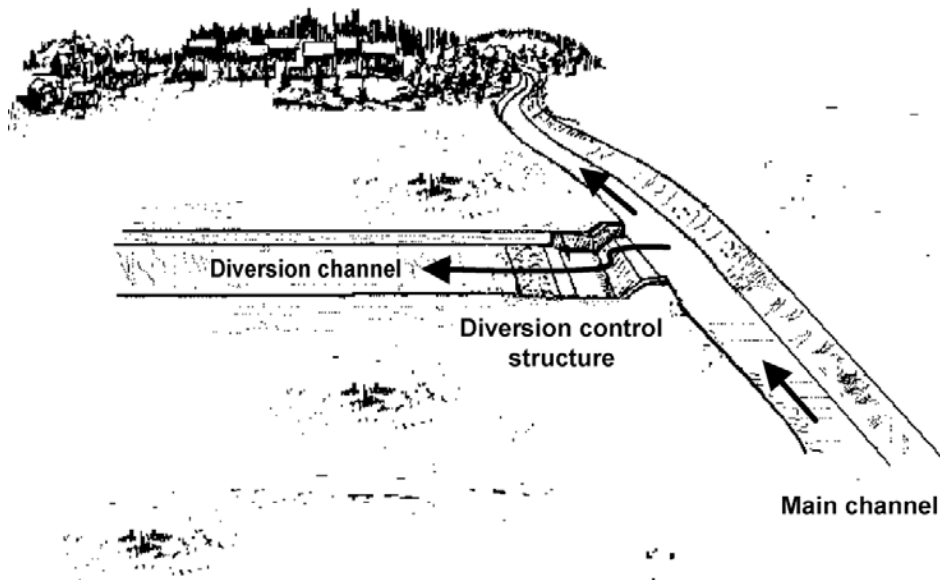
Figure 27.   *Sketch of proposed diversion control structure for alternative 3*

Figure 28 illustrates the configuration of the basin model for this alternative. This alternative includes a model of the without-project condition subbasins. It also includes a model of the diversion in the Chicken Ranch Slough watershed. The diversion channel is connected to a junction in the Strong Ranch Slough watershed. There, the diverted flow hydrograph is retrieved and added to the channel flow by including a junction, as shown. The total flow is then routed down Strong Ranch Slough.

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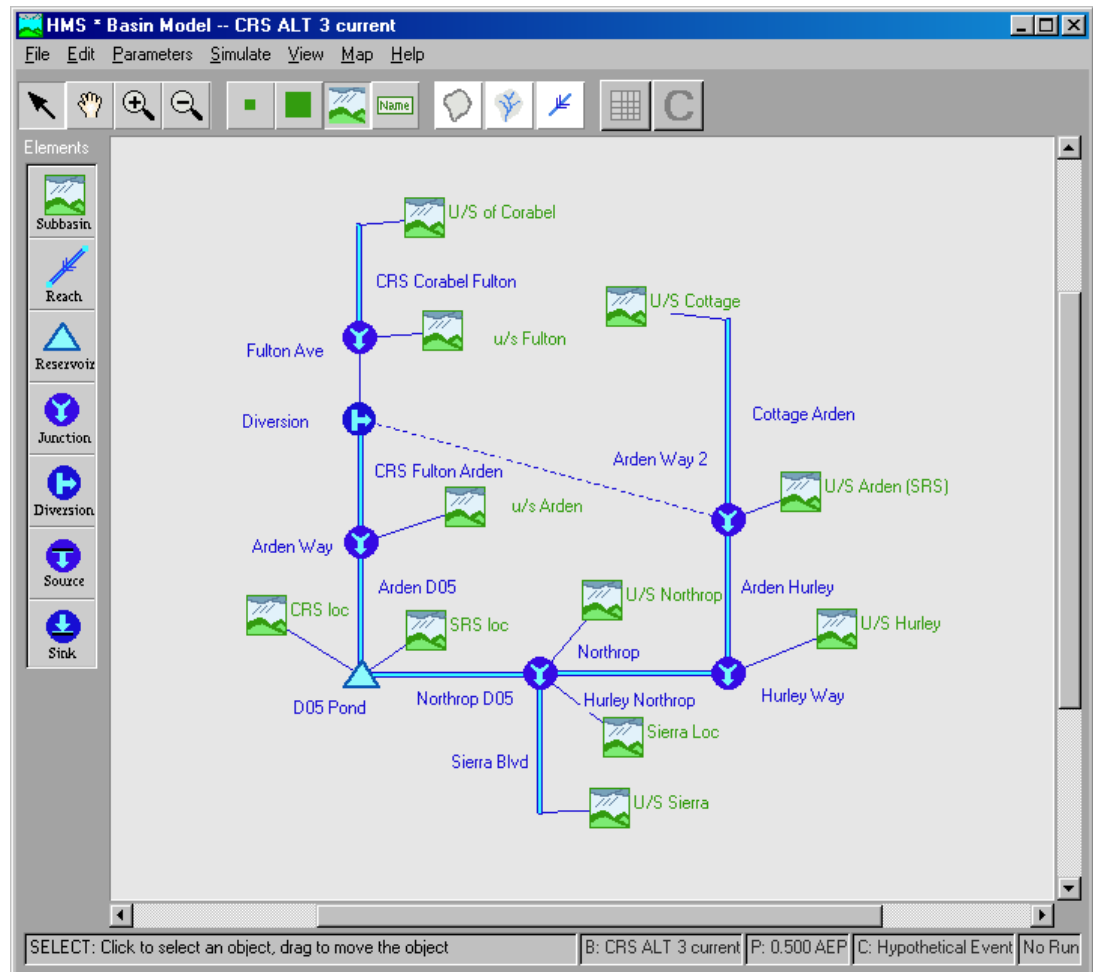
Figure 28.   Basin model for alternative 3

The properties of the diversion were specified by the analyst in the diversion form, which is illustrated in Figure 26. The rating in this case was derived by developing an HEC-RAS model of the spillway with proposed dimensions and running that program 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 8 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 8-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 8 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 two ways to copy basin models in HEC-HMS. The first way is to use the import tool. First, in the Project Definition form, the analyst selected **Component**, **Basin Model**, **Import**. When the import form appears, as illustrated in Figure 29, the analyst selected the *current* project from the list in the right-hand pane. When the list of basin models in the project was displayed, as shown, the analyst selected the without-plan model, assigned an appropriate name, and clicked **Import**. The new basin model was inspected carefully to confirm that all input is as expected. Alternatively, the analyst could select **File**, **Save Basin Model As** from the Basin Model form.
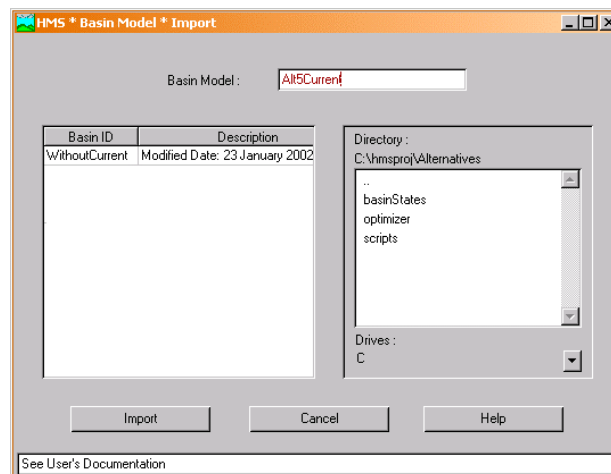
Figure 29.   Basin model import form

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 the equivalent years of record. Guidelines from EM 1110-2-1619 were used to estimate the equivalent years of record to be 10. 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.

Table 27.   EAD and inundation reduction benefit of alternatives

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

Figure 30.   Flow-frequency function with uncertainty distribution

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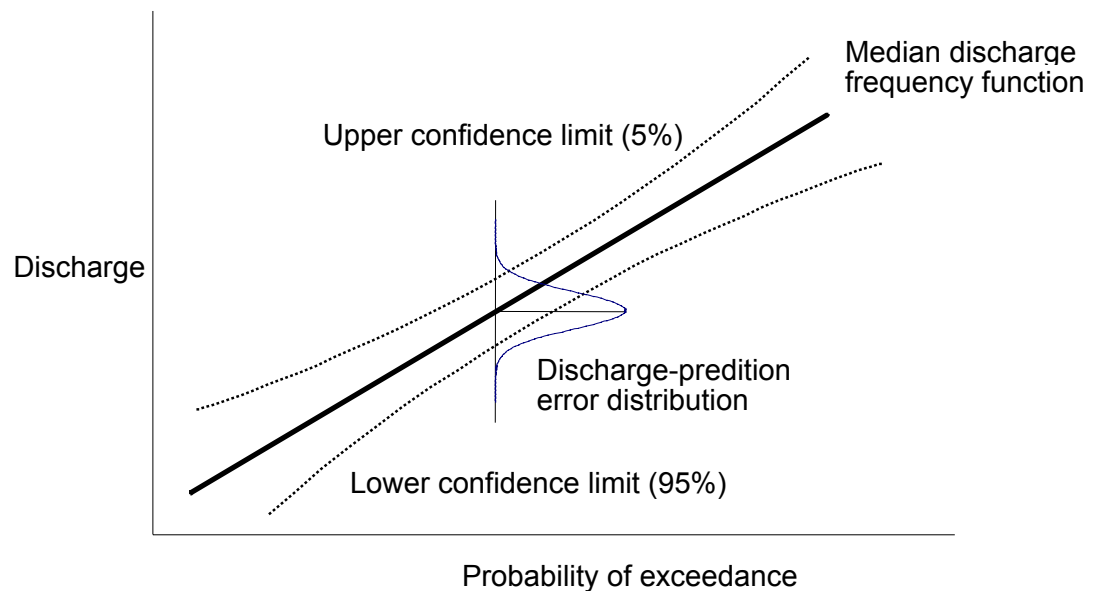 frequency functions. With this, the long-term risk associated with each plan can be computed. EM 1110-2-1619 describes the computational methods. EM 1110-2-1619 calls also for modeling the uncertainty in the frequency functions; this uncertainty is a consequence of lack of data and lack of certainty 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 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 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.

References

U.S. Army Corps of Engineers [USACE] (1994). *Hydrologic engineering analysis concepts for cost shared flood damage reduction studies, EP 1110-2-10*. Office of Chief of Engineers, Washington, DC.

USACE (1995). *Hydrologic engineering requirements for flood damage reduction studies, EM 1110-2-1419.* Office of Chief of Engineers, Washington, DC.

USACE (1996). *Risk-based analysis for flood damage-reduction studies, EM 1110-2-1619.* Office of Chief of Engineers, Washington, DC.

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

C H A P T E R 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.
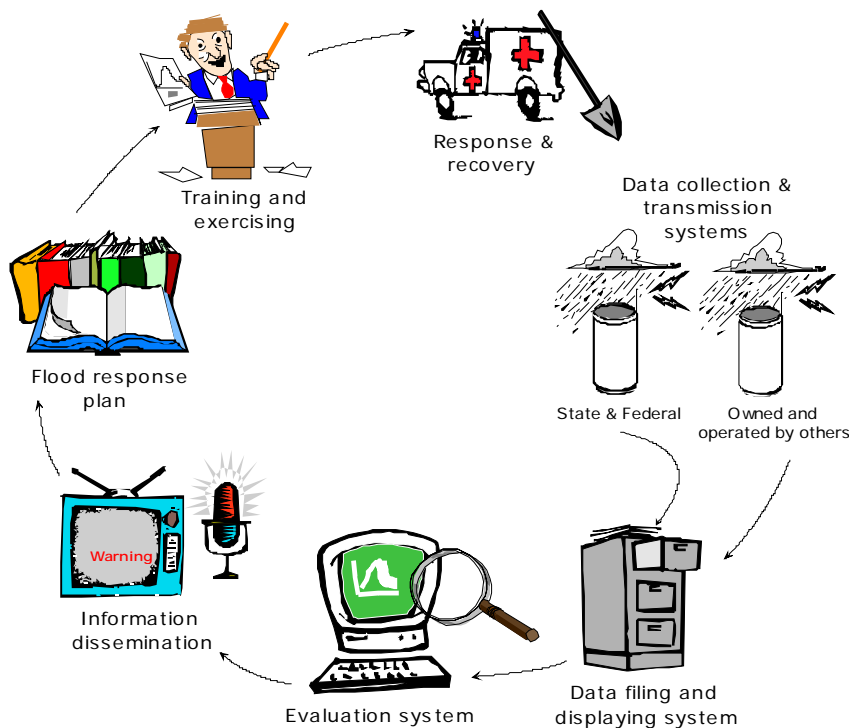
(c)2001, David Ford Consulting Engineers

Figure 31.   Components of flood response and emergency preparedness system

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 U.S. is UHF or VHF radio broadcast using the ALERT (Automated Local Evaluation in Real Time) standard. This standard was established by the National Weather Service and its cooperators (NOAA, 1997 and www.alertsystems.org). 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 40-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

Corps 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 Corps guidance is particularly relevant to flood warning system planning and design:

- ER 1105-2-100 *Planning guidance notebook*. This provides guidance and describes procedures for all civil works planning studies.

- EM 1110-2-1417 *Flood-runoff analysis*. This describes procedures for flood runoff analysis, which is critical to FWS planning.

- ETL 1110-2-540 *Hydrologic aspects of flood warning – preparedness programs*. This 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 Corps 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 (USACE, 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.
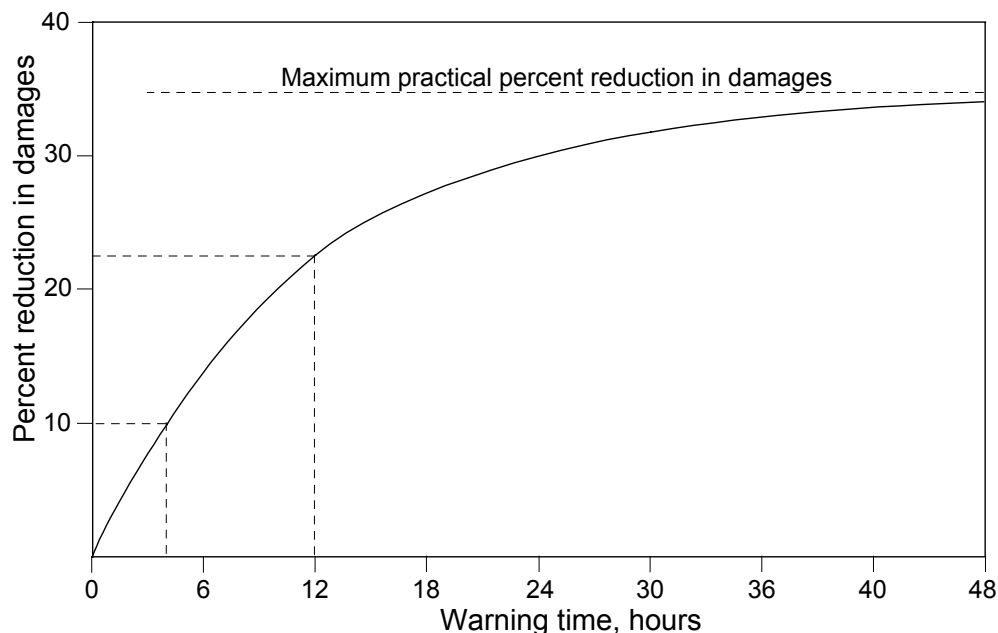
Figure 32.   Day curve

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. It 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 0 hours, a FWS provides no

direct tangible benefit. If the warning time is 12 hours, the curve predicts that the damage will decrease by 23%. The curve also suggests that no matter how great the warning time, the maximum possible reduction is about 35% 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 10 hours, and if 6 hours are required for detection, 1 hour required for emergency responder notification and decision making, and 1 hour required for public notification, then the actual mitigation time is only 2 hours. If the FWS reduces the detection time to 4 hours, the mitigation time increases to 4 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 U.S. 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. The 1997 flood caused $60 million in damage and took 3 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 20 feet in some locations.

Figure 33.   Urban flooding in the watershed for which FWS is proposed

Citizens asked the Corps 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, Corps 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, Corps 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%.

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:

$$E[T_w] = \int T_w(p)\,dp \tag{5}$$

where $E[T_w]$ = the expected value of warning time; p = annual exceedance probability (AEP) of the event considered; and $T_w(p)$ = the warning time for an event with specified AEP. The actual value will be computed with the following numerical approximation:

$$E[T_w] = \sum T_w(p_i)\,\Delta p_i \tag{6}$$

where p_i = annual exceedance probability (AEP) of event i; $T_w(p_i)$ = the warning time for event i; and Δp_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 the figure. 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.

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 the figure—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.
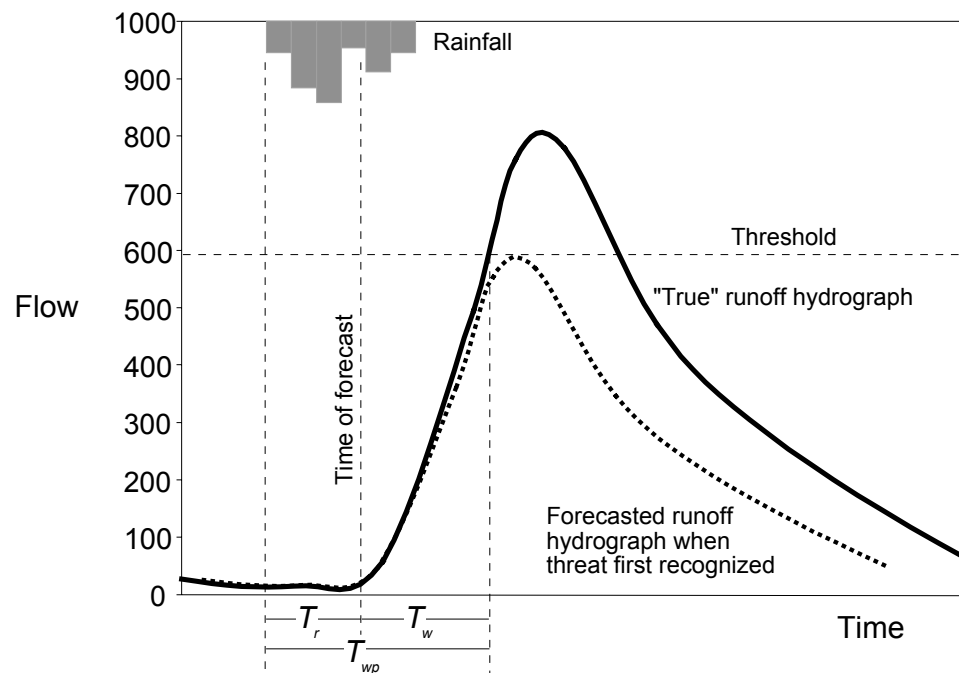
Figure 34.   Warning time components

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, ΔT, 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 ΔT of 30 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 60 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, 30 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.

    This computation scheme simulates the gradual formation of a storm, observation of the data over time, and attempted detection of the flood event.

5. 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}.

6. 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, she 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 the National Weather Service.

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 were 6 hours. Figure 35 shows the computed hydrograph for the 8 storms, using the rainfall for the entire 6-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.
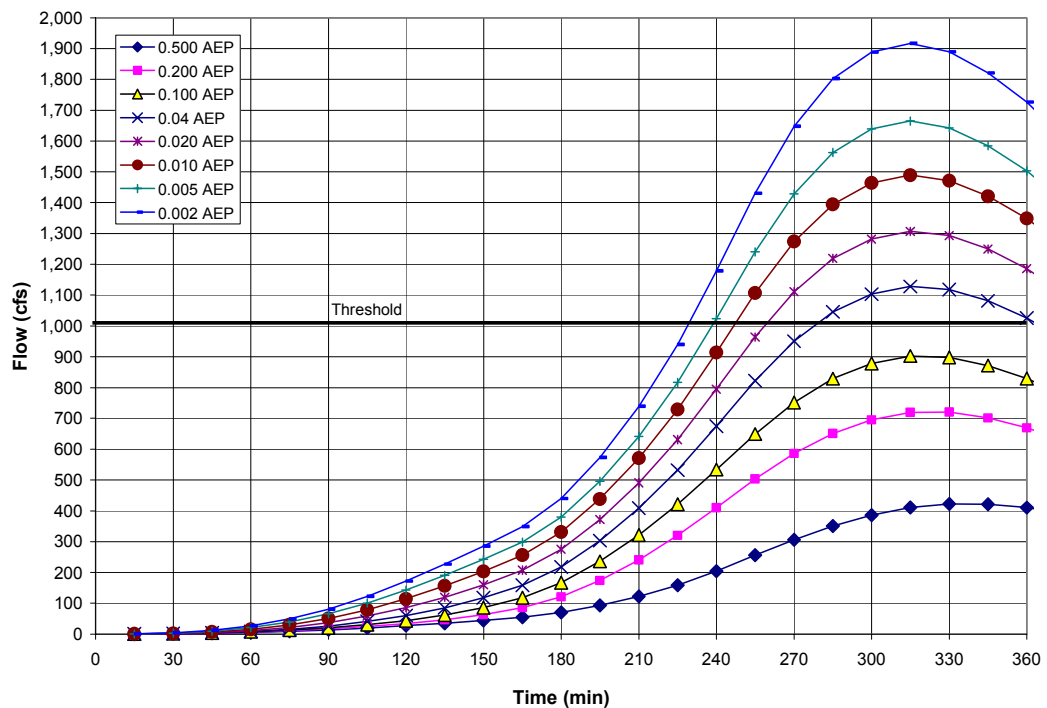
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Figure 35.   *Flow hydrographs at the critical location using entire precipitation events*

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[2], T_w (min)
0.500	—[1]	—[1]	—[1]
0.200	—[1]	—[1]	—[1]
0.100	—[1]	—[1]	—[1]
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

Notes:
1. Threshold not exceeded by event shown.
2. Maximum mitigation time is T_{wp}-T_r. Time for notification, decision making, etc. will reduce time actually available for mitigation.

To complete the runoff analysis as proposed, each of the hypothetical events was analyzed with increasing duration of rainfall data, starting with 60 minutes of data. The analyst increased the duration of data in 30-minute blocks and computed a runoff hydrograph.

To create the meteorologic models for this, the analyst used HEC-HMS and a combination of HEC-DSS data management tools. First, the HEC-DSS file in which 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 she found a file with extension .DSS. In each project, HEC-HMS stores in this file all the computed and input time series for the project, including, in this case, the rainfall hyetograph for the 6-hour design storm. Next, the analyst used one of the available tools for manipulating HEC-DSS files. In this case, she used DSSUTL, and with it created duplicates of the HEC-DSS record in which the 6-hour storm hyetograph is stored.

Records stored in an HEC-DSS file have a unique name consisting of 6 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 60-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 HEC-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 6-hour storm was deleted to create the 60-minute sample, and the final 270 minutes was deleted to create the 90-minute sample. Eleven such copies were made for each of the 8 frequency based storms, for durations from 60 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 storms that are not included in HEC-HMS.) To create the gage series, the analyst selected **Data**, **Precipitation Gages…** in the **Project Definition** form to access the **Precipitation Gage Manager**, which is shown in Figure 36. Then, to add a gage, the analyst selected **Edit**, **Add Gage**.
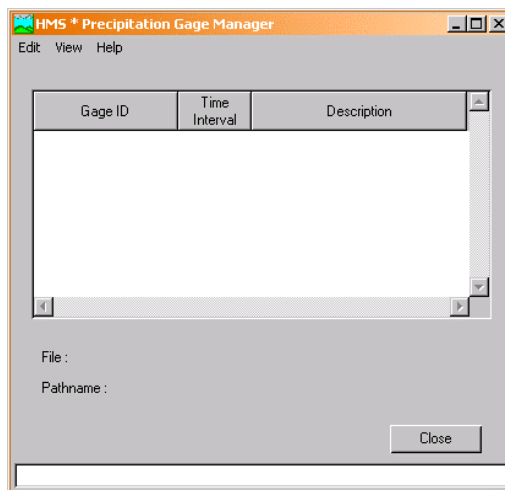
Figure 36.   Precipitation Gage Manager

When a new gage is defined, the HEC-HMS user has the option to retrieve the values from an existing HEC-DSS file or to enter manually the ordinates by typing in a spreadsheet-like grid. The analyst here selected the first option by clicking the **External DSS Record** radio button on the **New Precipitation Record** form, as shown in Figure 37.

Figure 37.   New Precipitation Record form

When **OK** is clicked, HEC-HMS displays the form illustrated in Figure 38. With this, almost any record from any HEC-DSS file to which the analyst has access can be selected and assigned to the gage. To select the appropriate record, the analyst clicked **Generate Catalog**, and picked from among those created with DSSUTL, as described above.

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Figure 38.   Form for selecting HEC-DSS record with data to be assigned to new gage

Note that by selecting the **Browse…** button on the **DSS Pathname Select** form, the analyst could navigate to another HEC-DSS file, including those stored on a server, on another computer linked to the local network, or on a permanent storage device, such as a CD-ROM.

This task of adding a gage and associating data from the HEC-DSS file with it was repeated to create one gage for each combination of storm and duration. The result was a set of 88 gages for the 8 frequency-based storms.

Next, the analyst created a meteorologic model for each of the cases, using the 88 rain gages to define the hyetographs. To do so, the analyst created a meteorologic model and selected **User Hyetograph**, as shown in Figure 39. In the column labeled **"Gage" ID**, the analyst selected from among the 88 gages.
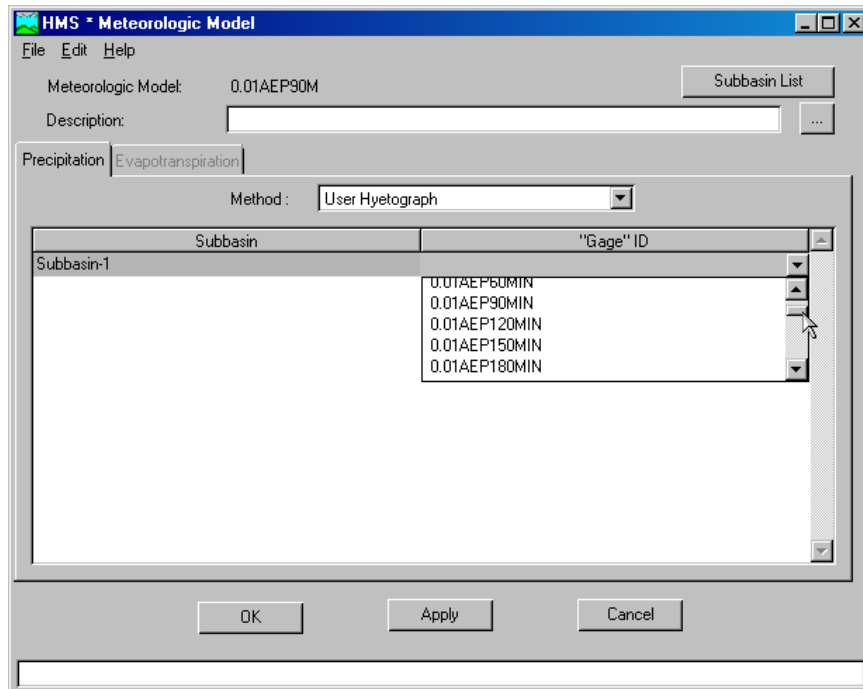
Figure 39.   User hyetograph is selected and gage identified with Meteorologic Model form

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 selected **Tools…**, **Run Configuration** to "assemble" the runs, using the basin model and control specifications in combination with each of the rainfall events.

Figure 40 shows the results of the HEC-HMS runs. For each frequency storm, the computed peak with each portion of the 6-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 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 of 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 20 minutes is required for notification and decision making, the mitigation time available for the 0.002-AEP event will be only 40 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.

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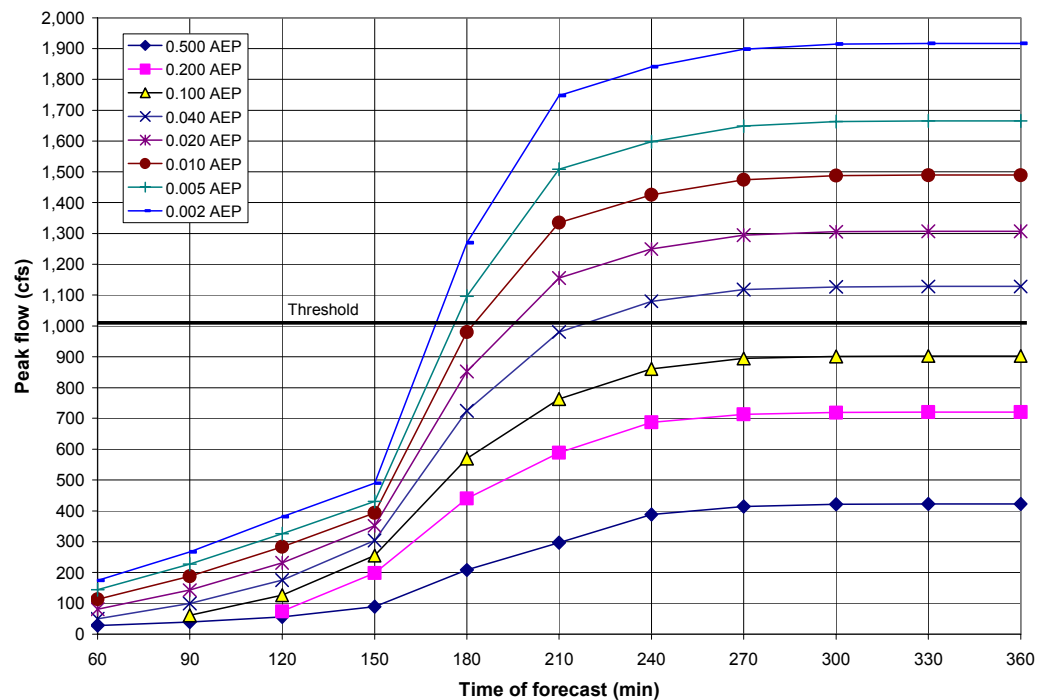
Figure 40.   *TOF versus peak flow at the critical location*

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 20 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.

References

National Hydrologic Warning Council (2001). ALERT (*www.alersystems.org*).

National Oceanic and Atmospheric Administration (1997). *Automated local flood-warning systems handbook.* Weather service hydrology handbook No.2. National Weather Service, Silver Spring, MD.

U.S. Army Corps of Engineers [USACE] (1994). *Framework for Estimating National Economic Development Benefits and Other Beneficial Effects of Flood Warning and Preparedness Systems.* Prepared by Charles Yoe. Institute for Water Resources, Alexandria, VA.

USACE (1996). *Hydrologic aspects of flood warning-preparedness* programs*. ETL 1110-20540*. Washington, D.C.

C H A P T E R   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, Corps 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 Corps 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, Corps 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). 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 Corps 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, *et al*. (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 Corps' 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" (World Meteorological Organization, 1983). In the U.S., the properties of a PMP commonly are defined by the National Weather Service.

Procedural guidance

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

- EM 1110-2-1411 *Standard project flood determination.* This provides background for the development of the standard project precipitation method used by the Corps.

- ER 1110-8-2 *Inflow design floods for dams and reservoirs.* This 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 *Hydraulic design of spillways.* This manual 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 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 powerplant 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. Corps 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 SCS curve number (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.

- **Direct-runoff transform**. 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 3 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 50%. 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 50% 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 15-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 U.S. in Hydrometeorological Reports (HMR). For example, the eastern US is covered by HMR No. 51 (NWS, 1977) and No. 52 (NWS, 1981). Because this is such a large area with many Corps projects, HEC developed software HMR52 (USACE, 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 6-hour increments. These 6-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% 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 6-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 6-hour values are distributed in an alternating block sequence, with largest values in the center.

For this watershed, the computation time interval selected was 15 minutes, so depths for durations shorter than 6 hours and for intervals less than 6 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.
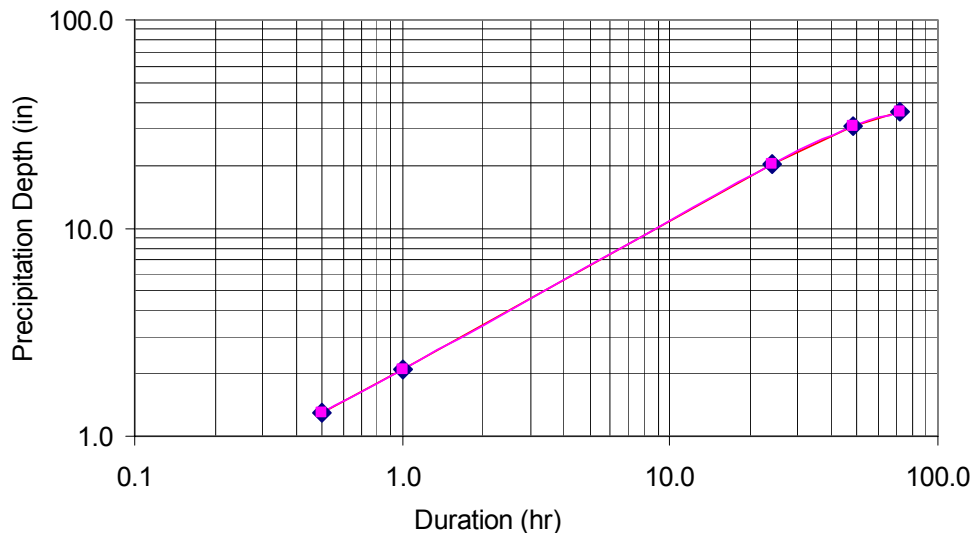

Figure 42.   PMP depth-duration curve for Bonanza Dam

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

To specify the PMP depths, the analyst used the hypothetical storm precipitation option. The form, which is shown in Figure 43, does not permit entry of a 72-hour rainfall depth, so depth for a duration of 96 hours (4 days) was estimated and entered. The peak volume stored in the reservoir is a function of the PMF peak discharge. A 2-day event could have been selected rather than the 4-day event. The

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2-day event would yield the same peak discharge, stage, and volume of water in the reservoir.
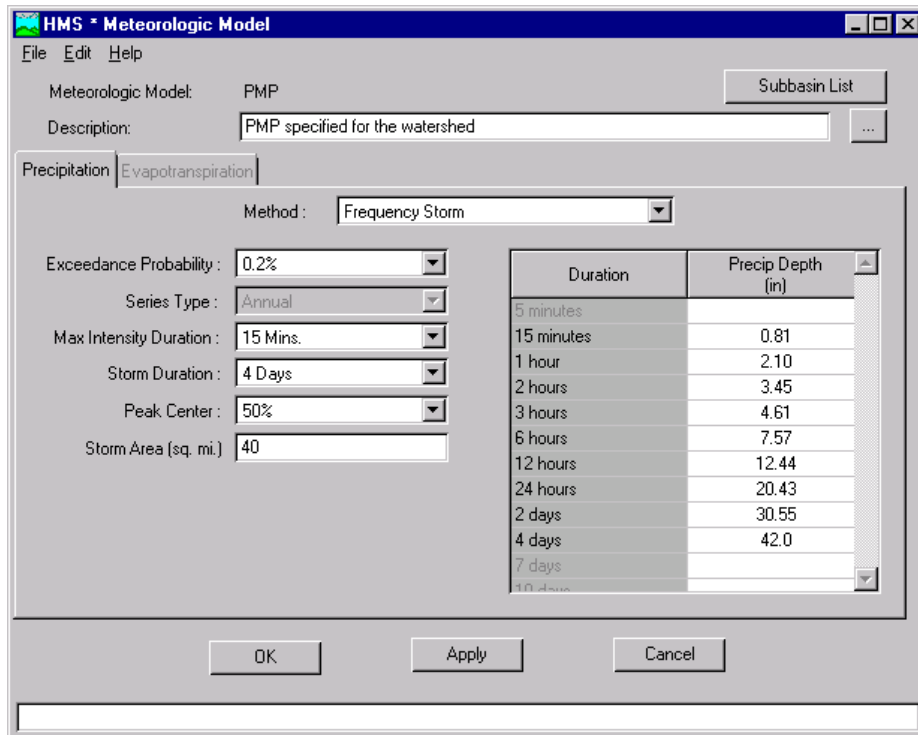

Figure 43.   PMP rainfall input

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.
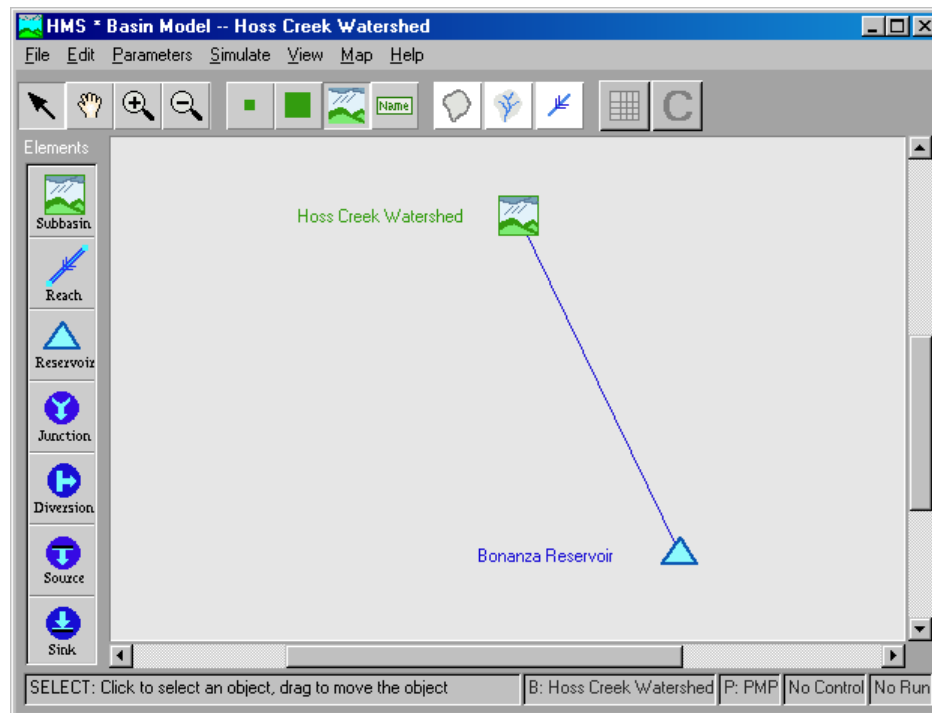
Figure 44.   Basin model for PMP evaluation

Table 31 shows properties of the existing reservoir and outlet works. 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. The elevation-discharge relationship would be developed with hydraulic modeling studies of the outlets and the existing spillway.

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 outflow from the reservoir must pass over the spillway. Thus, the discharge values shown in Table 31, which were used in the analysis, represent only spillway flow.

The analyst also considered the possibility of tailwater control. However, because all flow would pass over the elevated spillway, tailwater was not a factor.

Table 31.   *Stage-discharge-storage volume data for existing spillway*

Elevation (ft)	Discharge (cfs)	Reservoir volume (ac-ft)
3,900	0	0
4,100	0	31,400
4,184	0	123,300
4,185	560	124,680
4,186	1,584	126,300
4,187	2,910	127,950
4,188	4,480	129,650
4,189	6,261	131,420
4,190	8,230	133,150
4,191	10,371	135,000
4,192	12,671	136,700

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 Routing** form.

Application

The HEC-HMS model was completed, and the event simulated. A peak spillway discharge of 15,592 cfs was computed. The maximum water surface elevation in the reservoir was 4,193 feet. As the 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.
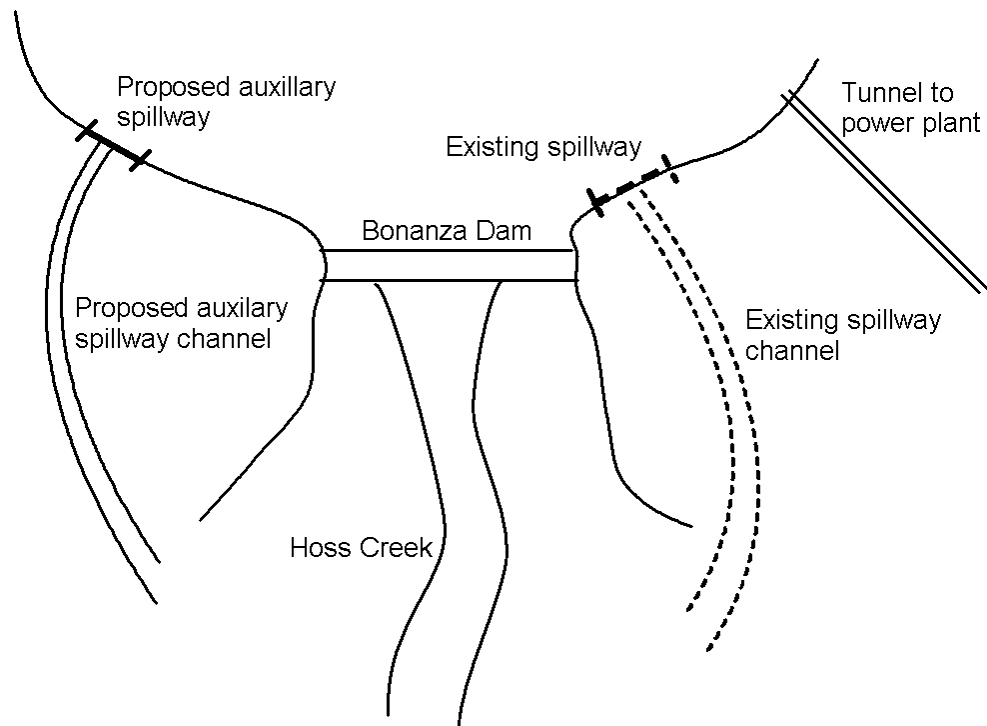
Figure 45.   Plan view of proposed design to increase spillway capacity

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. A candidate auxiliary-spillway crest elevation (and hence, a depth) was proposed.

2. The weir equation was used in a spreadsheet program to develop the elevation-outflow relationship. This equation is $Q = CLH^{3/2}$, where Q = discharge in cfs; C is the discharge coefficient (3.20 was used here), L = spillway crest length, and H = reservoir water surface elevation minus crest elevation. A range of reservoir surface elevations was used to develop the rating for the candidate crest elevation.

3. The elevation-discharge function for the candidate auxiliary spillway design was combined with the elevation-discharge function for the existing spillway. The resulting function was entered and used for simulation with HEC-HMS.

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 computed rating function for the auxiliary spillway with crest at elevation 4,186.0, and Figure 46 shows the HEC-HMS form in which the function is entered. As before, the analyst specified **Initial Elevation** equal 4,184 feet, indicating that the reservoir is initially full. This crest elevation for the auxiliary spill satisfied the criterion for freeboard. The maximum reservoir water surface elevation was 4,190.4; this is 1.6 feet below the top of the dam.

Table 32.   *Elevation-discharge function for main plus auxiliary spillway with auxiliary spillway crest at elevation 8186.0*

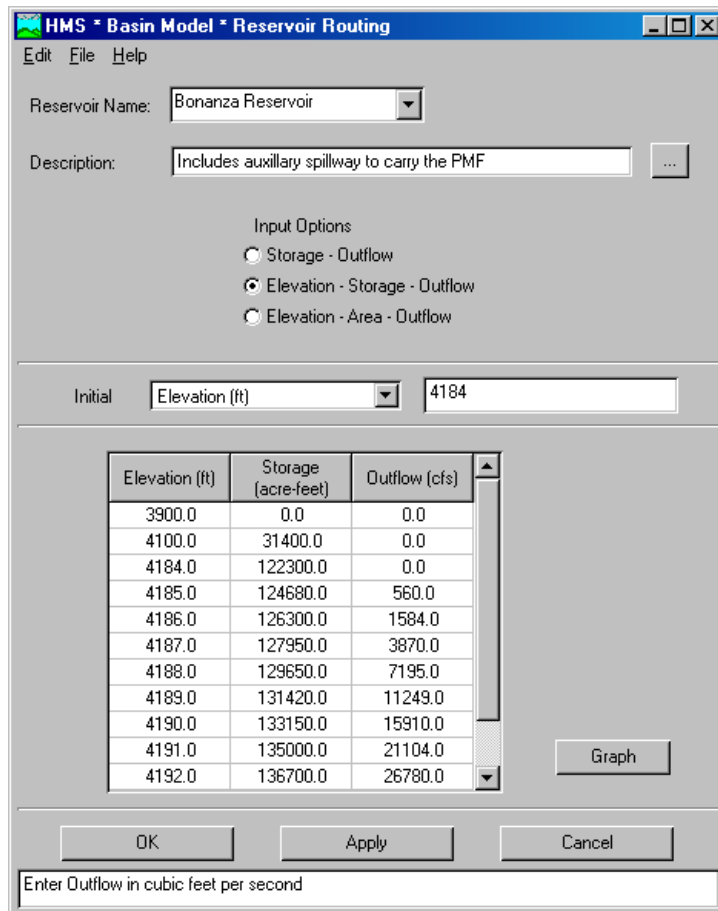| Elevation (ft)	Main spillway		Auxiliary spillway		Total discharge (cfs)
	Head (ft)	Discharge (cfs)	Head (ft)	Discharge (cfs)	
4,184	0	0		0	0
4,185	1	560	0	0	560
4,186	2	1,584	1	0	1,584
4,187	3	2,910	2	960	3,870
4,188	4	4,480	3	2,715	7,195
4,189	5	6,261	4	4,988	11,249
4,190	6	8,230	5	7,680	15,910
4,191	7	10,371	6	10,733	21,104
4,192	8	12,671	7	14,109	26,780

Figure 46.   *Reservoir data editor with spillway data*

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.

References

Chow, V.T., Maidment, D.R., and Mays, L.W. (1988). *Applied hydrology*. McGraw-Hill, New York, NY.

National Weather Service [NWS] (1977). *Probable maximum precipitation estimates, US east of 105th meidian, HMR No. 51*, Silver Springs, MD.

NWS (1981). *Application of probable maximum precipitation estimates, US east of the 105th meridian, HMR No. 52*, Silver Springs, MD.

U.S. Army Corps of Engineers [USACE] (1965). *Standard project flood determination. EM 1110-2-1411*. Washington, D.C.

USACE (1984). *HMR52 user's manual*. Hydrologic Engineering Center, Davis, CA.

USACE (1990). *Hydraulic design of spillways. EM 1110-2-1603*. Washington, D.C.

USACE (1991). *Inflow Design Floods for Dams and Reservoirs. ER 1110-8-2 (FR)*, Washington, DC 20314-1000.

World Meteorological Organization (1983). *Guide to hydrologic practices, Vol. II: Analysis, forecasting and other applications. WMO-No. 168*, 4th edition. Geneva, Switzerland.

C H A P T E R   7

Stream restoration studies

The Corps 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 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 Freshwater Initiative (Nature Conservancy, 2001).

Hydrologic engineering study objectives and outputs

Measures of achievement of stream restoration goals are not defined in Corps' 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 Corps' 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 Corps offices to restore aquatic habitats and mitigate development impacts:

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- Water Resources Development Act (WRDA) 1986, Section 206. This legislation directs the Corps to carry out aquatic ecosystem restoration and protection projects.

- WRDA 1986, Section 1135. This authorizes the Corps to modify existing project structures and operations to restore environmental quality. This subsequently was amended to include restoration project areas that are outside Corps project lands, but which were impacted by the project.

- WRDA 1974, Section 22. With this, Congress gave the Corps of Engineers 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 the Corps 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 EMs and ERs cited in earlier chapters.

Case study: Channel maintenance along Stirling Branch

Watershed description

Stirling Branch is a tributary of Deer Creek in a developing area in the western U.S. 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 3% in the upper reaches and about 1% 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 47(*a*), 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.

The local government has now reconsidered the wisdom of this channel modification and the maintenance, and has appealed to the Corps 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 47(*b*), with ground-level shrubs and trees of moderate density planted in the stream corridor. Corps 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.
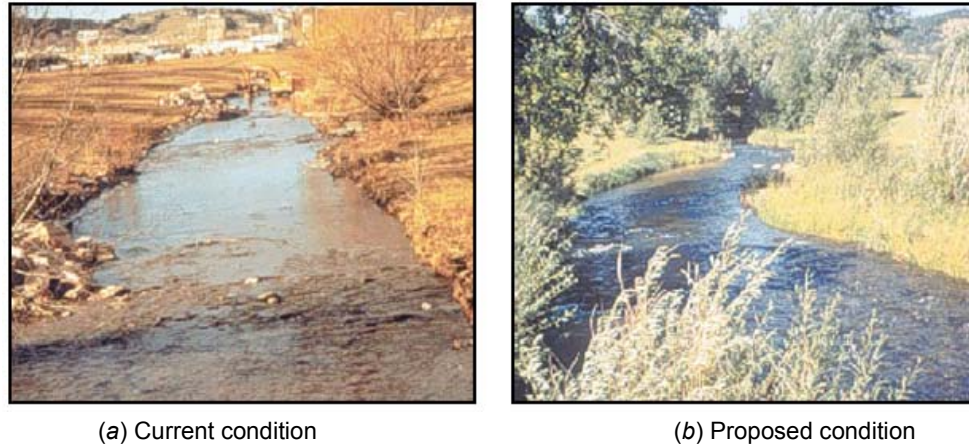
(*a*) Current condition   (*b*) Proposed condition

Figure 47.   Before and after photos of "restored" stream (Fischenich, 1999)

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 bankfull 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 stormwater 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 48(*a*), 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 48(*b*), 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 48(*c*), trees are combined with lower vegetation. In this case, velocity is retarded by both the undergrowth and the tree branches.

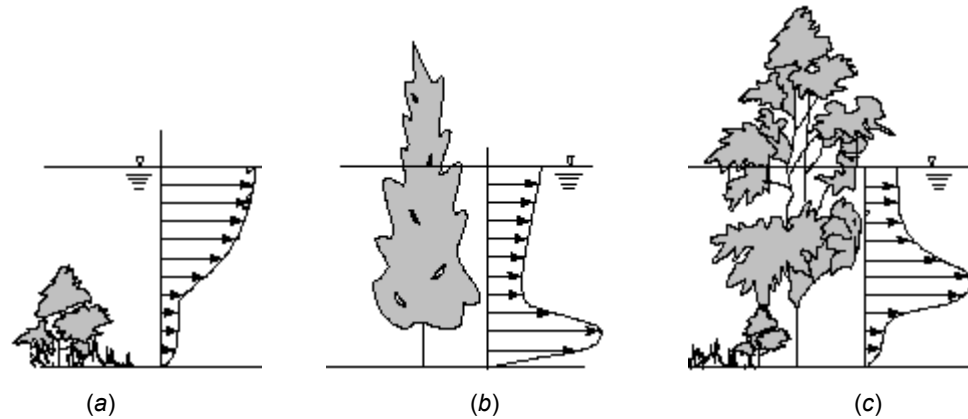(*a*)   (*b*)   (*c*)

Figure 48.   Illustration of velocity profiles with submerged and unsubmerged vegetation (Fischenich, 2000)

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 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 6 subbasins (primarily for convenience of modeling the channels). For each, the SCS curve number (CN) method was selected for runoff volume computation, and the SCS unit hydrograph (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 geographic information system (GIS) tools with coverages of land use and soil type.

The SCS UH lag was estimated for each subbasin as 60% 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 (Soil Conservation Service, 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 48 (*b*) and (*c*). To represent these, the analyst selected the 8-point Muskingum-Cunge routing method and entered geometric data appropriate for each case, as shown in Figure 49.
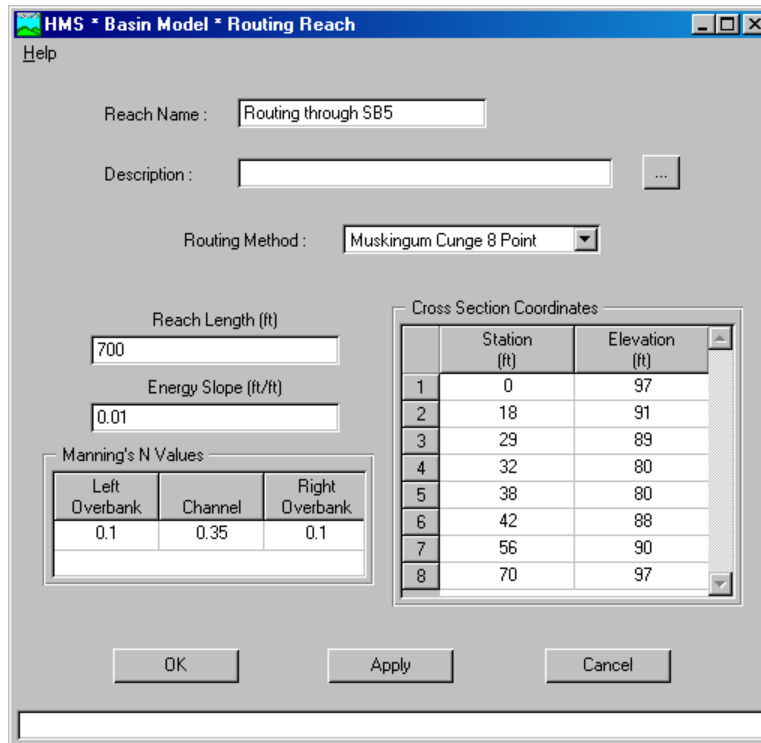
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Figure 49.   Routing reach schematic for 8-point Muskingum Cunge

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 nonsubmersed vegetation:

$$n = k_n R^{2/3} \left[\frac{C_d Veg_d}{2g} \right]^{1/2} \tag{7}$$

in which C_d = coefficient to account for drag characteristics of vegetation; Veg_d = vegetation density; and g = gravitational constant. Vegetation density is defined as:

$$Veg_d = \frac{\sum A_i}{AL} \tag{8}$$

in which A_i = area of vegetation below water surface, projected onto a plane perpendicular to direction of flow; A = cross-sectional area; and L = characteristic length. Flippin-Dudley, *et al.*, (1998) suggest that C_d can be estimated as:

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

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

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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, she 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 4 minutes was selected.

2. A meteorologic model was prepared for each of the 3 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 3 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 3 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.
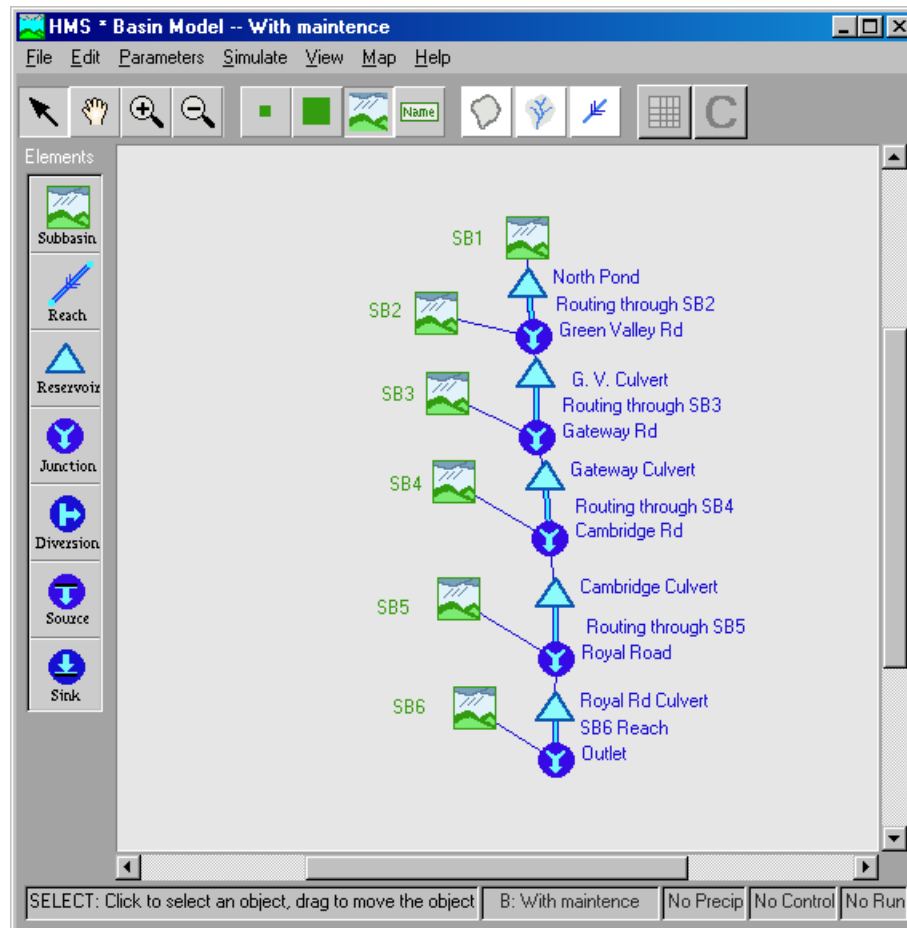
Figure 50.   Basin schematic of Stirling Branch

The analyst defined 9 runs, combining the 3 meteorologic models with the 3 basin models. Peak flows at the watershed outlet are shown in Table 33.

Table 33.   Peak flows of restoration alternatives, in 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, then using HEC-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.

References

Barnes, H.H. (1967). _Roughness characteristics of natural channels. Water-supply paper 1849._ U.S. Geological Survey, Washington, D.C.

Fischenich, C. (1996). _Hydraulic impacts of riparian vegetation: Computation of resistance. EIRP Technical Report EL-96-01_. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Fischenich, C. (1999). _Overview of stream restoration technology: State of the science, EMRRP Vol. 2, No. 3_, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Fischenich, C. (2000). _Resistance due to vegetation_, _EMRRP-SR-07_. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Fischenich, C. (2001). _Technologies for urban stream restoration and watershed management_, _EMRPP Bulletin No. 01_. U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Flippin-Dudley, S.J., _et al_. (1998). _Evaluation of flow-resistance equations for vegetated channels and floodplains. EMRRP Technical Report EL-98-2_. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

National Research Council (1992). _Restoration of aquatic ecosystems: Science, technology, and public policy_. Committee on restoration of aquatic ecosystems. Washington, DC.

Nature Conservancy (2001). _Freshwater initiative (www.freshwaters.org)._

Soil Conservation Service (1971). _National engineering handbook_, _Section 4: Hydrology_. USDA, Springfield, VA.

Soil Conservation Service (1986). *Urban hydrology for small watersheds, Technical Report 55*. USDA, Springfield, VA.