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	Engineering and Design HYDROLOGIC ENGINEERING ANALYSIS CONCEPTS FOR COST-SHARED FLOOD DAMAGE REDUCITON STUDIES	
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Foreword

Hydrologic engineering is a civil engineering discipline involving the analysis of water and its systems as it moves above, on, through, and beneath the surface of the earth. Water is a critical and integral element in planning and evaluating flood damage reduction measures and actions. For these studies, hydrologic engineers have a major role in defining the flood hazard, and in locating, sizing, and assuring the functional and operational integrity of the projects.

This document describes the study processes performed by U.S. Army Corps of Engineers hydrologic engineers for Federal flood damage reduction studies. The objective is to enable Corps staff, the cost-shared partners, and others involved in the planning process to gain a better understanding of the hydrologic engineering study scope, strategies, and methods of analysis. It is intended that with this better understanding, the study team participants will more clearly define and grasp the choices available for the conduct of the hydrologic engineering analysis and will reach a mutual agreement on the study requirements.

This document is applicable to HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having civil works responsibilities.

FOR THE COMMANDER:

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Engineering and Design
HYDROLOGIC ENGINEERING ANALYSIS CONCEPTS FOR COST-SHARED FLOOD DAMAGE REDUCTION STUDIES

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Chapter 1
Introduction

1-1. Purpose

a. This publication describes study processes performed by U.S. Army Corps of Engineers (USACE) hydrologic engineers for Federal flood damage reduction projects. The goal is to enable Corps staff and cost-share partners to gain an understanding of hydrologic engineering procedures, including the study scope, strategies, and methods of analysis. With a common understanding, the study team members can clearly define and grasp the choices available for performing the hydrologic engineering study and arrive at a mutual agreement on study requirements.

b. Appropriate references to other pamphlets, manuals, documents, and texts are included if more detailed explanations are desired. References shown throughout this document may be found in Appendix A.

1-2. Applicability

This pamphlet applies to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having civil works responsibilities.

1-3. Overview of Corps Flood Damage Reduction Studies

The Corps undertakes studies of water and related land resource problems from directives or authorizations issued by Congress. Congressional authorities are contained in public laws or in resolutions. Study authorizations are either for specific studies or for standing program authorities, such as the continuing authorities program. The focus of the studies is to determine whether a Federal flood damage reduction project should be recommended in accordance with Army policies. Corps studies for planning, engineering, and design of flood damage reduction projects are predicated on these legislative requirements and institutional policies (ER 1105-2-100 and EP 1105-2-10).

a. Planning studies.

(1) Project planning studies are conducted in two phases. The first phase, resulting in a reconnaissance report, is fully funded by the Federal Government. It normally requires 12 months for completion, determines if there is a Federal interest (benefits of the project exceed the costs for at least one alternative) and if there is non-Federal support (a local sponsor willing to cost-share). The hydrologic engineering analysis for the reconnaissance phase should establish existing condition hydrology if adequate time and funding are available. If the reconnaissance report is favorable, an Initial Project Management Plan is prepared to detail the time, cost, and work schedule necessary to complete all facets of the subsequent feasibility study. A Feasibility Cost-Sharing Agreement is signed with the local (non-Federal) sponsor.

(2) The feasibility phase is cost-shared equally between the Federal Government and non-Federal sponsor. It may take up to 4 years to complete and results in recommendations to Congress concerning Federal participation in reducing the flood problem identified in the study. This report contains the detailed hydrologic analysis necessary to determine the severity of the existing flood problem, and to evaluate the success of various alternatives in alleviating the problem. Detailed economics, plan formulation, and a baseline cost estimate for the recommended plan are also necessary in this phase. The feasibility report typically recommends the project which provides the maximum net benefits. The Project Management Plan is prepared late in the feasibility study to determine time and funding requirements for the detailed engineering design and construction phases following feasibility. A positive recommendation for Federal participation results in the project proceeding into preconstruction engineering and design (PED). The cost-sharing requirements for the recommended project and items of non-Federal sponsor cooperation are to be included in the feasibility report. Additional information concerning feasibility investigations is referenced in ER 1105-2-100 and ER 1110-2-1150.

b. PED studies.

(1) The PED phase continues design efforts following the feasibility study and encompasses the more detailed construction planning and engineering necessary for building the project. The major items resulting from the PED phase are design memoranda and plans and specifications.

(2) A design memorandum (DM) is the primary document developed in the PED phase. Detailed engineering and design are documented during preparation of the DM leading to construction of the project. The DM emphasis is on areas of structural analysis, soils testing and exploration, real estate analyses, cost engineering, etc. Where a project is large or has several major components, more than one design memorandum may be necessary.

(3) Following completion and approval of a DM, plans and specifications are prepared, which allow the project to be bid and constructed. For most projects, the PED phase is expected to be completed within 2 years.

(4) For projects where considerable time has elapsed since completion of the feasibility report or where conditions have changed enough to require project reformulation, a general design memorandum (GDM) may be necessary. The intent is to provide sufficient engineering analysis during feasibility, along with prompt and continuous funding into the PED phase, to preclude the need for a general reevaluation report. The feasibility report, along with the engineering appendices, should allow a smooth progression through PED for most projects. Additional information on the PED phase is referenced in ER 1110-2-1150.

c. Construction engineering and design. Once preconstruction engineering and design are complete, any remaining engineering and design are accomplished concurrent with construction activities. This phase includes the design memorandums and plans and specifications to construct any remaining project components. Construction of each project component occurs after completion of plans and specifications for that component.

d. Continuing authority studies. These standing study and construction authorities are conducted in a two-phase process. Additional information on continuing authority studies is available in ER 1105-2-100.

1-4. Hydrologic Engineering

a. Hydrologic engineering is a critical technical element in the planning of flood damage reduction measures and actions. It is a civil engineering discipline involving the analysis of water and its systems as it moves above, on, through, and beneath the surface of the earth as defined by the hydrologic cycle (See Figure 2-1). Hydrologic engineers have a major participatory role in defining the flood hazard, locating and sizing flood damage reduction projects, and determining and assuring the functional and operational integrity of the project.

b. Hydrologic engineers utilize data such as precipitation and streamflow in the planning, design, and operation of projects. Analysis techniques focus on determining the magnitude and frequency of hydrologic events (precipitation and streamflow) at locations of interest. The analysis approaches generally involve relating known measurements of these phenomena to study areas having little or no measured data. The techniques used include: information transfer, simplified methods, statistical computations, and computer program models of the hydrologic systems.

c. To understand the data requirements and the analytical approaches applied, one must grasp the basic concepts of flood analysis and data needs. Chapter 2 describes these processes. Subsequent chapters define the analytical methods used by the Corps to perform flood hazard analyses for with- and without-project conditions.

Chapter 2
Data Needs and Hydrologic Processes of Floods

2-1. Overview

a. Hydrologic cycle. Water occurs on, in, and over the surface of the earth in many places, forms, and phases. The transformation from one phase to another and motion from one location to another are referred to as the hydrologic cycle. Major elements of the hydrologic cycle are shown in Figure 2-1.

b. Runoff. The process may be conceptualized as starting with precipitation occurring on the land surface. A portion of the precipitation is lost to evapotranspiration, infiltration, depression storage, and interception. The portion that is not lost becomes precipitation excess or runoff as:

Precipitation - Losses = Precipitation excess or runoff

c. Drainage systems. The surface runoff enters small hillside gullies and ditches, flows to brooks and creeks, and then to rivers that flow into the oceans. These systems, as shown by Figure 2-2, consist of a network of flow conveyance channels that occupy the lowest part of the landscape. The ridge of the land surface, or rim separating runoff networks, is called the drainage divide. The area of the land that encloses the divide may be referred to as the drainage area, watershed, or catchment of the stream.

d. Runoff hydrograph. The runoff from the watershed that occurs over time at the watershed outlet is a runoff hydrograph. Figure 2-3 shows a runoff hydrograph from a watershed with ordinates of discharge versus time. The runoff hydrograph enters the main stream channel, is added to the base flow (flow existing without the rainfall excess occurring) in the channel, and is combined with other runoff hydrographs as it is translated through the main stream system. The translation of the combined hydrographs through the stream system is also called flood routing.

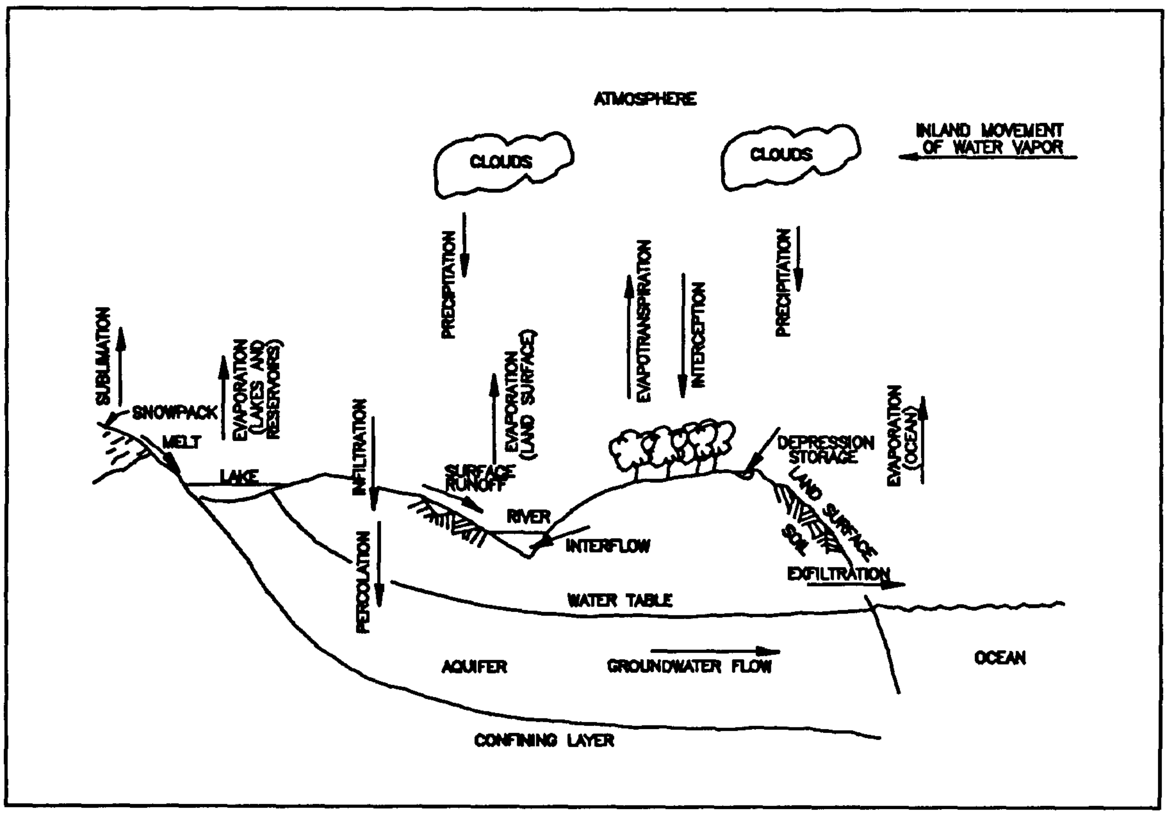


Figure 2-1. The hydrologic cycle

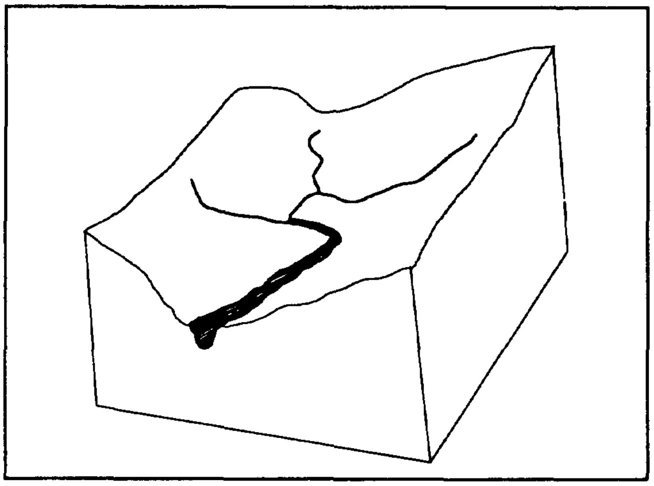


Figure 2-2. Drainage basin

e. Organization. The primary interests are the peak hydrograph discharge, the corresponding water level reached in the channel and overbank, and the frequency with which specific stages are reached. Paragraph 2-2 concentrates on developing the hydrograph and peak discharge, paragraphs 2-3 and 2-4 on water level determination, and paragraph 2-5 on frequency analysis.

2-2. Precipitation Runoff Relationship

a. Precipitation. Precipitation is derived from atmospheric moisture, resulting primarily from evaporation from the ocean. The predominate forms of precipitation are rain and snow, with hail, fog, drizzle, sleet, etc. being less important. The form of precipitation at the earth's surface is influenced by other climatic factors such as wind, temperature, atmospheric pressure, and humidity. Geographic factors such as latitude, altitude, topography, and location of land and water surfaces also influence the nature and amount of precipitation. The primary form of precipitation that causes runoff and flooding is rainfall, with melting snow also a contributor in some regions. Spatial extent, time variations, and intensity are important factors contributing to the runoff process. Areal distribution of precipitation is important and is highly correlated to the time history of runoff.

(1) Rainfall measurement. Rainfall intensities are measured by rain gages, which are either manually read or mechanically recorded. Manual gages are relatively inexpensive to install and read; however, rainfall information is normally available only in 24-hr increments. For most watersheds, rainfall intervals of less than 24 hr are necessary to adequately define the rainfall effects on the runoff hydrograph. Automated recorders are considerably more expensive, but can give rainfall intensities for increments as small as 5 minutes, necessary for small urban catchments. Figure 2-4 shows an automated precipitation recording gage. The National Weather Service (NWS) maintains a network of both types of gages throughout the United States; however, this network often has only limited data for a specific watershed. Small, urban watersheds may require the installation of one or more rainfall recorders to give site-specific information for a study area.

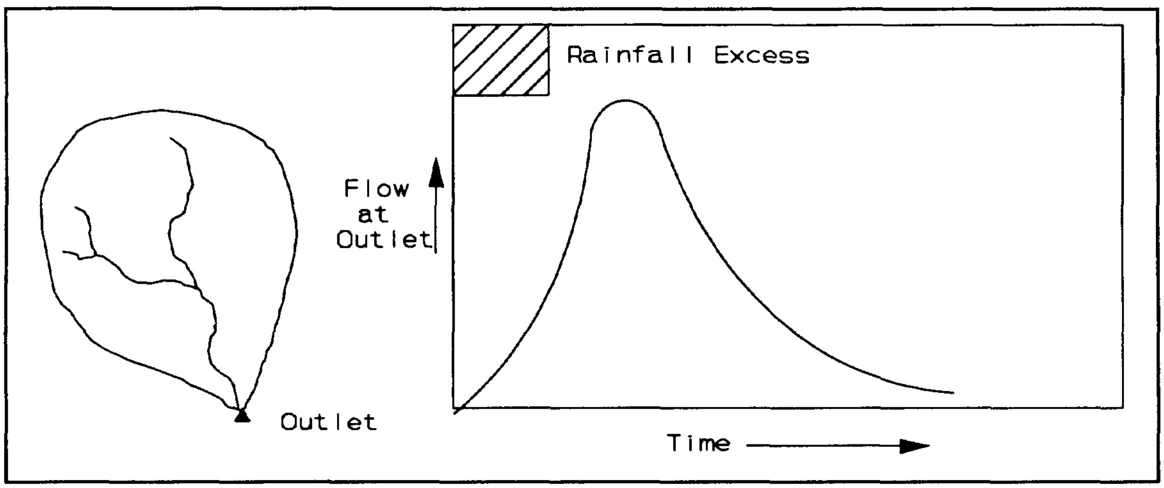


Figure 2-3. Runoff hydrograph

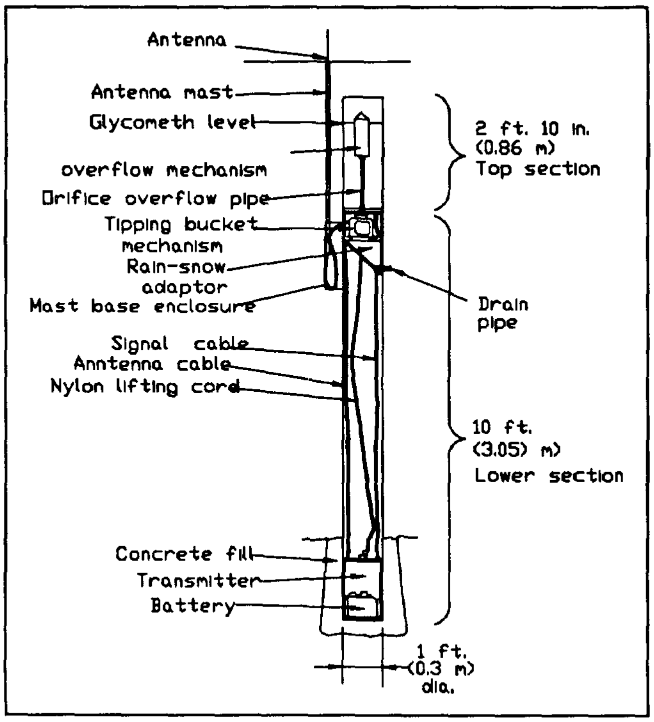


Figure 2-4. Automated rain gage

(2) Snowpack measurement.

(a) Snowfall is that part of precipitation which occurs as ice crystals. The aerial extent, water equivalent, depth of the snowpack, and how fast it melts contribute to the runoff process. The influence of snowfall on flooding is more important in northern and mountainous climates than in other sections of the United States.

(b) The snowpack depth is measured either manually, or by automated means. Manual measurement usually involves catching the snowfall in a cylinder, or cutting a sample from the snowpack, and then melting the collected snow for equivalent water content. Automated means can be used in remote areas and often consist of a "pillow" which records the increasing weight of the pack with time. Air temperature at the snowpack's surface is also necessary to predict the rate of melting, and the corresponding water excess to the streams.

b. Losses. Losses to precipitation falling on the earth and runoff into stream channels include evapotranspiration (evaporation from the ground surface and through foliage), depression storage (surface irregularities or "puddles"), interception (rainfall coating foliage), and infiltration (movement or transmission of surface water into the soil). Losses from evapotranspiration for flood events are generally considered negligible. Interception and depression storage losses depend on the surface topography and foliage of the system, but remain somewhat constant from event to event. Infiltration is the dominant source of losses during a flood event.

(1) Infiltration.

(a) Infiltration is a complex process involving the conceptual sequences of surface entry, transmission or percolation through the soil, and depletion of storage capacity of the soil. The infiltration rate decreases as the soil becomes saturated, thus resulting in greater runoff. Infiltration capacity can also significantly change over time due to development effects on the land surface.

(b) The major factors affecting infiltration are antecedent moisture conditions, land cover, and soil type. Soils and land use cover vary spatially over the watershed, whereas antecedent moisture conditions vary from event to event. Land use cover may also vary seasonally (vegetal cover) or over a period of time (urbanization). Information on soil type and land use is collected to aid in the estimate of infiltration losses.

(c) The effects of antecedent moisture conditions, soil type, and land use cover are conceptually depicted by Figure 2-5. The buckets represent the storage capacity of the soil, which becomes smaller when saturated, as shown by Figure 2-5a. In Figure 2-5b, the soil characteristics were changed to demonstrate the transmission variability of different soils. In Figure 2-5c, the surface entry of the soil has been reduced because of urbanization (imperviousness) of the land surface (USACE 1981).

(2) Infiltration measurement. Although many attempts have been made to measure losses directly, only limited success has been achieved. Losses on a specific watershed are usually inferred by measuring basin average rainfall using one or more gages (input to the basin), measuring the runoff hydrograph at a stream gage (output from the basin, or rainfall excess), and determining losses by subtracting rainfall excess from the rainfall. These losses could be distributed over the time of the storm, determining an average loss per time period to use for other rainfall events for which no discharge data are available.

c. Discharge hydrographs. Discharge hydrographs are generally considered to have two parts, direct runoff and base flow. Direct runoff is the rainfall excess received from recent storm runoff, while base, or groundwater, flow occurs regardless of the storm runoff. Base

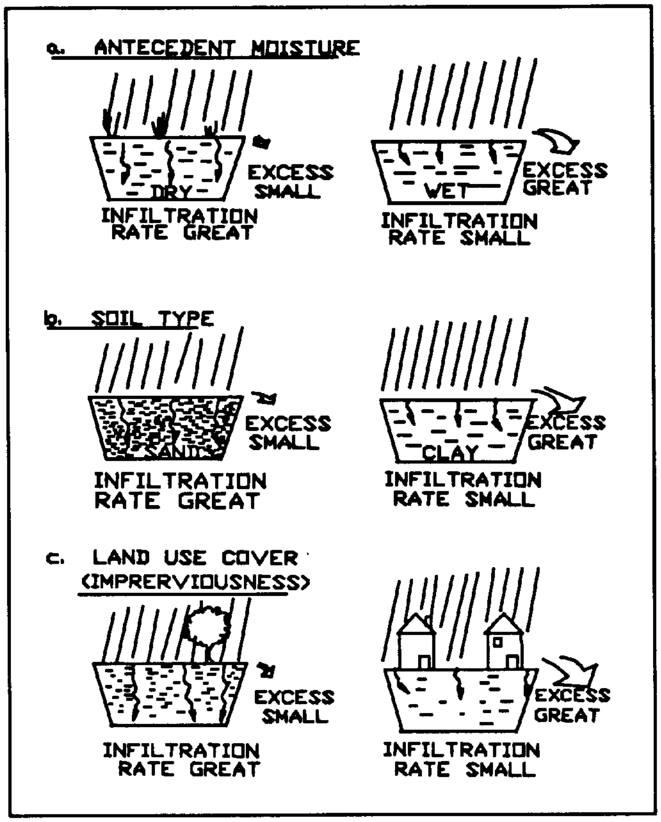


Figure 2-5. Infiltration losses

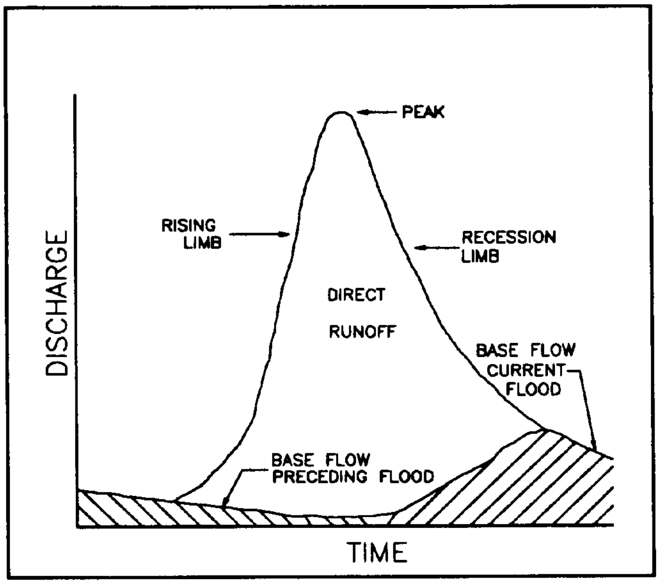


Figure 2-6. Discharge hydrograph features

flow is a relatively small part of the overall hydrograph and is important primarily on large watersheds. Base flow and direct runoff are shown on Figure 2-6. Hydrograph characteristics such as peak discharge, time to peak, and volume of runoff are based on the shape of the hydrograph. In turn, the shape is dependent on precipitation patterns, losses, and basin characteristics.

(1) Intensity patterns. Time-intensity patterns of rainfall excess can have a significant effect on the peak, shape, and duration of the hydrograph. Figure 2-7 shows examples of the effects of various intensity patterns. Changes in storm intensity must last for hours or days to cause distinguishable effects on the hydrograph for a large watershed. For small basins, clearly defined peaks in the hydrographs may be caused by a few minutes of intense rainfall excess.

(2) Characteristics affecting hydrograph. Precipitation affects runoff directly only if the physical characteristics of the watershed are relatively constant. However, these characteristics are often not uniform within a

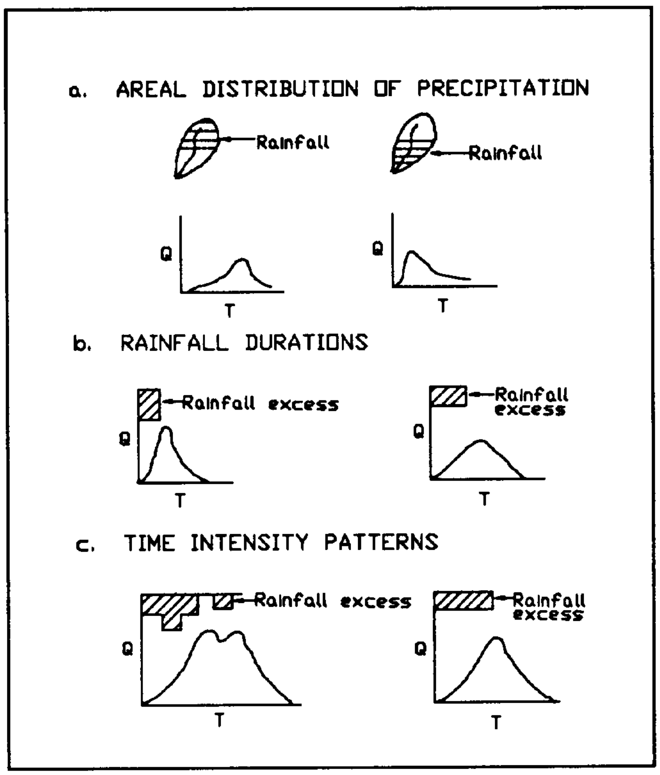


Figure 2-7. Rainfall characteristic effects on runoff hydrographs

watershed or between watersheds. Basin characteristics affecting the hydrograph include:

- Size of the watershed.
- Shape of the watershed.
- Length of the main channel.
- Land and channel slopes.
- Roughness of land and channels.
- Drainage density.
- Valley storage.

(3) Effects of physical characteristics. The effects of the physical characteristics of a watershed on the runoff hydrograph are conceptualized in Figures 2-8 and 2-9. As the runoff enters the main channels, the volume, shape, peak flow, and timing all affect the magnitude of flow. These characteristics are defined primarily through field reconnaissance and map analysis.

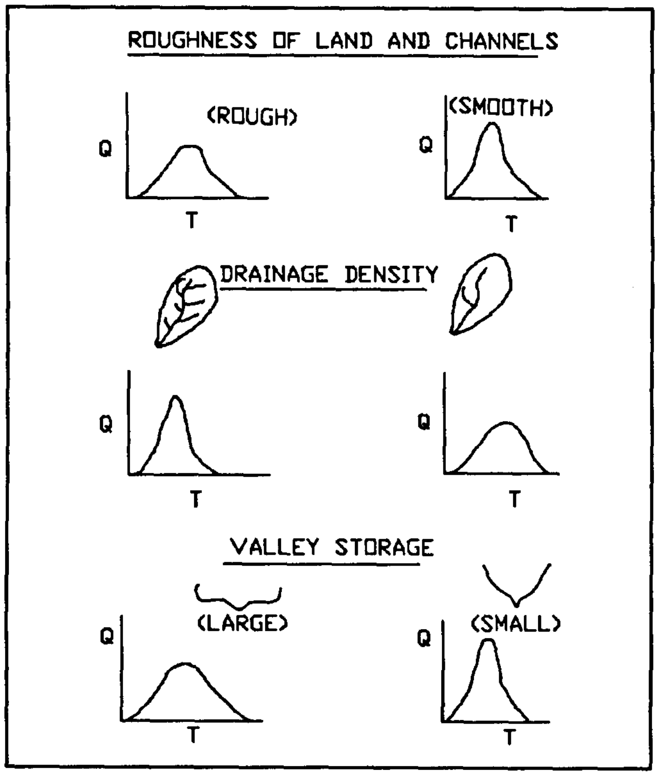


Figure 2-8. Effects of basin characteristics on runoff hydrographs

d. Hydrograph measurements.

(1) Stream gages. Runoff hydrographs and direct runoff from a storm may be determined directly by measurement at a stream gage. A stream gage could be as

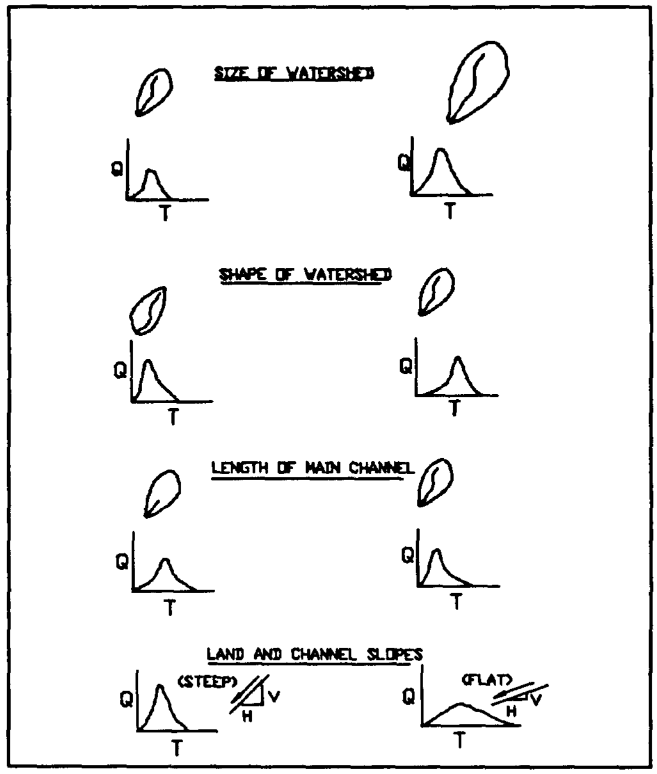


Figure 2-9. Additional effects of basin characteristic on runoff hydrographs

simple as a graduated board read once a day, or as sophisticated as an automated gage recording in 5-minute intervals and reporting by satellite telemetry. The cost of these installations varies significantly.

(2) Recorders. A manual recorder is quite inexpensive, but gives only stage, or water level readings, usually once per day. These gage records produce a stage hydrograph, but no information on discharge. A continuous recorder measures water level at predetermined intervals, providing a continuous trace of water level changes. Water levels are converted to discharge by periodic physical measurements of the stream cross-sectional area and river velocity, made with current meters. These discharge measurements are made once a week to once a month for normal flows, and as often as possible during floods. Over time, these measurements can define a relationship between water level and discharge, allowing one to estimate discharge based on the water level.

(3) Gage installation. Because of the expense, stream gages are not as numerous as one might wish. When no gages exist in the study watershed, it may be necessary to install one or more for a limited data

collection program. This activity must be accomplished well in advance of the hydrologic analysis and other general study activities. These data are supplemented with additional information derived by methods discussed in later chapters.

2-3. Channel Characteristics

a. General.

(1) Channel systems. Most streams flow within a channel system bordered on one or both sides by a relatively flat area called a valley or floodplain. For in-channel flow, the velocity is less nearer the bottom and sides than it is nearer the center and surface due to boundary friction. For straight channel reaches with relatively constant dimensions, the flow approaches one-dimensional flow in the downstream direction. Channel bed deposition and scour occur depending on channel slope, bed material, and velocity of flow.

(2) Flow patterns. Channels, however, are seldom straight for long reaches, and channel bends and curves have an important effect on the flow. As the stream enters the bend, the flow near the surface tends to move towards the concave bank and the flow near the bottom moves toward the convex bank, as shown in Figure 2-10. This flow pattern results in erosion on the concave (outside) side of the bend and deposition on the convex (inside) side of the bend. The flow pattern is somewhat spiral-shaped, or three-dimensional, in its movement.

(3) Meandering. The stream is thus constantly moving laterally or meandering in its natural state, with deposition occurring on one side and erosion on the other. Meandering occurs slowly during normal flows, with the rate increasing considerably during floods. This process, plus overbank deposition of sediments during floods, creates the floodplain shown in Figure 2-11.

(4) Alteration of flow patterns. In addition to bends, other alterations to flow patterns are caused by changes in flow, in the cross-sectional geometry of the channel area, and in the boundary roughness of the channel area. These alterations can cause eddies, backwaters, drawdowns, and jumps. Changes in cross-sectional areas may result from expansions, contractions, and obstructions to the flow area. For flow within banks, these changes may occur naturally, or from obstructions, such as boulders and debris. The changes may also result from man's channel construction, bridges, pipeline crossings, and numerous other modifications.

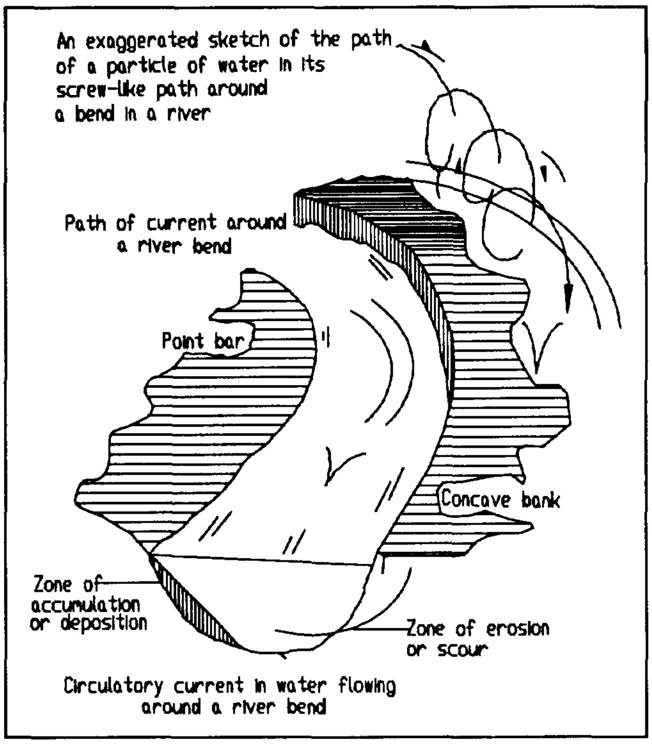


Figure 2-10. Channel flow patterns

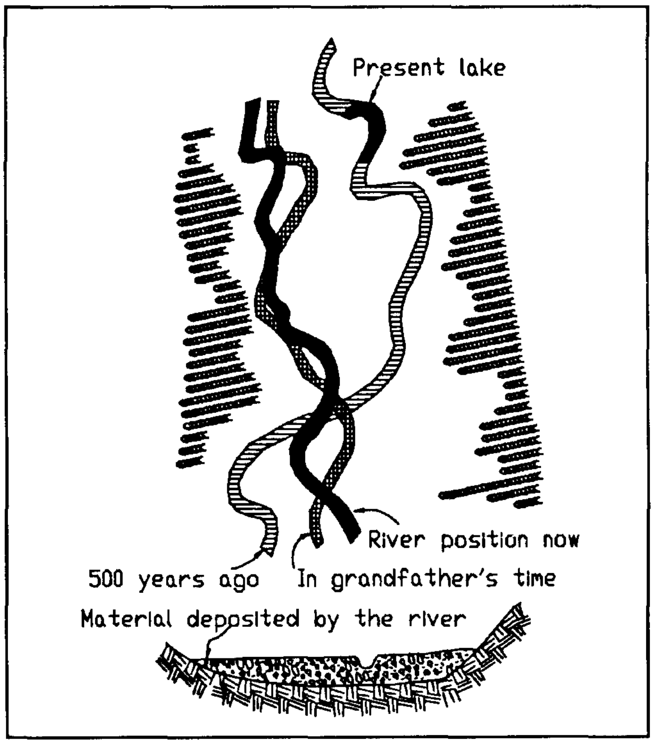


Figure 2-11. Floodplain development concepts

(5) Channel capacity. All of these physical effects result in a specific capacity for the channel, which can vary somewhat along the reach of the stream. Channel capacity is an important variable. Typically, damage occurs when the channel flow capacity is exceeded.

b. Field measurements.

(1) General. Measurements of stage and discharge have been previously discussed in paragraph 2-2*c*. Changes in flow patterns are largely determined from field surveys of the channel and overbank geometry throughout the study reach. The location of survey data is based on examination of the stream reach and determining where significant changes in channel and overbank geometry occur. Bridge obstructions are particularly significant.

(2) Sedimentation. Where sedimentation is important, measurements of sediment flow as well as water discharge are needed. Data collection and analysis at sediment sampling sites are expensive, but necessary to address the existing sediment regime and how various flood damage reduction projects may affect it. Suspended sediment samples are collected at a discharge site at similar intervals as discharge measurements. Over time, these measurements produce a relationship of water discharge to sediment discharge, so that knowledge of the stage can allow estimates of both water and suspended sediment discharge. An estimate of sediment moving along the bed of the stream (bed load, or unmeasured load) is also necessary for complete definition of the sediment load for a given discharge.

2-4. Flood Characteristics

a. General. Flooding is a natural characteristic of a stream system and can be considered an overbank flow. It occurs when water in the stream system exceeds the channel capacity, causing an overflow onto the valley or floodplain. Flood damage is the destruction or loss of property caused by water that cannot be carried within the normal channel. Flooding is usually the result of rainfall excess or snowmelt, but occasionally can be from failure of engineered structures.

b. Flow characteristics. As the flood hydrograph moves through the stream system, effects on flow characteristics can be dramatic. High velocities in the channel may cause bank erosion and scour to the bed, increasing the sediment load, which is subsequently deposited in areas of slower velocities, such as the floodplain. Severe floods have produced major changes in channel and overbank characteristics.

c. Movement of flood hydrograph. The movement of the flood hydrograph through the stream system affects the hydrograph shape due to the travel time of the flood and to the natural storage in the floodplain, or to man-made storage such as in reservoirs. As the flood waters increase in height and flow into the overbank areas occurs, the storage in the overbanks tends to delay and reduce the peak as shown in Figure 2-12.

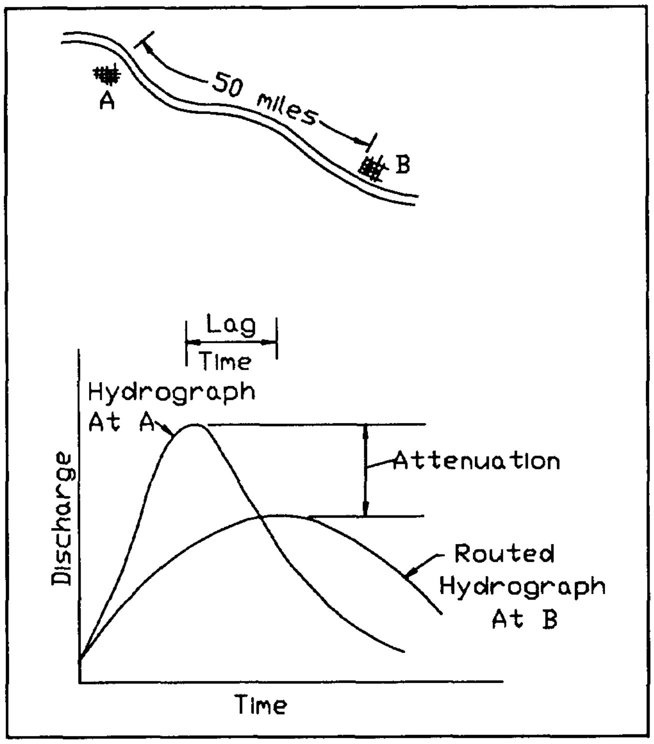


Figure 2-12. Effects of flood hydrograph translation

d. Analysis requirements. Analysis of flood movement or routing may include the determination of the peak stage or elevation at all key points. Usually peak stages are determined separately through river hydraulic studies. Hydrology, therefore, normally encompasses the development of surface water runoff, hydrograph combination, and routing to determine peak discharges at all key locations. Hydraulic analysis for flood damage reduction studies utilizes these discharges to determine the peak water surface elevation. How often the flood occurs (frequency) must then be determined.

2-5. Frequency Analysis

a. General. Frequency forms the third primary analysis requirement, along with water level and discharge. The determination of economic benefits of a project requires a knowledge of how often flooding occurs at various flood levels. This requirement is met through the analysis of stage- or discharge-frequency curves for conditions of interest.

b. Methods of analysis. This analysis may be developed by statistical methods if a long-term hydrologic record exists at a stream gage in the study reach. Typically, however, long-record data are scarce for most hydrologic analyses. Even if such a record is present, other locations, having limited data, also must be evaluated for frequency. Therefore, frequency determinations usually consist of the application of hypothetical storms of specified frequency (10-, 2-, and 1-percent annual chance exceedance, etc.) to a hydrologic model of the watershed to determine discharge-frequency relationships at all desired locations.

c. Hydrologic models. Hydrologic models are often calibrated so that observed rainfall frequency approximately corresponds to discharge frequency. Loss rates are usually adjusted, based on judgement, to reflect the severity of the hypothetical floods.

Chapter 3
Flood Studies

3-1. Overview

a. Development of work plan. The hydrologic engineer must develop a work plan appropriate to the flood problem being studied and the type of flood damage reduction alternatives under investigation. The flood hazard must be defined to determine the tangible damage resulting, both for present conditions in the watershed and for a future time period, if significant changes in the watershed may occur. A level of detail commensurate with the type of analysis must be determined, a method of formulation and evaluation of the proposed flood damage reduction alternatives adopted, and the consequences (both positive and negative) addressed.

b. Hydrologic engineering. This chapter describes, in general terms, the hydrologic work necessary for a flood study. Interested readers may review ER 1110-2-1460 for additional information on hydrologic engineering management and for hydrologic engineering required for feasibility studies, respectively. Additional information on hydrologic engineering and other engineering disciplines in the feasibility and PED phases is referenced in ER 1110-2-1150.

3-2. Definition of the Flood Hazard

The study process and how the plan formulation and evaluation evolve must be defined to provide the hydrologic data needed by other disciplines. The method of analysis, level of detail, time requirements, and format of the hydrologic information must be commensurate with the needs of the study team, including the cost-sharing partner.

a. General. Most flood studies require the definition of a stage-frequency relationship at key locations in the watershed, and how these relationships change in time, both with and without various flood damage reduction projects. Flood frequencies ranging from a 50-percent chance exceedance event through the Standard Project Flood or a 0.2-percent chance exceedance frequency event[1] are selected to define a full range of frequency events. Existing, or base (start-of-project operation), conditions form one "snapshot" of land use conditions to evaluate. At least one additional time period, usually 20-50 years in the future, based on available land use planning information, is chosen to form a second time period. If land use differences between the two times are great, additional periods may be interpolated. Similarly, additional periods could be extrapolated. The same two times are also used to evaluate the changes in the watershed's flood hydrology caused by the various flood reduction measures evaluated. Figure 3-1 illustrates frequency curves at a location undergoing continuous urbanization, resulting in changes to the relationship.

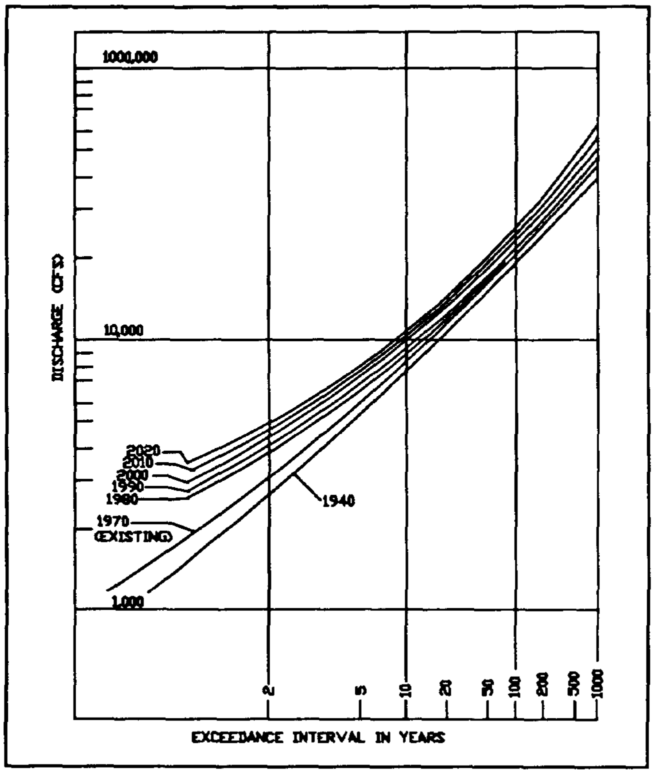


Figure 3-1. Frequency curves developed for an urbanizing area

b. Hydrologic information. The hydrologic engineering information needed could include the following:

(1) Discharge hydrographs.
(2) Peak discharge frequency.
(3) Runoff volume frequency.
(4) Water surface profiles.
(5) Flood inundation boundaries.
(6) Flow velocities.
(7) Warning times.

[1] Exceedance frequency is percent chance an event may occur in any given year. An event with a 0.2-percent chance exceedance frequency will occur once every 500 years, on the average, but can occur in any given year.

(8) Duration of flooding.
(9) Sediment deposition and erosion quantities.
(10) Operational performance of projects (amount of flood reduction).

Of the above information, accurate flood inundation determinations have the most impact on project evaluation. The extent and depth of flooding for with- and without-project conditions result in the estimate of flood damage reduction benefits, which are the basis for determining the economic feasibility of flood reduction projects. For agricultural flood studies, duration and time of year of flooding are important. The quality of the hydrologic work, as well as the survey and mapping information, can significantly affect the determination of project feasibility.

c. Field presence. The hydrologic engineer must spend time in the field throughout all phases of the analysis, from the reconnaissance through the actual construction. A field presence is required to gather data needed for the various phases of the study and to maintain continuous contact with local interests involved with the proposed project. Credibility is quickly lost when the engineers involved in the project recommendations have spent little or no time in the study area. Field visits should in many cases include other members of the study team and the local sponsor. The hydrologic engineer's field presence is needed for:

(1) Determination of topographic and survey needs.

(2) Determination of high-water marks and times of flood peaks by interviewing local residents and researching newspaper files.

(3) Obtaining the characteristics of flood events, including debris and sediment problems, flood dates, velocity patterns, and changes to the stream and watershed since historic floods have occurred.

(4) Estimation of friction values (Manning's n) for the channel and floodplain.

(5) Identification of obstructions to flood flow (bridges, dams, logjams, levees, road embankments, etc.), and historical floods which caused overtopping of these obstructions, including how often they occurred and whether debris or ice was a factor.

(6) Operation procedures for existing structures (dams, pumping plants, drains, frequency of channel dredging, etc.).

(7) Identification of the type and location of potential flood loss reduction measures, and any constraints on these measures (relocations, environmental damage, etc.).

d. Level of detail. Most of the hydrologic engineering effort is concentrated in the reconnaissance-phase and feasibility-phase studies.

e. Reconnaissance-phase study. The effort in the reconnaissance phase emphasizes the use of existing data, primarily due to the short duration of the study. Much information is obtained from local residents and officials, existing studies of the watershed, from measured and readily available data, and from field reviews and engineering judgement. If time and funding are available, it is desirable to establish the existing condition hydrology and flood profiles to avoid major changes in the feasibility phase. The reconnaissance-phase study evaluates several potential alternatives to determine if at least one is economically justified. If economic justification exists and there is a local sponsor willing to cost share, the study continues into the feasibility phase.

f. Feasibility-phase study. The majority of hydrologic engineering work is performed in this phase. The analysis must be sufficiently rigorous so that the project recommended in the feasibility report is essentially what is constructed, after detailed PED is completed. The hydrologic engineer, working closely with the study manager, economist, cost engineer, and other members of the Interdisciplinary Planning Team (IPT), completes the with- and without-project evaluations so a plan that maximizes net benefits is identified at completion of feasibility planning. This end result requires a continuous exchange of technical information among the various disciplines as follows:

(1) Technical data to the hydrologic engineer:

(a) Survey and mapping information.

(b) Maps showing land use, soil types, vegetation, storm sewer layouts, bridge plans, and other information from local agencies.

(c) Rainfall information from the NWS or other agencies.

(d) Stage, discharge, and sediment information from the U.S. Geological Survey (USGS) or other agencies.

(e) Potential alternatives to analyze, in conjunction with the study team and the local sponsor.

(2) Hydrologic data to the study team:

(a) Specification of topographic data needed by surveyors.

(b) Stage-frequency relations for without-project (base conditions), future without-project, and with-project conditions.

(c) Effects (reduced flooding) of each flood mitigation component to the study manager.

(d) Component capacities or dimensions to the designer and cost engineer.

(e) Velocity, sediment, duration, depth, and other information to the environmental specialist and permit specialist.

(f) Flood inundation boundaries with and without a project to mapping and to the real estate specialist.

(3) The information is furnished via an iterative process, leading to the refinement of the recommended project.

g. Design memoranda.

(1) Detailed hydraulic design. The emphasis in this phase is on the detailed hydraulic design aspects, because no additional plan formulation, economics, etc. should be necessary. The hydrologic engineer is more involved in refining the detailed design of the project components. The overall component capacities, general design, etc. are held relatively constant from that recommended in the feasibility report. For instance, the feasibility report may have recommended 5 miles of channel modifications having specified channel dimensions. The design memoranda would refine these dimensions to fit the channel through existing building and bridge constraints; perform detailed hydraulic design of tributary junctions, bridge transitions, drop structures, and channel protection; and conduct detailed sediment transport studies to identify operation and maintenance requirements, and other hydraulic design aspects.

(2) Additional activities. Additional topographic site surveys and subsurface information are normally obtained in this phase so that structural design, geotechnical analysis, cost engineering, and other activities can be finalized. The hydraulic design is often updated to reflect changes in analysis parameters prior to completion of detailed design. Additional information on the design memoranda phase is referenced in ER 1110-2-1150.

h. Construction and operation. Unforeseen problems during construction frequently involve further modification and adaptation of the hydraulic design for onsite conditions. Similarly, most projects require detailed operation and maintenance manuals and hydrologic engineering information can be a critical part of these manuals. The operation of reservoirs, pumping stations, and other flood mitigation measures can require considerable hydrologic operation studies to determine the best operating procedures. Post-construction studies are necessary for most projects. These studies monitor sediment deposition and scour caused by the project, ensure that adequate hydrologic design capacity is maintained, monitor the correctness of the data used in analyzing the project, and estimate the remaining useful life of the project.

3-3. Formulation and Evaluation of Solutions

a. Methods of solution. Appropriate application methods of solution are required to provide the necessary information to the study team and to evaluate both positive and negative aspects of the project. Simple versus complex methods and a frequency approach versus a period-of-record approach are considered, depending on the phase of the study and the type of flood damage reduction measures being evaluated. A simple regression equation giving peak discharge, knowing drainage area and slope, may be acceptable for a reconnaissance-phase effort to size a channel, but is inadequate for determining adverse effects downstream.

b. Investigation sites. Locations where hydrologic information is needed must be identified. This process must include the study team's requirements as envisioned at the time of the determination. In general, these points include:

(1) Locations of tributary junctions with the main channel, to evaluate significant flood flow changes.

(2) Stream gages, to calibrate model output to actual data.

(3) Points along reaches where flood damage reduction measures are to be evaluated.

(4) Locations in the watershed where land use, soil type, etc. change significantly.

(5) Damage centers, to compute damage with and without a project.

Figure 3-2 illustrates how a watershed could be subdivided to determine where hydrologic information is necessary.

3-4. Impact Assessment

Analysis results are given by the impact of the flood damage reduction measures on flooding. These impacts are quantified through evaluation of with- and without-project comparisons of flooding represented by flooded area, depth, and frequency curves.

a. Project formulation. Evaluations are conducted to determine the performance characteristics of multiple sizes and configurations of measures. A range of flood events is analyzed for each measure. Each measure is analyzed to determine attributable flood damage reduction, costs, and benefits. The analysis will determine the project that maximizes the net benefits or NED plan. The project identified as the NED plan is that typically recommended for project design and construction. The risk of exceedance is also displayed for the selected design event to better illustrate the likelihood of design exceedance.

b. Design exceedance. The project design is exceeded when top of protection is exceeded or structural failure occurs. Every project can and eventually will be exceeded if it remains in place over a long period of time. When a flood overtops the protection, significant damage, possibly more than the damage prior to the project, could be experienced. Part of the analysis is to evaluate the consequences of design exceedance, and to design the project to minimize damage to both the project and the protected area.

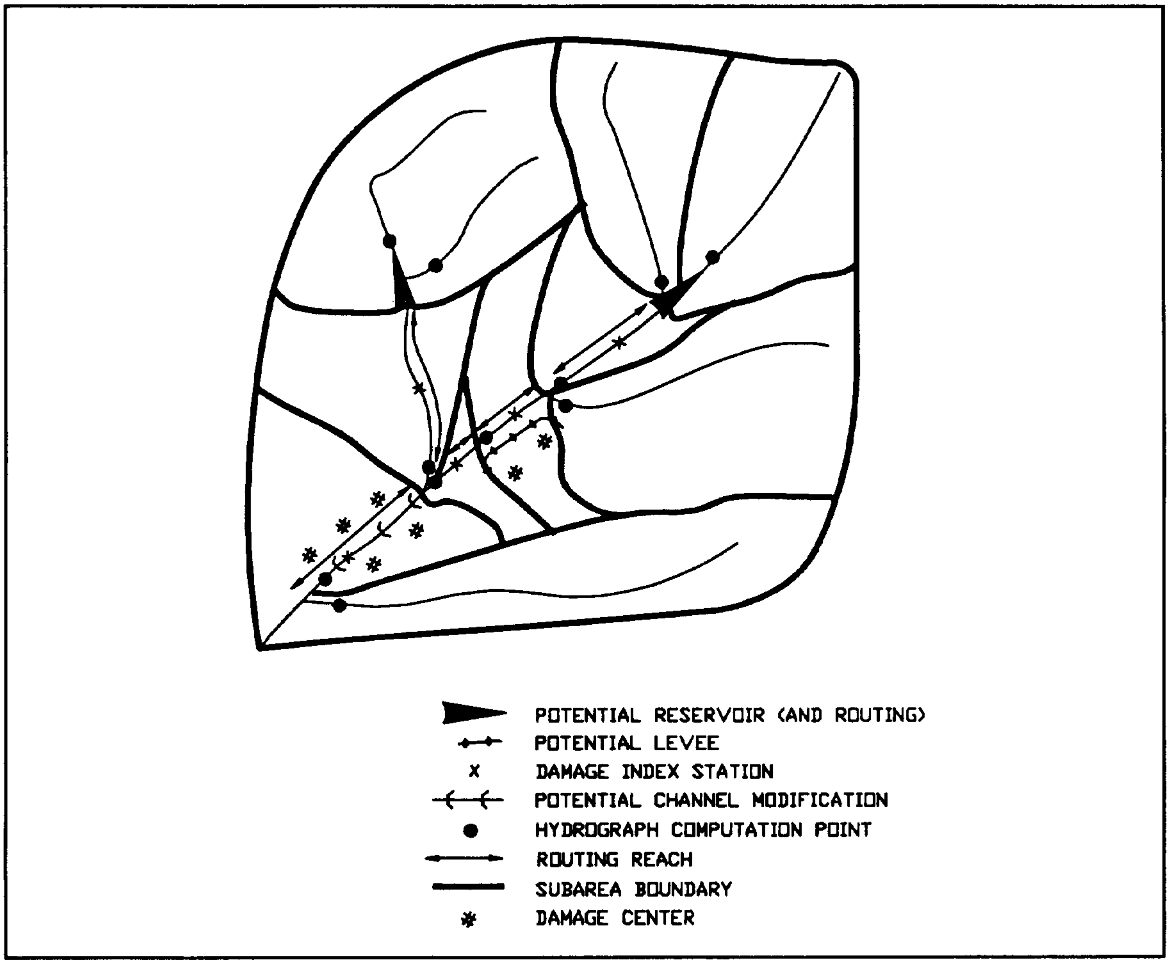


Figure 3-2. Subarea delineation

c. Positive/negative effects. Quantification of the positive effects of the project associated with its reduced flood damage is the basis for economic justification. However, the hydrologic analysis must address negative consequences as well. The consequences of the major types of flood damage reduction projects include but are not limited to the following:

(1) Reservoirs.

(a) Positive impacts.

- Flood flow and stage reduction at downstream locations.
- Downstream damage reduction.
- Source of water for multiple uses during low flow.
- Recreation usage.
- Hydropower generation.

(b) Possible negative impacts.

- Permanent inundation of reservoir lands.
- Eventual filling of reservoir with sediment.
- Changes to downstream sediment regime, usually erosion.
- Conveyance encroachment from lack of large floods.
- Changes in water quality.
- Increased losses due to evaporation.
- Elimination/reduction in fish spawning.

(2) Levees.

(a) Positive impacts.

- No flooding from exterior until design is exceeded.
- Protection to properties behind levee.

(b) Possible negative impacts.

- Induced flooding upstream and downstream of levee if extensive loss in valley storage.
- Potential for sudden large losses when levee design is exceeded.
- Interior flooding.
- Closures of openings may be required.

(3) Channelization.

(a) Positive impacts.

- Flood stage reductions even for events exceeding the design capacity.
- Local damage reduction through project reach.

(b) Possible negative impacts.

- Potential effect on fish spawning and wildlife habitat.
- Changed sediment transport characteristics.
- Increased channel maintenance requirements.
- Induced flooding downstream if extensive loss in valley storage.

(4) Diversions.

(a) Positive impacts.

- Flood stage reductions even for events exceeding the design capacity.
- Local damage reduction through project reach.

(b) Possible negative impacts.

- Changed sediment transport characteristics caused by uneven diversion of sediments.
- Induced flooding downstream of diversion reentry point.

(5) Nonstructural.

(a) Positive impacts.

- Individual structures protected.
- Essentially no change to environment.

(b) Possible negative impacts.

- High residual damage - infrastructure not protected.
- Emergency response required on event-by-event basis.

Chapter 4
Determining Flood Flows by Frequency Methods

4-1. Overview

a. Measures of flood severity. The majority of flood damage reduction studies require the evaluation of peak discharge, often used as the main measure of flood severity. Other variables, such as the total runoff volume, may also be critical for certain studies. Flood studies require frequency estimates in order to judge the performance of proposed flood damage reduction projects. The development of peak discharge-frequency relationships for a catchment is an important part of flood evaluation for Corps studies. This relationship is linked with elevation-discharge data and with elevation-damage data using risk-based analysis procedures to arrive at estimates of expected annual damage for with- and without-project conditions.

b. Discharge-frequency estimates. Some degree of uncertainty exists in all discharge-frequency estimates. This uncertainty results from insufficient information. The more data available, the better the estimate of discharge-frequency. In a typical flood damage reduction study, a certain amount of known (gaged) data will exist, but some of the study area may have no gaged data. Consequently, a combination of gaged and ungaged techniques are often used for the hydrologic analysis.

4-2. Analysis for Gaged Areas

The development of discharge-frequency relationships at gaged locations is a well-documented process involving statistical analysis of annual peak discharges. Figure 4-1 shows the results of a statistical analysis of recorded data. The analysis requires an adequate length of stream gaged record, with the data being both homogenous and of good quality. References (Water Resources Council 1982, EM 1110-2-1415) give the complete technical detail necessary for statistical analysis of stream-gaged records.

a. Record length.

(1) Although opinions vary as to a minimum record length, at least ten years of data is generally recommended. As one might suspect, ten years of data would seem a very limited amount to estimate, say, the 1-percent chance exceedance frequency peak discharge. The absence of significant peak discharges, such as during an extensive drought, or the occurrence of several floods during this short period would result in a poor estimate of the flood-frequency relationship. A "rule of thumb" suggests that the rarest flood that can be predicted with a reasonable level of confidence is about double the period of record. A 5-percent chance exceedance frequency (20-year) flood would be the largest for 10 years of data.

(2) Major changes in the estimates of return periods of rare floods are not unusual as one acquires more data. Obviously, the longer the period of gaged data, the more confidence one could have in the final result. Thirty or more years of data is generally desired for "good" statistical frequency estimates. Even if one has a lengthy record, comparison and modification of the relationship derived by statistical means is often made. This effort may involve regional studies considering nearby gages, and hypothetical floods developed with hydrologic models.

b. Record homogeneity/quality. As the record becomes lengthy, one becomes more concerned with changes in the catchment upstream of the gage, potentially resulting in a non-homogenous data record. Typical examples of non-homogenous records often cited are the urbanization of the land upstream of the gage, or the installation of a major reservoir. These man-induced changes result in different runoff volumes, hydrograph shapes, and peak discharges for similar storms. If significant changes occur during a period of recorded data, adjustments to or separation of the record is necessary. Quality of the data should also be considered, as stream gaging techniques can only estimate the total discharge during flood events. The USGS, the source for most gage data, evaluates the quality of its data at each of its gaged sites. A description of "Excellent" means that 95 percent of the daily discharges are within 5 percent of the true value, "Good"--within 10 percent, "Fair"--within 15 percent, and "Poor"--less than "Fair." Accuracy and confidence level are much lower for a statistical analysis of gaged data with a poor or fair rating than data with a good or excellent rating.

c. Need for ungaged techniques. When statistical analyses of gaged data are performed for a long-record station, the resulting estimate of discharge-frequency is considered the most accurate of any technique available. However, this relationship is only valid at the gage, and not at points significantly removed from the site. Thus, ungaged methods are almost always required along with statistical methods. Besides giving a precise estimate of discharge-frequency, gaged data allow one to compare the

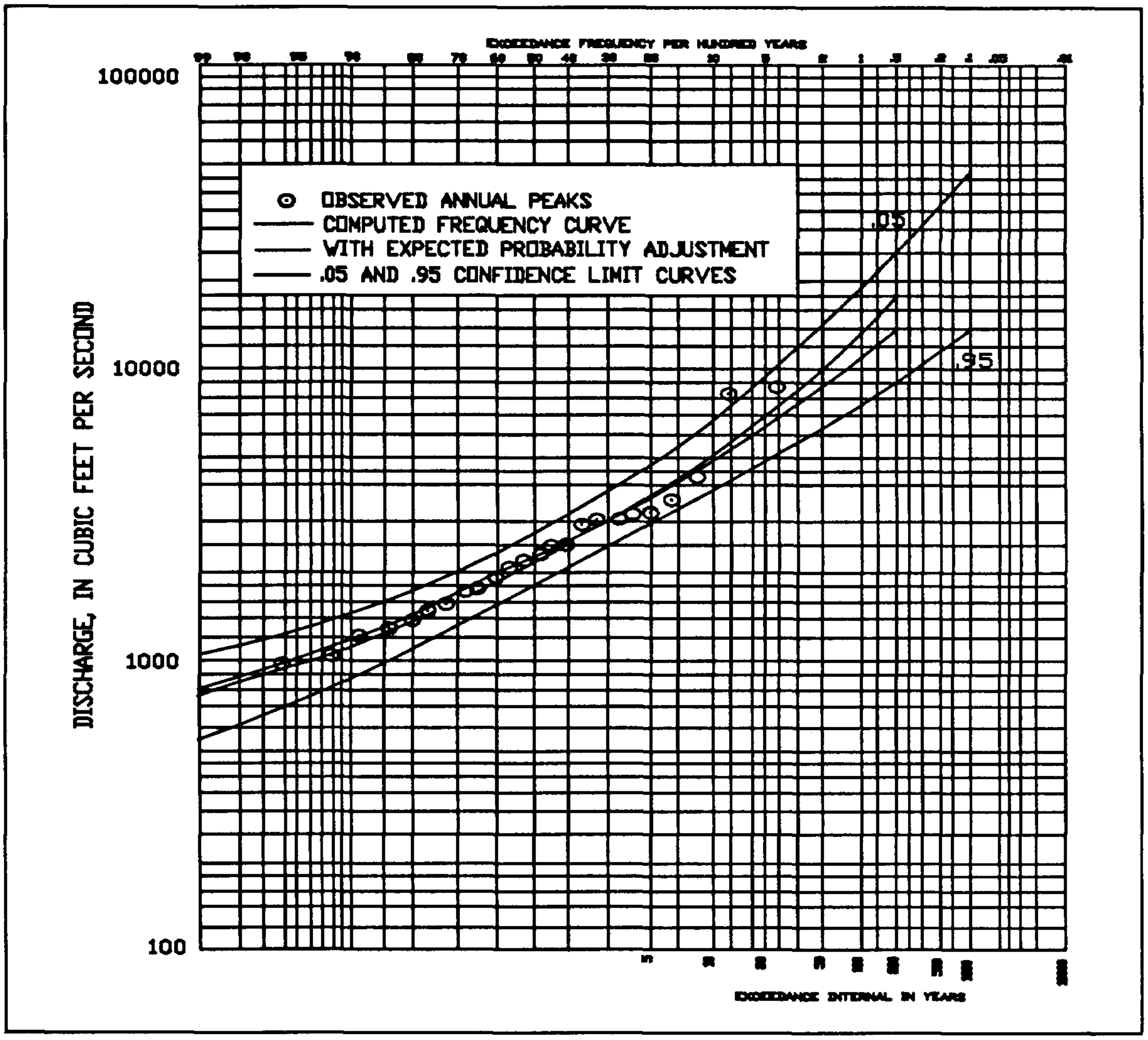


Figure 4-1. Flood frequency analysis by statistical methods

results of ungaged techniques and calibrate and/or verify the hydrologic methods used to estimate discharge-frequency relationship for ungaged areas.

4-3. Simplified Analysis for Ungaged Areas

Ungaged areas are those that have insufficient records to perform a statistical frequency analysis of peak discharge. This usually means no gages at all, but could also include sites that have only a few years of gaged data available. A wide variety of different techniques exist to determine discharge-frequency for ungaged areas. The following descriptions range from the simplest to the most complex.

a. Simplified equations. These methods involve the application of empirical relationships or simple envelope curves to estimate a peak discharge. They are usually applicable for only a certain size of catchment or for a specific type of discharge. Examples include the rational formula (Q = CIA, for very small areas) and the Myers Formula where discharge is function of area, giving the potential maximum possible discharge (McCuen 1989). These methods are easy to apply, but the results are of dubious quality. These techniques are applicable for certain preliminary level studies. Figure 4-2 illustrates the most widely used simplified equation: the rational

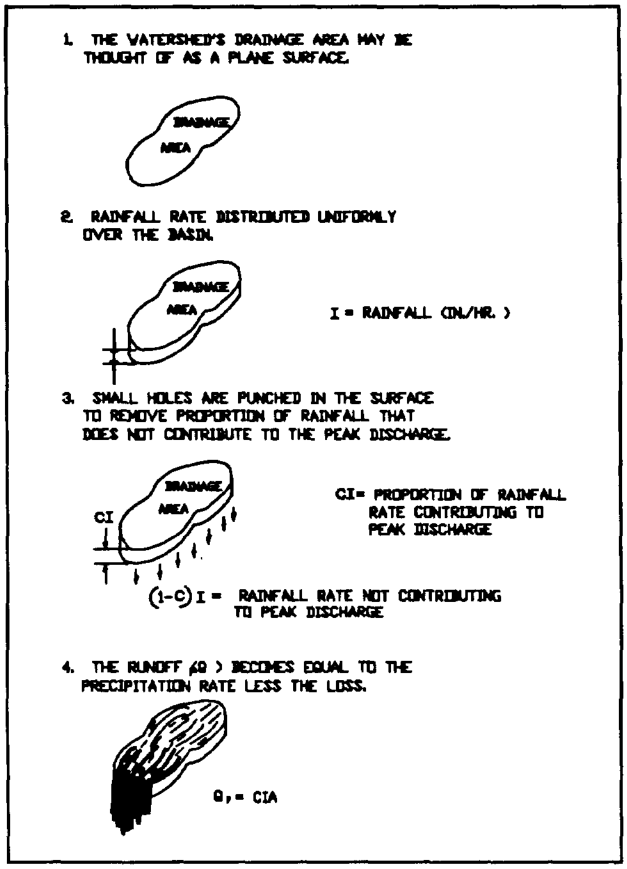


Figure 4-2. Example of simplified equations

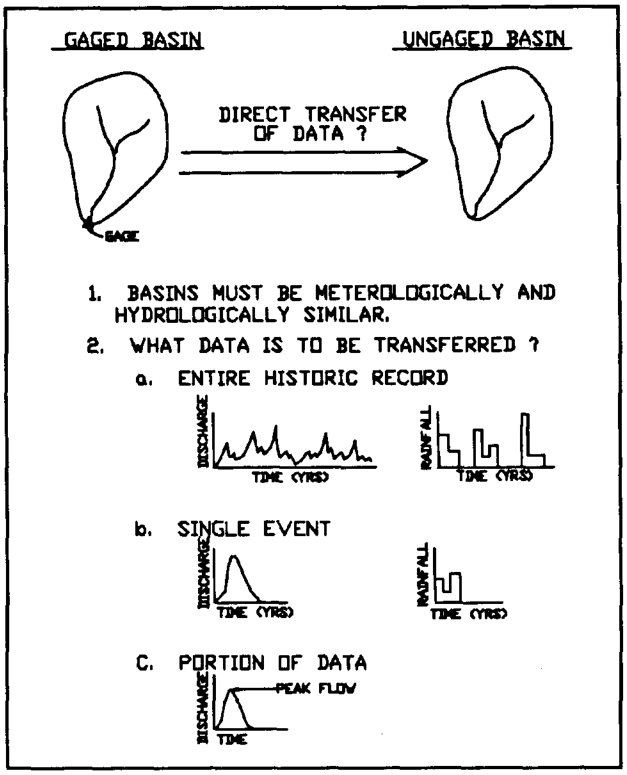


Figure 4-3. Example of transfer techniques

formula. It is still the main method used to determine design discharges for sizing storm sewers.

b. Transfer methods. This technique is also rather simple to apply, but the results are of appreciably better quality. It consists of a simple transfer of measured data from a gaged to an ungaged site, with the data being modified, as necessary, to reflect the conditions at the ungaged site. The modification could be a simple ratio of drainage area of the gaged versus the ungaged site, or be considerably more sophisticated. While discharge, sediment, and other gaged data are transferred to an ungaged site, precipitation data are most commonly transferred. Unless the region is mountainous, precipitation can be readily transferred a moderate distance without adjustment. The transferred data are assumed to be as likely to have occurred on the ungaged portion of the study watershed as on the gaged portion. Figure 4-3 illustrates the use of transfer techniques, which could be valid in any phase of the overall process.

c. Regression analysis.

(1) This method is a more detailed and sophisticated subset of transfer techniques and its development involves considerable work effort. Fortunately, regression analyses for peak discharges have been performed for most portions of the United States, usually by the USGS from gaged data (USGS 1983). Figure 4-4 illustrates the use of regression analysis. This technique develops the desired information (usually peak discharge for given frequencies) from a statistical analysis of long-term gaged records. A regression analysis is then performed linking the calculated peak discharge for each frequency to measurable parameters, like area, slope, stream length, etc. A prediction equation results which allows one to calculate a value for, say, the peak discharge knowing the drainage area and slope of the ungaged watershed. Differences between the discharge calculated with the regression equation and that found with a statistical analysis are called "residuals." These residuals may be mapped and used to adjust the discharge calculated for ungaged catchments. The regression analysis also allows one to estimate the accuracy of the prediction equation results.

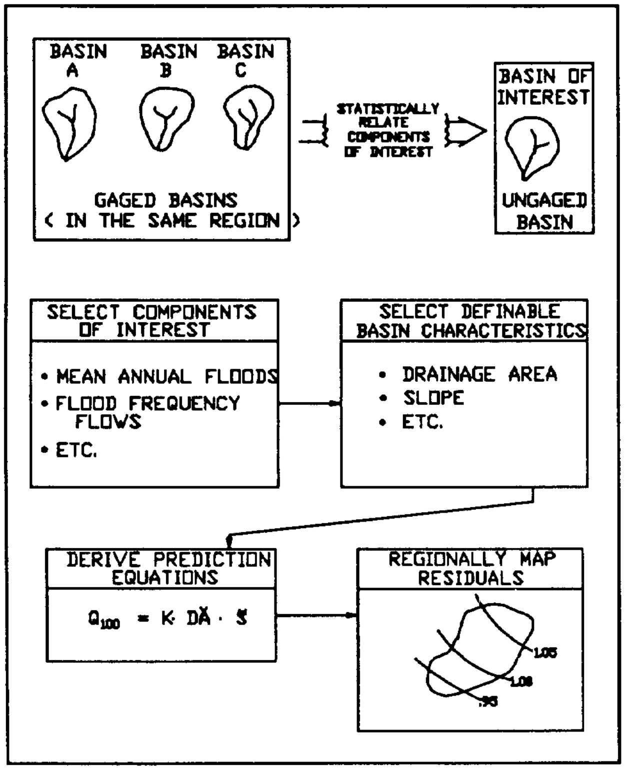


Figure 4-4. Example of regression analysis

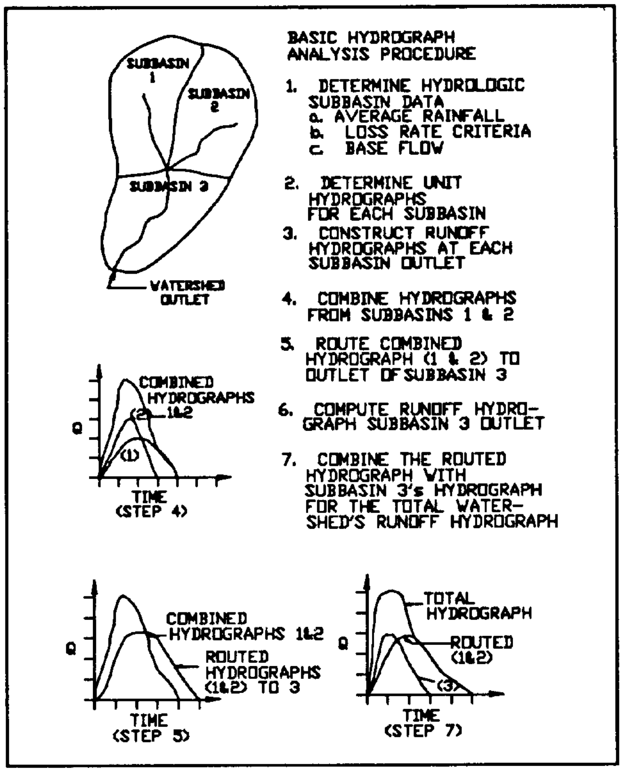


Figure 4-5. Example of hydrologic modeling

(2) Regression techniques are applicable in all phases of a hydrologic study and are valuable in evaluating the reasonableness of peak discharges determined with a hydrologic model. The main drawback to the technique is that only a peak discharge is available and there is no way to estimate how the peak discharge will change if a flood damage reduction structure is placed in the system. This technique is often used where only a peak discharge is needed to estimate flood severity, with flood insurance studies being a typical example. Regression analysis is considered by many to be less accurate in estimating a peak discharge than statistical analysis of gaged data at a site, but more accurate than hydrologic modeling.

4-4. Detailed Analysis for Ungaged Locations

a. The preceding simplified methods can be applied with minimal effort, but all have the same deficiency--how does the flood hydrograph change as it moves through the watershed system and how does the application of flood damage reduction measures affect the flood discharge? The only way in which these questions can be answered lies in detailed hydrologic modeling of the watershed. Figure 4-5 shows a schematic diagram of a typical hydrologic simulation using a model. A hydrologic model is a computer program that simulates the response of a hydrologic system based on meteorologic and physical watershed characteristics. The successful application of a hydrologic model in no easy task and requires knowledge and experience to prepare and operate the model and evaluate the validity of the results.

b. In addition, calibration of the model to some known data is important to gain confidence when applying the model to estimate unknown or rare events. Operation of the model for historical conditions (for calibration and/or verification), and for existing and future conditions (for establishing the severity of the flood problem and the effects of various flood reduction alternatives) is the basis for the overall flood reduction analysis.

c. There are many hydrologic models available to determine runoff hydrographs from a watershed. The procedures by which these models operate vary widely and not all models are applicable to a specific study area. The use of a single- event model versus a continuous simulation model (illustrated in Figure 4-6), actual versus hypothetical (frequency) rainfall, various loss rate functions, modeling of subsurface flow and losses, unit hydrograph versus kenematic wave methods, hydraulic versus

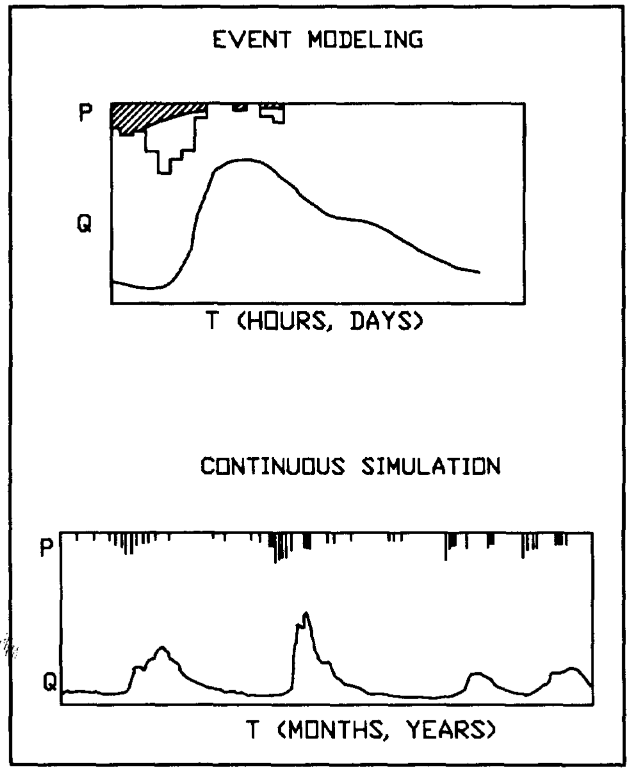


Figure 4-6. Slope event versus a continuous simulation model

kinematic wave methods, hydraulic versus hydrologic routing, etc. are features of the various models. Some models are considerably more detailed and sophisticated than others, requiring a higher level of expertise. The rainfall-runoff process, which these programs model, is presented in Chapter 5.

Chapter 5
Determining Flood Flows by Precipitation-Runoff Analysis Methods

5-1. Introduction

Detailed hydrologic modeling is usually required for flood damage reduction studies. This area of hydrologic engineering, along with river hydraulics, normally takes the bulk of time and money in a study. This effort requires determination of how to subdivide the watershed to give required hydrologic information at points of interest, to develop the precipitation, loss, runoff, discharge, and routing information, and to calibrate and verify the model. Detailed modeling usually takes place during the feasibility phase. This chapter describes the various components of the hydrologic modeling performed.

5-2. Watershed/Subbasin Delineation

Delineation of the watershed into subareas to determine discharge information was discussed in paragraph 3-3. The study team must also participate by defining their information needs during this process. Location of damage reaches, potential flood damage reduction measures, political boundaries, and other items may cause further modification subareas to provide the necessary hydrologic data.

5-3. Analysis Approaches

a. General. The two main methods for determining flood runoff can be described as single-event analysis or continuous simulation, as illustrated in Figure 4-6. The former refers to determining the runoff from a single storm-flood event (the flood of 1986 or the 2-percent chance hypothetical flood). The main problem with this technique is a lack of knowledge of the antecedent soil moisture, especially for hypothetical floods. Assumptions as to wet or dry soil conditions may have a significant effect on the corresponding runoff.

b. Continuous simulation. The continuous simulation technique overcomes this problem as all periods of streamflow (droughts, floods, and all events in between) are simulated. This process is much more satisfactory in that more of the streamflow process is analyzed, but continuous simulation computer models are generally more data intensive and time-consuming to operate than event models. A lack of knowledge of other hydrologic variables needed for continuous models (evaporation, interception, subsurface and groundwater flow, etc.) may cause the results to be no more and perhaps less accurate than those of the single-event model. Continuous simulation models are often used where agricultural flood damage is extensive, because the time of year in which the flood occurs is important for damage calculations. Also, agricultural flood damage analysis may be required for relatively frequent events, such as the once- or twice-per-year flood. A flood this frequent is not usually suitable for event modeling.

c. Single-event analysis. Single-event models are typically used in urban flood damage analyses, since time of year is generally not important and the project design is for a rarer frequency, like the 1-percent chance flood. This publication will address only the hydrologic analysis related to a single- event model.

5-4. Precipitation/Runoff

Each subarea contributes a discharge hydrograph to the water moving throughout the overall watershed. Runoff from the several subareas is combined to yield the total discharge hydrograph at the outlet. Subbasin characteristics used to compute runoff include: rainfall, losses, transforms, and base flow.

a. Precipitation. Precipitation is atmospheric water in all its many forms. Flood reduction studies are primarily concerned with rainfall, with snowfall/snowmelt also of concern in certain regions of the United States. Rainfall is also further defined as being historical (recorded) or hypothetical.

(1) Historical rainfall. The engineer requires historical or actual rainfall for one or more storm events that produced flooding in the study watershed. The purpose of this historical rainfall is to calibrate the overall hydrologic model, ensuring that the model's output is representative of the basin. The actual rainfall that occurred over the study watershed produced a flood that was measured at one or more gages, or that reached heights that were remembered by local residents and then surveyed to determine high-water mark elevations. Rainfall input is used by the hydrologic model to produce flood hydrograph output at a gage site or a water surface elevation at a point of a known high-water mark. If the model's output is reasonably close to known discharges or water surface elevations, the model is considered to be calibrated and ready for use in developing discharge-frequency relationships. Historical rainfall for several actual storm-flood events would be desired, with the rainfall time sequence also being necessary. Depending on the size of the

watershed, incremental rainfall values ranging from 5-minute intervals to 24-hr increments would be necessary. Figure 5-1 shows an example of historic rainfall for application to a hydrologic model.

(2) Frequency rainfall.

(a) Hypothetical rainfall is required to determine discharge hydrographs for specific flood frequencies. Hypothetical rainfall is taken from past studies of the NWS, with Technical Publication (TP) 40 (NWS 1961), TP 49 (NWS 1964), and National Oceanic and Atmospheric Administration (NOAA) HYDRO-35 (NWS 1977) being the sources of these data for the 35 states east of the Rocky Mountains. The other 13 states in the continental United States have individual state atlases (NOAA 1973) to give the detailed information required in mountainous terrain. Alaska (NWS 1963, 1965a) and Hawaii (NWS 1962, 1965b) also have guidance specific to those states. Figure 5-2 gives an example of the type of information in NWS TP 40.

(b) Rainfall information is extracted at the location of the study watershed for each duration for a given frequency. The rainfall is incremented to determine depth in each time period, adjusted to reflect storm occurrence over an area rather than a point, and arranged in an appropriate pattern. An example of the adopted storm pattern for a given frequency and watershed is shown in Figure 5-3. Each frequency desired, from 50- through 0.2-percent chance exceedance storms, is developed in a similar fashion. Six or seven separate frequency storms are often required to give sufficient points to determine the resulting discharge-frequency curve with hydrologic modeling.

(3) Standard project storm.

(a) The hypothetical Standard Project Storm (SPS) is generated using a standard procedure (USACE 1965) for areas east of 105 deg longitude. For western areas, SPS's are normally generated by adjusting and transposing rare observed events to the study area from hydrologically and meteorologically similar areas. An example of an SPS, arranged for appreciation, is shown in Figure 5-4.

(b) The SPS is used to develop the Standard Project Flood (SPF). The SPF is the flood that can be expected from the most severe combination of meteorologic and hydrologic conditions that are considered reasonably characteristic of the region. The primary application of the

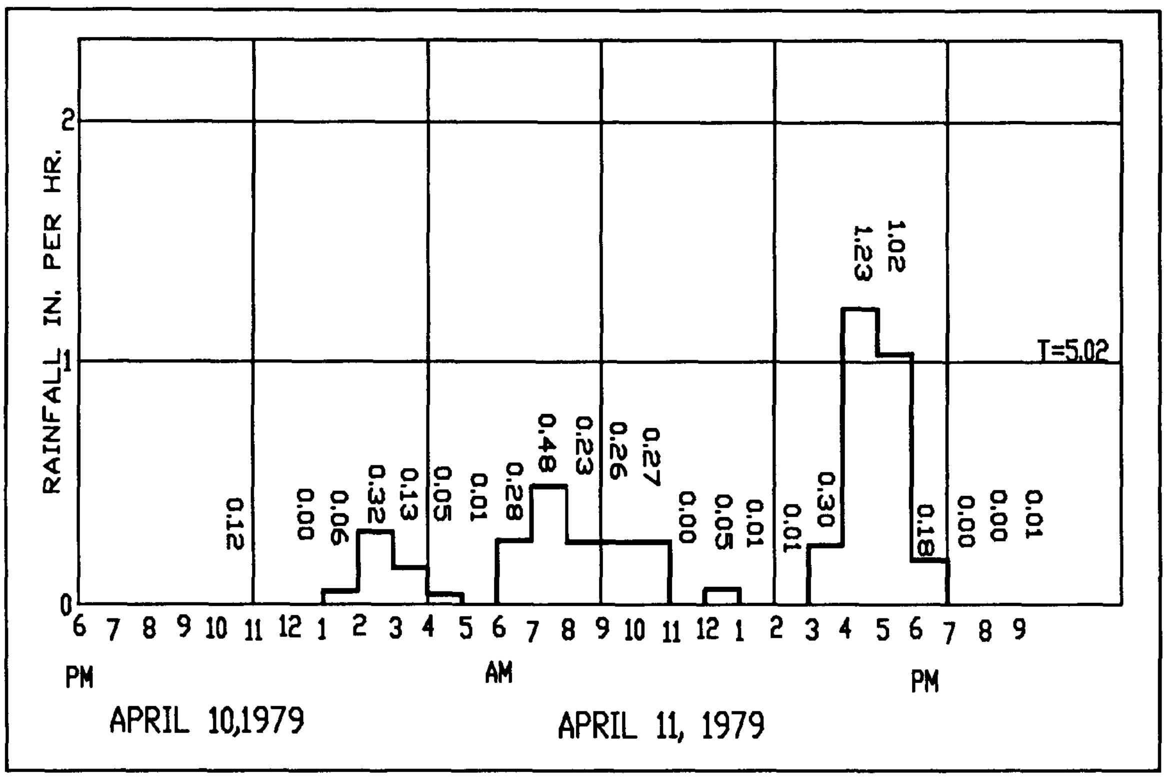


Figure 5-1. Example of historic rainfall

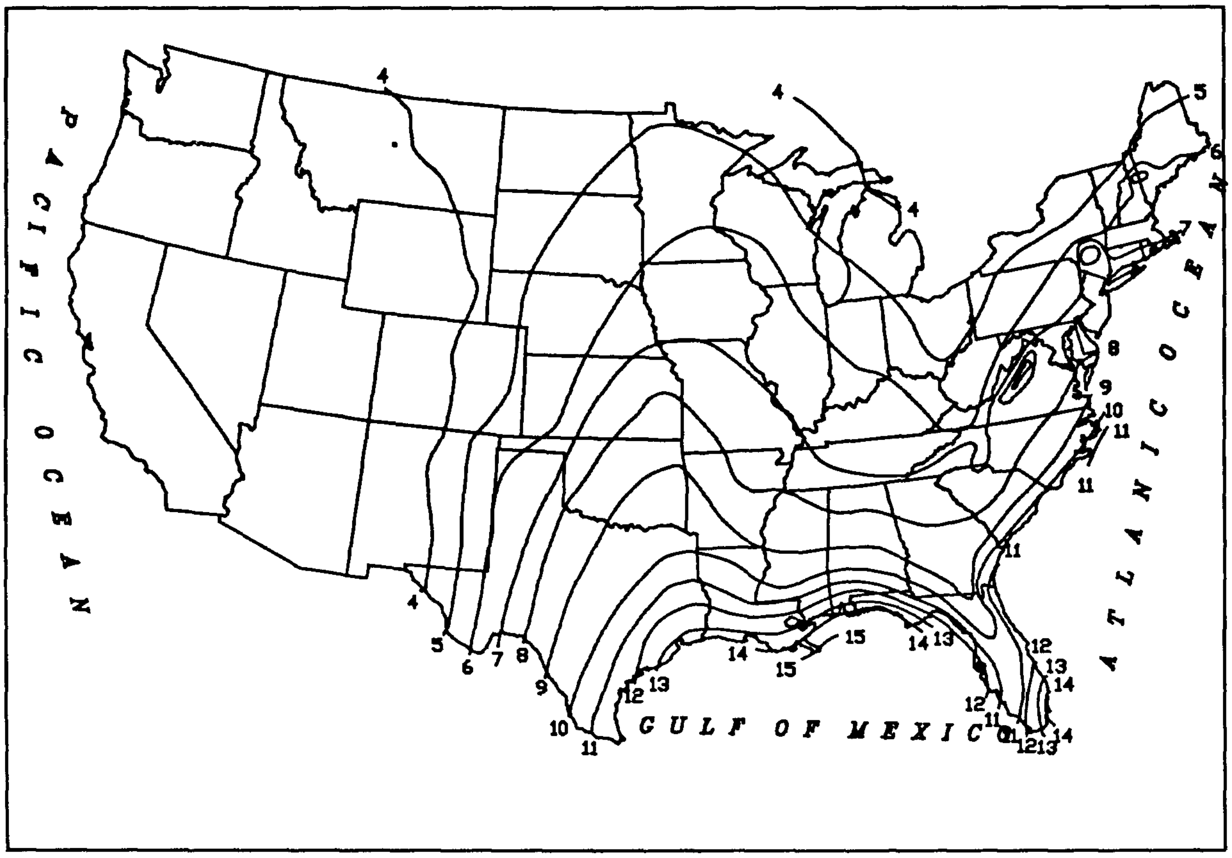


Figure 5-2. 100-year, 24-hr-duration rainfall map

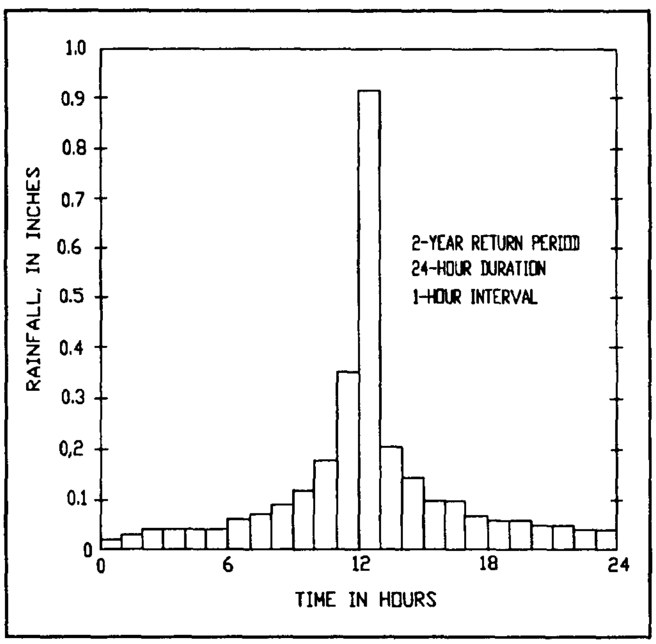


Figure 5-3. Typical time distribution for a hypothetical storm

SPF is to evaluate the performance of projects for an extreme event. Although a specific frequency cannot be assigned to the SPF, a return period of a few hundred to a few thousand years is commonly associated with the event.

(4) Probable maximum storm. This hypothetical event is normally required when dams and reservoirs are under consideration. Failure of a dam by overtopping could be a catastrophe for which no risk of failure would be allowed. Consequently, the Probable Maximum Storm, or PMS, (NWS 1982) is used for dam and spillway design to ensure that there is essentially no risk of design exceedance. Figure 5-5 shows a PMS arranged for use in a hydrologic model. The PMS is based on meteorologic studies of potential water in the atmosphere under the most extreme conditions.

(5) Snowfall/snowmelt.

(a) Snowfall is important in mountainous regions and in the northern portions of the United States. Unlike

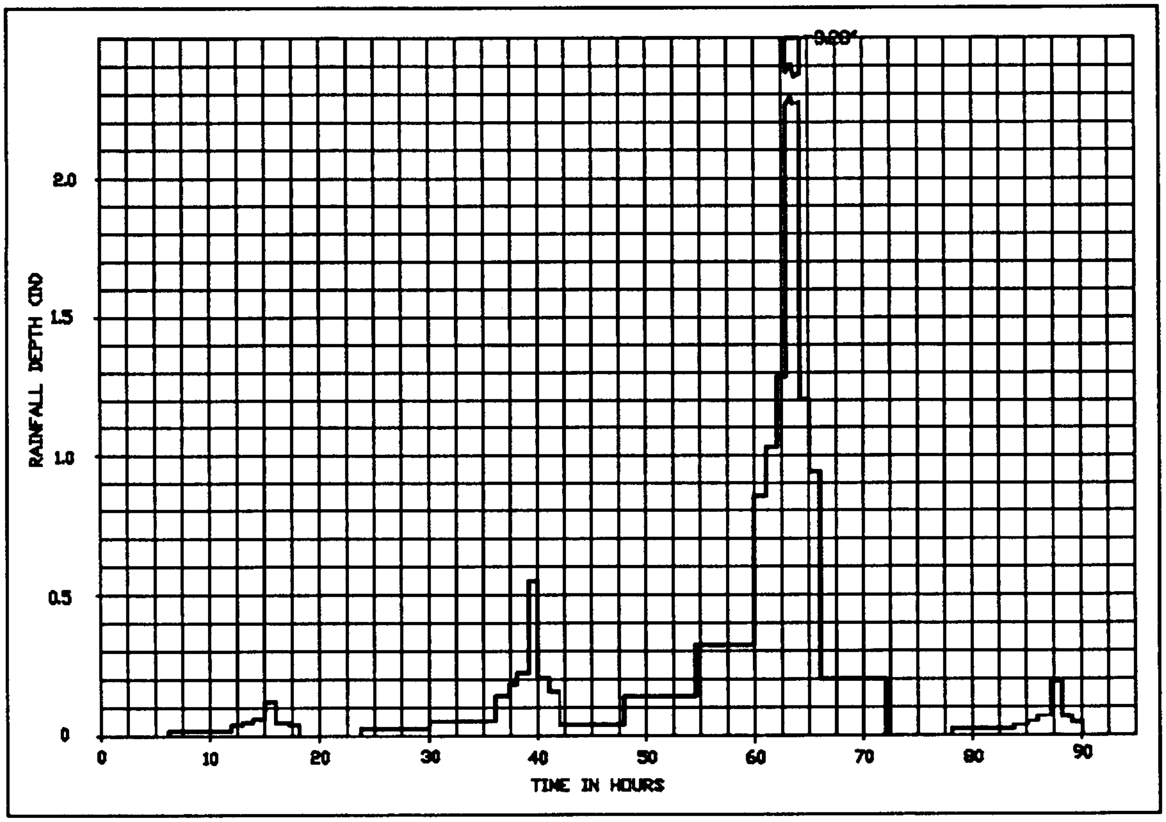


Figure 5-4. SPS arrangement

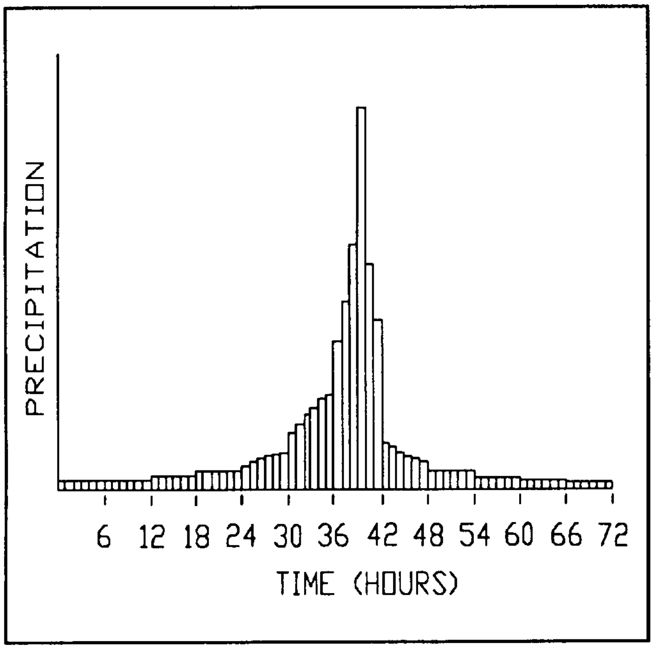


Figure 5-5. Probable maximum storm arrangement

rainfall events, snowfall can accumulate throughout the winter as a snowpack, which melts when warmer weather occurs. Therefore, the important variables for snow are: depth of snowpack and corresponding water content, air temperature, and topographic elevation. The last variable is important because the air temperature decreases with increasing elevation, and most air temperature monitoring gages are located in lower elevations. Depth of snowpack is monitored by physical measurements or by remote telemetry, with the corresponding water content determined.

(b) Snowpack information is critical for reservoir operation or structures receiving meltwater runoff, which includes most of the reservoirs in the western United States. Flood studies involving snowmelt are based on recorded data when available. When snow data are not available, it may be estimated by knowing rainfall and air temperatures, and converting to an estimated snowfall. No hypothetical basis is available for determining a synthetic snowmelt event.

b. Losses.

(1) General. Many methods are available for determining losses during a rainfall-runoff event, ranging from quite simple to very complex. For an event-type analysis, loss rates have been estimated using the uniform and initial method, the U.S. Soil Conservation Service (SCS) Curve Number method, the Horton technique, the Green-Ampt procedure, the exponential method, etc. (USACE 1990a). For a continuous simulation analysis, loss rate estimates could range from a simple runoff coefficient to a complete soil moisture accounting system.

(2) Adjustment of loss rates. The appropriate method is largely up to the judgement of the hydrologic engineer. Since the loss rates during a runoff event are not known, loss rates may be adjusted during the calibration analysis to allow a better reproduction of the known hydrograph or high-water marks by the model. Loss rates may also be adjusted depending on the storm severity, since the same loss rate would not be expected for a 50-percent chance (2-year) storm as for a 1-percent chance (100-year) storm. A rare storm is typically one in a series of events, which tend to increase the soil's antecedent moisture level and the corresponding runoff. Consequently, loss rates during a rare event would be expected to be less than a more common storm event. Loss rate adjustment is one way in which the argument in favor of continuous simulation models may be partially addressed. Figure 5-6 gives examples of simple loss rate accounting procedures.

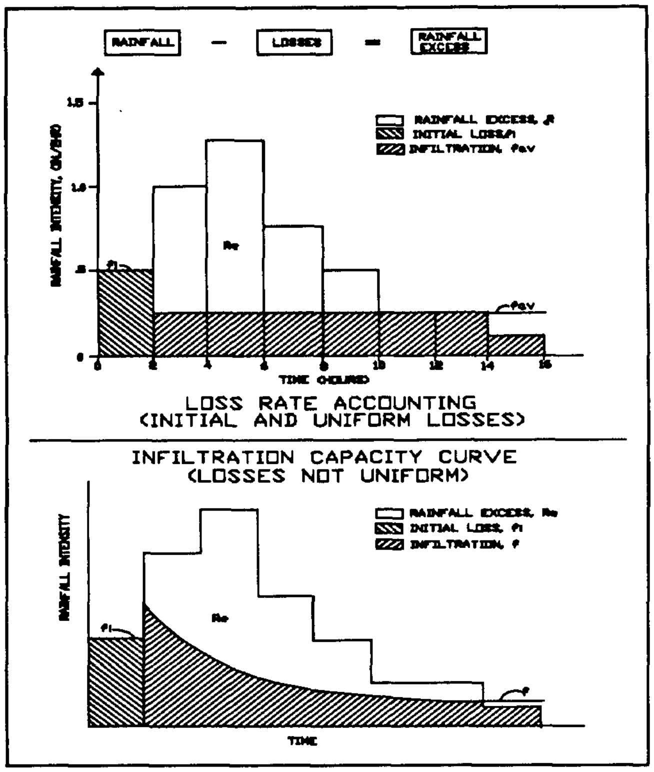


Figure 5-6. Examples of simple loss rate accounting

c. Runoff transformations. After precipitation and loss rate analyses are complete, the engineer is left with an estimated runoff from the watershed expressed in inches per time period for the storm. Runoff in cubic feet per second, rather than (for instance) inches per hour, is needed for hydrograph analysis. Consequently, a transformation is required to obtain runoff quantities in the desired format. Most hydrologic modeling makes this transform using the unit hydrograph technique. Occasionally in highly urbanized catchments, the kinematic wave technique is used. The selection of which technique to use is normally up to the hydrologic engineer.

(1) Unit hydrograph method.

(a) This technique was first developed in the 1930's and is still the predominate technique used in the Corps for a runoff transformation. Many unit hydrograph (UHG) methods are available, with the main ones being the Snyder, Clark and SCS techniques (USACE 1990a). The unit hydrograph technique involves the development of a "pattern" hydrograph, representing the runoff of 1 in. (or unit) of rainfall excess, occurring uniformly during a specified duration (1 hour, 1 day, etc.) over a specified watershed. The assumption is that any other rainfall excess (more or less than 1 in.) during the same duration produces a similar hydrograph with the discharge ordinates proportionally higher or lower than those of the unit hydrograph. Figure 5-7 illustrates this concept.

(b) Preferably, the UHG is derived from recorded rainfall-runoff events recorded at stream gages. These "known" unit hydrographs may be related to measurable basin parameters through regression analyses to determine unit hydrograph parameters at ungaged sites throughout the watershed. This procedure is the same as described in paragraph 4-3. Where no gage data are available, generalized techniques, such as the SCS methods, are appropriate.

(c) The advantages of the unit hydrograph method include: extensive experience with usage, well-documented theory, and applicability to the development and use of regional parameters. The disadvantage is that rainfall excess over the basin is transformed to a discharge hydrograph at the mouth, without specific

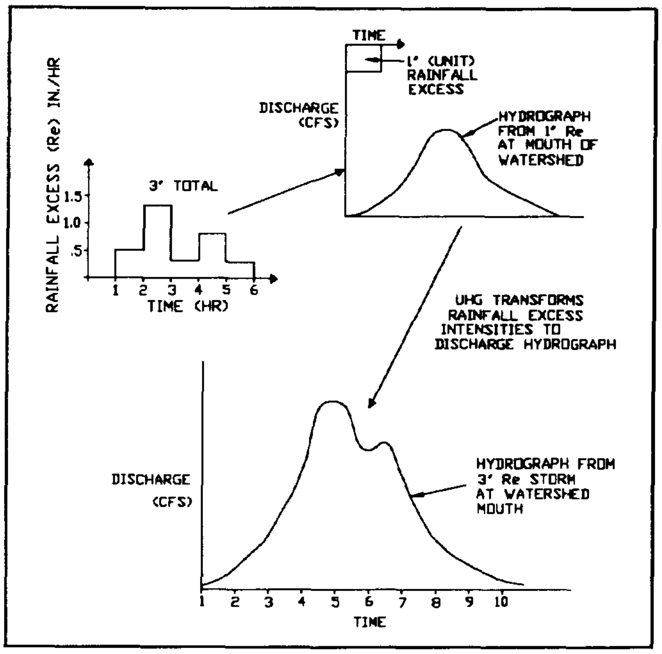


Figure 5-7. Unit hydrograph concept

regional parameters. The disadvantage is that rainfall excess over the basin is transformed to a discharge hydrograph at the mouth, without specific accounting for the movement of runoff over land surfaces. Unit hydrographs may differ somewhat as storm intensities increase; therefore, using the same unit hydrograph for a 2-in. storm and for a 10-in. storm is generally not advisable.

(2) Kinematic wave method.

(a) This technique was developed in the 1950's and attempts to trace the movement of runoff through the watershed to the basin outlet. The main assumption of this technique is that water moves "kinematically," or at the slope of the land surface or channel bottom. This movement is modeled by use of "typical" lengths and slopes for overland flow, collector channels, and the tributary or main channel. Friction values must also be assigned to each element. Figures 5-8 and 5-9 show conceptually the watershed modeling and individual elements used in applying the kinematic wave procedure, respectively.

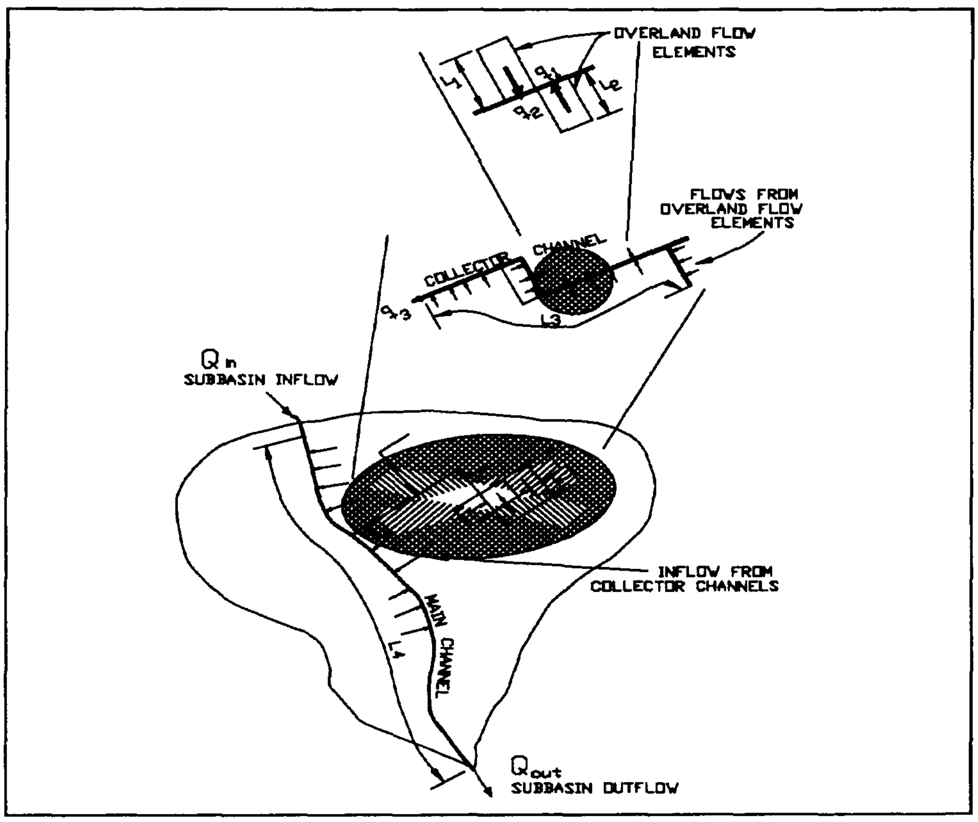


Figure 5-8. Watershed modeling using the kinematic wave method

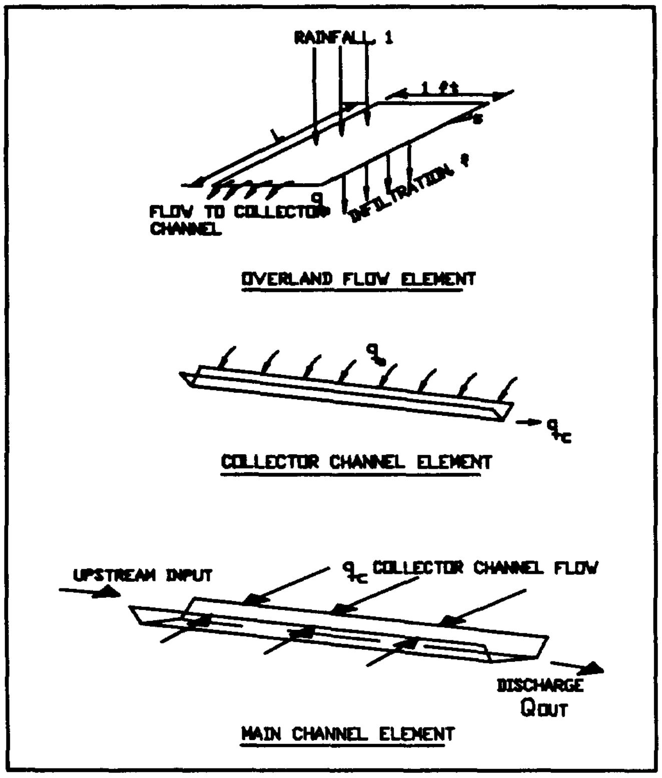


Figure 5-9. Elements used in kinematic wave calculations

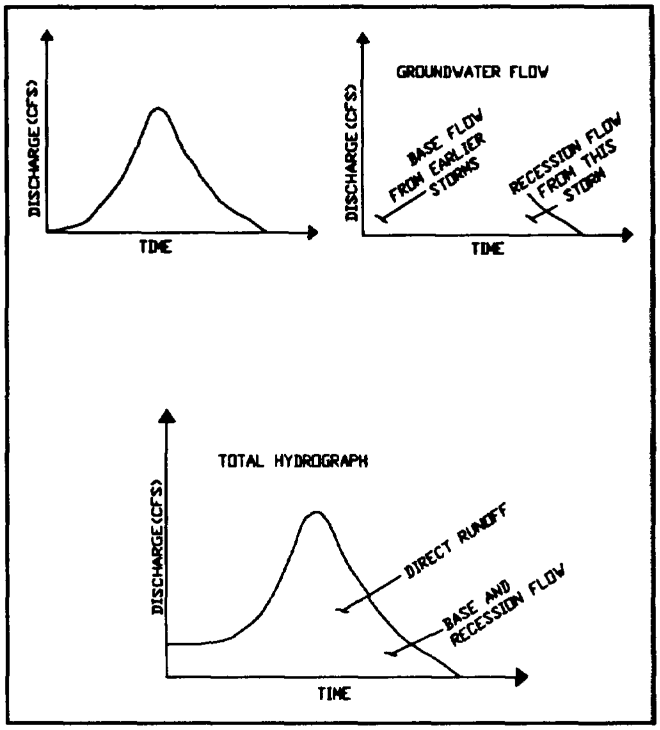


Figure 5-10. Base/recession flow hydrograph components

(b) Application of this procedure requires considerable judgment in selection of appropriate variables for each flow strip and to evaluate the discharge hydrograph output for reasonableness. The advantage of this technique is that it is more physically based and conceptually complete in terms of the physics of runoff. The main disadvantages are difficulty in determining average strip lengths, slopes, and roughnesses, and reduced applicability for low-slope land surfaces and channels.

d. Base flow and recession flow. The preceding discussion focused on rainfall excess and the resulting direct runoff. The resulting discharge hydrograph does not include the streamflow that would have occurred without any rainfall excess, or the water that enters the stream from groundwater flow well after direct runoff has ended. The former inflow is called base flow and the latter is termed recession flow. Figure 5-10 illustrates base and recession flow segments of the total discharge hydrograph. Base and recession flow are relatively small portions of the runoff hydrograph for small watersheds that are sometimes ignored, especially for small urban catchments. These parameters become important as the basin area increases and certainly cannot be ignored for large watersheds.

5-5. Routing Concepts

After the foregoing analysis is complete, a discharge hydrograph has been computed at the outlet of a subarea. This hydrograph moves downstream, combines with otherhydrographs, and moves through the channel and floodplain towards the mouth of the main river. Means of accounting for hydrograph movement is by routing. Routing is simply a method of translating the hydrograph in time and accounting for the hydrograph's change in shape as it moves through the stream system. Hydrologic routing accounts for changes in the time distribution of volume and employs a relatively straightforward computation procedure. Figure 2-12 illustrates the basic concept of hydrologic routing. Hydraulic routing, or unsteady flow computation, is much more difficult to apply and can include the effects of pressure and momentum changes. The application of hydraulic routing requires an engineer with special experience and is further addressed in Chapter 6.

a. Hydrologic routing computations.

(1) Routing techniques. Many techniques are available for hydrologic routing, ranging from simple graphical methods to more complex techniques. These methods include: lag-average, Tatum, Muskingum, Muskingum-Cunge, modified Puls routing and others (USACE 1990a). All methods attempt to account for translation time through the reach and for reach storage. The selection of an appropriate routing procedure depends on the judgment of the engineer, the availability of information to determine routing parameters, and the type of flood damage reduction project under investigation.

(2) Reservoir and Puls routing. The most conceptually complete methods are reservoir (flat pool) and Puls routing. The procedures for both are similar and directly account for the storage available in the routing reach. Figure 5-11 shows the results of a typical reservoir and channel operation. Figure 4-5 shows the routing operation as part of the overall modeling process.

(3) Routing example. Possibly the easiest way to visualize a routing operation is with a reservoir example. A dam constricts the outflow to whatever opening is designed through the dam structure (conduit and spillway). Consequently the inflow hydrograph is largely stored behind the dam and released at a lower rate through the outlet, over a much longer time period. The storage benind the dam and the characteristics of the outlet structure must be known to determine the outflow hydrograph from the dam. A hydrologic analysis of the latter two features will result in a storage versus outflow relationship. This relationship plus the inflow hydrograph can be used to route the inflow hydrograph through storage, determi..ing the outflow hydrograph and the maximum pool stage. This operation is important to determine the adequacy of the spillway discharge capacity and to ensure that the dam is higher than the design pool elevation.

(4) Routing reaches. The subdivision of a total watershed into subareas determines the routing reaches

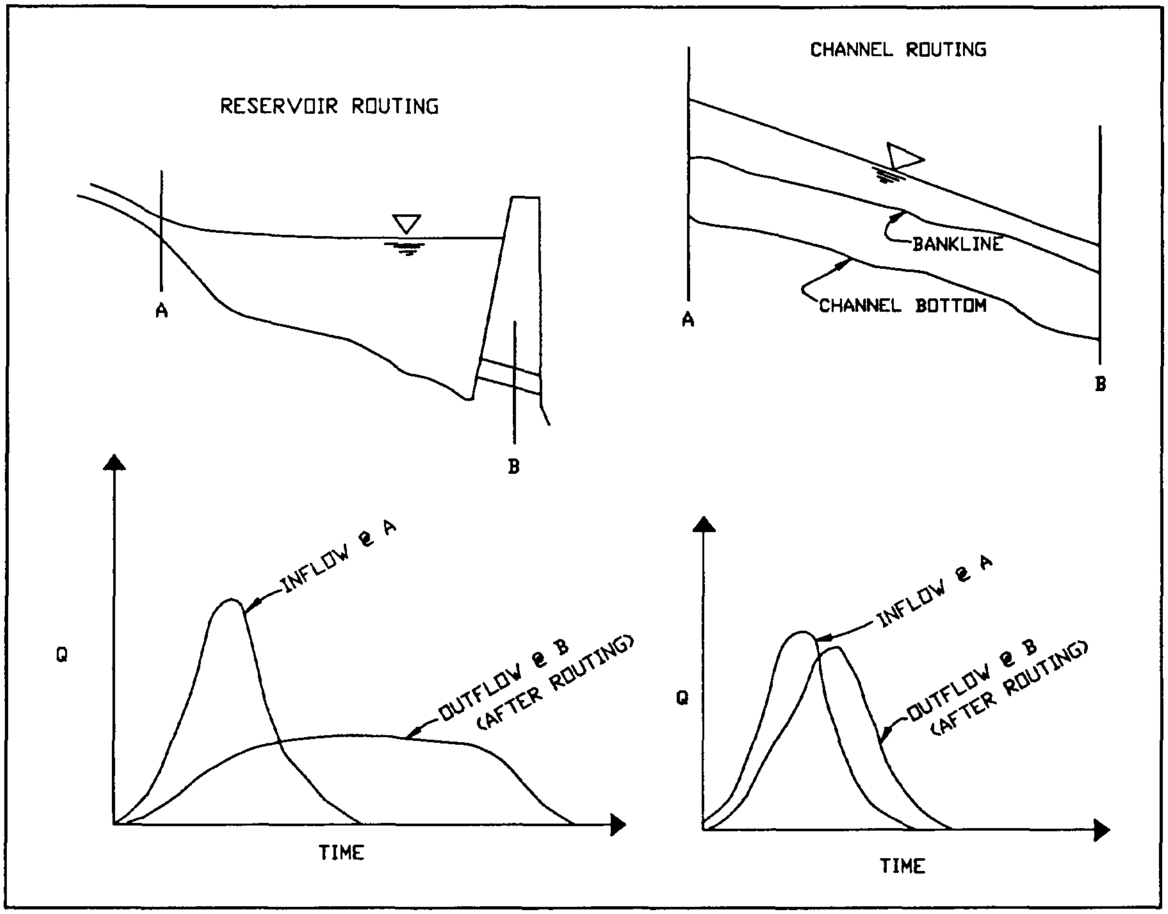


Figure 5-11. Examples of reservoir and channel routing

required. Travel times and storage within these reaches are determined so that routing operations may be carried out and the total hydrograph may be translated downstream. Figure 5-11 shows that reservoir routing greatly affects both timing and shape of the outflow (routed) hydrograph, while channel routing mainly affects the timing of the outflow (routed) hydrograph.

(5) Flood reduction components. Routing studies are important to evaluate the effects of flood reduction components throughout the watershed. Reservoir routings are carried well downstream to evaluate reduced flooding attributable to the structure. Local protection projects (levees and channel modifications) may affect nearby areas adversely by removing or reducing storage available. The magnitude of these changes can only be addressed by routing studies with and without the flood reduction component.

5-6. Calibration of the Model

a. General. All of the foregoing components are incorporated into the overall hydrologic model to simulate discharge hydrographs and determine discharge-frequency relationships throughout the watershed. However, prior to developing this information, the model must be operated for storm-flood events having known input and output to ensure that the model is reproducing actual floods. This process is called "calibration" and is a key part of the total hydrologic modeling process.

b. Calibration process. Historic rainfall from one or more storms is used as input to the total model, which consists of a number of subareas and routing reaches. The model determines losses and rainfall excess, transforms excess to discharge hydrographs, and routes and combines the hydrographs through the watershed. Calculated hydrographs are compared with recorded hydrographs at gage locations in the watershed. When the model reasonably reproduces known hydrographs at the gages, the model is considered to be calculated. If the reproduction of an actual event is poor, one could consider adjusting loss rates, runoff transform coefficients, routing coefficients, etc. (within reasonable limits) to obtain an improved simulation.

c. With calibration, the modeler can have increased confidence that the application of hypothetical (frequency) rainfalls to the model should result in representative runoff hydrographs of that frequency event. Calibration is completed when discharge hydrographs, measured versus calculated, may be compared. Figure 5-12 shows a successful calibration of model output compared to recorded discharge information at a stream gage. In the absence of extensive gaged data, comparison of a calculated peak discharge against that calculated by the regression analyses of paragraph 4-3*c*, or against high-water marks (after calculating a water surface profile with the hydrologic model's output for peak discharge) may be used to calibrate the model.

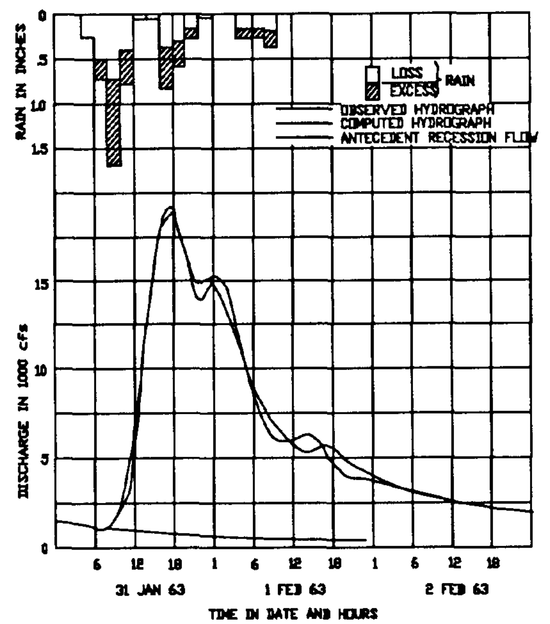


Figure 5-12. Example of a successful calibration

5-7. Verification of the Model

Verification is the final process in hydrologic modeling, after satisfactory calibration has been achieved. Model verification is the process of utilizing additional known data not used in the calibration process to verify that the calibrated model will give good results for unknown storm-flood events. The calibrated model is used with additional historic rainfall to give discharge hydrographs for comparison with gage data. No adjustments of the calibrated model are made in the verification process. The highest level of confidence in model output is achieved when the calibrated model successfully reproduces the known hydrographs with this additional historic data. However, verification is not always possible, as sufficient known storm-flood events may not be available for both calibration and verification.

Chapter 6
Determination of Flood Elevations

6-1. River Hydraulics

Chapter 5 presented methods for determining a peak discharge or volume of runoff from a flood event. However, much of flood analysis and design requires the severity of a flood to be measured in terms of a depth, water surface elevation, or area flooded, rather than peak discharge. This chapter describes the general methods used to determine water surface elevation, given flow.

a. Simple versus complex. Many methods exist for making the conversion from peak discharge to flood elevation, ranging from a simple rating curve to multidimensional analysis. Each requires increased increments of time, money, and engineering experience to be successfully applied. Paragraphs 6-2 through 6-5 describe the most common methods and give a basis for proper method selection.

b. Steady versus unsteady analysis. Flood elevation analyses may be subdivided into those based on steady flow (discharge is constant with time) and those based on unsteady flow (discharge varies with time). The latter is closer to the real-world situation; however, the great majority of analyses of river hydraulics can be made assuming steady flow. Unsteady flow evaluations are considerably more complex. Paragraphs 6-3 and 6-4 describe these two types of analyses.

c. Rigid versus mobile boundary. Alluvial streams experience modifications to their geometry with time, due to sediment transport. Erosion and deposition cause increases or decreases in a stream's flow capacity, which can be reflected by changed flood elevations. However, most flood elevation determinations may be satisfactorily made by assuming that the stream boundary is rigid, greatly simplifying the river hydraulics analysis. Paragraph 6-6 discusses mobile boundary hydraulics and its application.

6-2. Development and Use of Rating Relationships

a. Gage sites. The conversion of discharge to river stage, or water surface elevation, is most accurate (and easiest) when performed at a gage. Continuous measurements of stage, along with periodic measurements of flow, serve to give a direct relationship for discharge, when the stage is known. Figure 6-1 gives an example of a stage-to-discharge relationship at a river gage. This relationship is developed by many years of data accumulation at the gage site. As seen, many points are available for discharges within banks or that slightly exceed bank-full stages. Higher discharges occur infrequently, only during floods, and only a few points in this portion of the rating curve may be available. The fewer actual data points, the more uncertain the relationship.

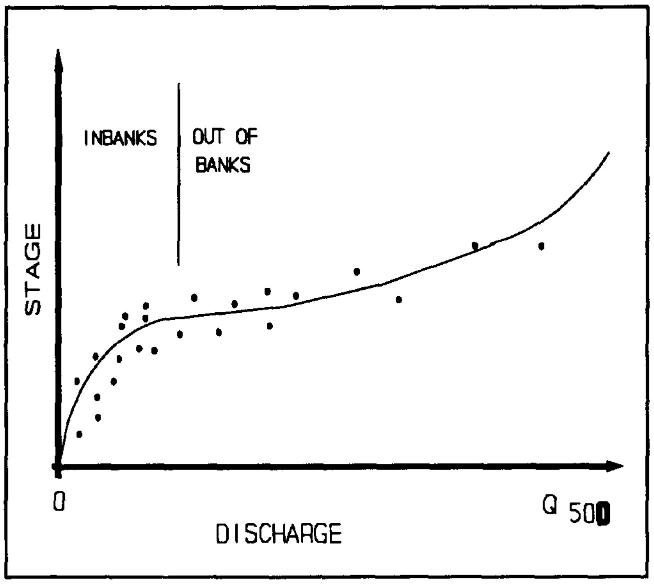


Figure 6-1. Rating curve developed from gaged data

b. Rating curves. Changes in land use, channel configuration, and boundary conditions serve to cause differences in water surface elevations for the same discharge. As mentioned earlier in this report, discharge measurements are not absolute and an error of 5 percent or so compared to the "true" discharge is not unusual. Consequently, a rating curve is usually a best-fit relationship drawn through the accumulated data points. Similar recurrences of past discharges may result in stages somewhat higher or lower than the past stages recorded.

c. Usefulness of rating relationship. As one moves upstream or downstream from a gage site, the rating relationship provides less useful information. Synthesizing a rating relationship at ungaged locations normally requires computations of water surface profiles using a computer program. Consequently, a measured rating relationship is most useful at the gaged site for calibrating a river hydraulics model to reproduce known stages for measured discharges.

6-3. Steady-Flow River Hydraulics

The use of available gaged data alone is seldom sufficient for a flood study. A flood study is normally performed for a length of stream, with flood information necessary throughout the reach, not just at a gage site. This requires the calculation of water surface elevations at many locations along the reach. This establishes a water surface elevation profile for a given flood discharge, and is usually accomplished by using a computer program. These programs assume steady, gradually varied flow with a rigid boundary. A steady flow assumption postulates that the discharge changes so slowly with time that it can be assumed to be constant for the computation period. A gradually varied flow assumption states that depth and velocity for a specific discharge change in very small increments with distance as calculations proceed along a reach of river. For the vast majority of all water surface profile computations, these two key assumptions are quite acceptable and form the basis for steady-flow river hydraulics analysis (USACE 1990b).

a. Basic principles.

(1) Given the above two assumptions, steady-flow river hydraulics analysis utilizes the conservation of mass (continuity) and energy principles (Chow 1959). Figure 6-2 shows the basic equations for computing steady, gradually varied profiles.

(2) The conservation of energy equation states that energy cannot be created or destroyed. Changes in energy levels from one point to another in the stream system occur when flowing water loses elevation in overcoming friction effects between the two points. These energy losses are primarily from boundary friction, with some additional losses due to cross-section geometry fluctuations. Changes in area and velocity at each point are calculated by the continuity equation. Velocity at each point is found by use of Manning's equation.

(3) Methods and procedures for steady, gradually varied river hydraulics analysis are well-founded and understood. However, application of the technique requires the acquisition of considerable input data.

b. Geometric data.

(1) Introduction.

(a) The geometry of the stream reach under investigation must be defined. This requires surveying and mapping work. Aerial contour mapping gives the most information on the overbank areas, with supplemental channel cross sections taken in the field. Crossing obstructions must also be described. Although acquisition of this survey data is expensive, the data have a variety of uses besides hydraulic modeling, including elevation of structures for economic analysis and topographic information for structural flood control measures.

(b) Cross-sectional locations coincide with the calculation steps of the finite difference profile analysis process. They are commonly located for the physical and hydraulic reasons listed below.

- Where distinct changes in stream bed slope occur.
- Immediately upstream and downstream of locations where changes in discharge occur.
- Where variations in geometry, including abrupt expansions and contractions in flow geometry, occur.
- Where variations in channel and overbank resistance occur.
- At bends in the stream to ensure that channel and overbank reach lengths are correctly defined.

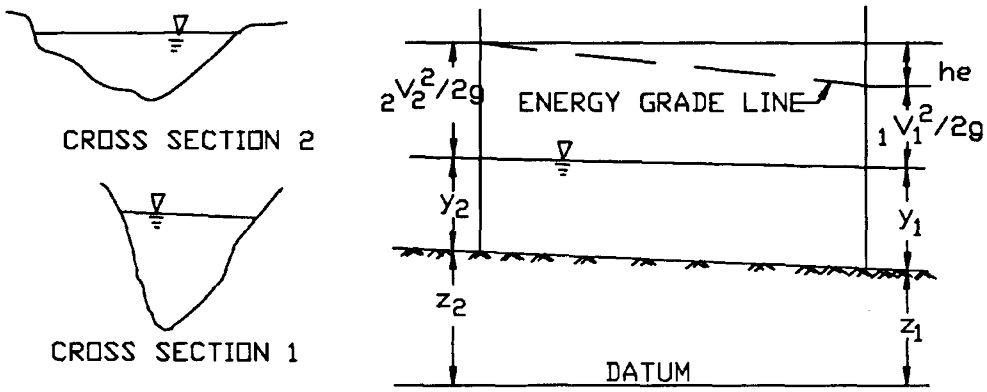
(c) Interpolated cross sections may be required to provide sufficient computation points to accurately compute the energy loss (USACE 1986).

(2) Friction loss coefficient data. Loss coefficients are determined by the hydraulic engineer from field inspection of the study reach, comparison with published references, and by engineering judgement. Friction loss coefficients (Manning's n) are often used as the main adjustment parameter to improve the calibration of the hydraulic model.

(3) Discharge data. Discharge is read from discharge-frequency relationships that are determined by hydrologic modeling or statistical analyses, as described in previous chapters.

(4) Other data. Other needed information (expansion-contraction losses, flow regime, boundary conditions, etc.) usually require minimal time and effort to develop.

(5) Calibration data. Models using gradually varied, steady-flow assumptions are calibrated to reproduce known water surface elevations with known discharges at gage sites. The main calibration technique is the adjustment of "n" values, the hydraulic parameter which contains



Continuity Equation

$$Q = A_1 \times V_1 = A_2 \times V_2 \quad \text{Equation 1}$$

Manning's Equation

$$V = (1.486/n) \times R^{0.67} \times s_f^{0.5} \quad \text{Equation 2}$$

Energy Equation

$$z_2 + y_2 + \alpha_2(V_2^2/2g) = z_1 + y_1 + \alpha_1(V_1^2/2g) + h_e \quad \text{Equation 3}$$

where

Q = discharge, cubic feet per second
A = cross-sectional area, square feet
V = average velocity, feet per second
n = Manning's coefficient of friction, dimensionless
R = hydraulic radius (area/wetted perimeter), feet
s_f = friction slope, feet/foot
z = elevation of channel invert
y = channel depth, feet
g = gravitational constant, feet/sec/sec
z + y = water surface elevation, feet above a datum (usually mean sea level)
h_e = energy loss between sections, feet

α = velocity distribution coefficient, dimensionless

$\alpha(V^2/2g)$ = velocity head, feet

Figure 6-2. Gradually varied, steady-flow equations

contains the most uncertainty. Without sufficient gaged data, the hydrologic engineer attempts calibration by reproducing high-water marks from one or more actual floods, as obtained from interviews with local residents.

c. Applications. Gradually varied steady-flow techniques are the primary method of determining flood elevations for most hydrologic analyses and have a wide applicability for Corps hydrologic studies. Common applications include:

(1) Development of flood profiles for land use planning for flood insurance/floodplain studies.

(2) Development of flood profiles for urban and rural flood damage evaluations.

(3) Determination of changes in flood elevations due to structural flood control improvements.

d. Limitations. Gradually varied, steady-flow analysis can be considered applicable as long as (1) the discharge is steady with time and gradually varied with distance, (2) the discharge can be considered one-dimensional (a single elevation for one cross section), (3) the river slope is small (less than one in ten, so a hydrostatic pressure assumption is correct), and (4) each cross section is rigid (no significant scour or deposition). When any of these assumptions is not acceptable, other techniques must be used.

e. Need for advanced analysis. The complexity of determining flood elevations increases significantly when more advanced analysis techniques are required. These techniques are usually necessary when the discharge changes rapidly with time, thereby causing flow momentum to become significant. The kinds of situations requiring more detailed computational analysis include:

(1) Dam break analysis.

(2) Flood elevation predictions at multiple points and times for very mild slopes.

(3) Where downstream boundary effects are changing, such as those caused by tidal fluctuations.

(4) Where flow is rapidly varying, such as during hydropower operations, during locking operations, sudden opening or closing of gates, abrupt start and stopping of pumping plants, and flash floods on small streams.

6-4. Unsteady-Flow River Hydraulics

The next higher level in river hydraulics computational difficulty is the application of one-dimensional unsteady, varied flow analysis. One-dimensional means that one elevation is still characteristic of each computational point, or cross section; however, now the computations are being performed at all time periods as well as all points along the center line of the river. Changes along the channel length can also be gradually varied with this technique. Figure 6-3 illustrates the difference between the results of steady versus unsteady analysis. Differences between steady and unsteady flow analysis can also be visualized by imagining one is standing on a riverbank and observing the moving water. Steady-flow analysis is adequate when the water surface appears to rise and fall uniformly, without any observation of curving streamlines. Unsteady flow analysis is necessary if one would observe an advancing wave front moving downstream, with obvious curvature to the streamlines. Figure 6-4 further illustrates this concept.

a. Hydrologic versus hydraulic routing. Unsteady flow analysis is often referred to as hydraulic routing, because elevations, velocity, and discharge information are being calculated at all time periods and for each desired location. Unsteady flow analysis can be broken into two groupings: hydrologic or hydraulic routing. Hydrologic routing is discussed in paragraph 5-5*a*. Hydraulic routing includes both continuity and momentum conservation and yields information on velocity, discharge, water surface elevation, travel times, etc. at each computational point. This section will be concerned only with hydraulic routing, or gradually varied unsteady flow.

b. Basic principles. Unsteady flow analysis is required when the inertial effects of flow, resulting in unbalanced momentum, are large enough that they can no longer be ignored. The listing of unsteady flow situations in paragraph 6-3*e* represents many of these cases. The basic equations for one-dimensional unsteady flow analysis are given in Figure 6-5. As seen, the difference between steady and unsteady flow analysis is the inclusion of the local acceleration term in Equation 2, along with the more rigorous presentation of the continuity equation in Equation 1. Solution of the unsteady flow equation is difficult and requires significant computational operations, necessitating a high-speed computer. A number of unsteady flow analysis programs are available, e.g., Fread (1978).

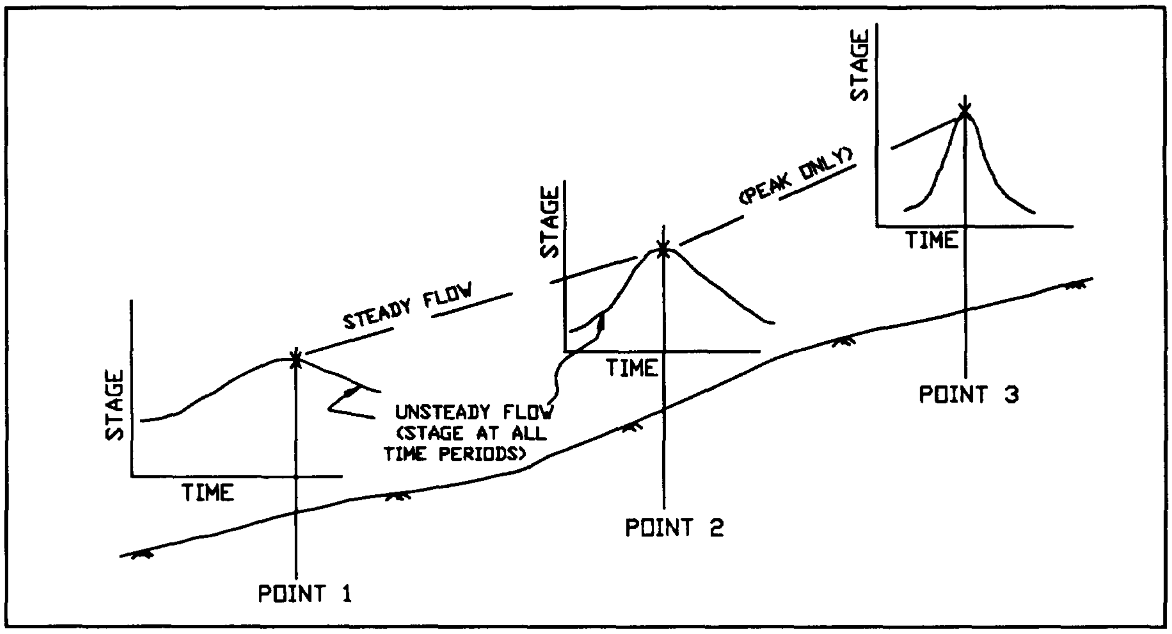


Figure 6-3. Steady- versus unsteady-flow analysis

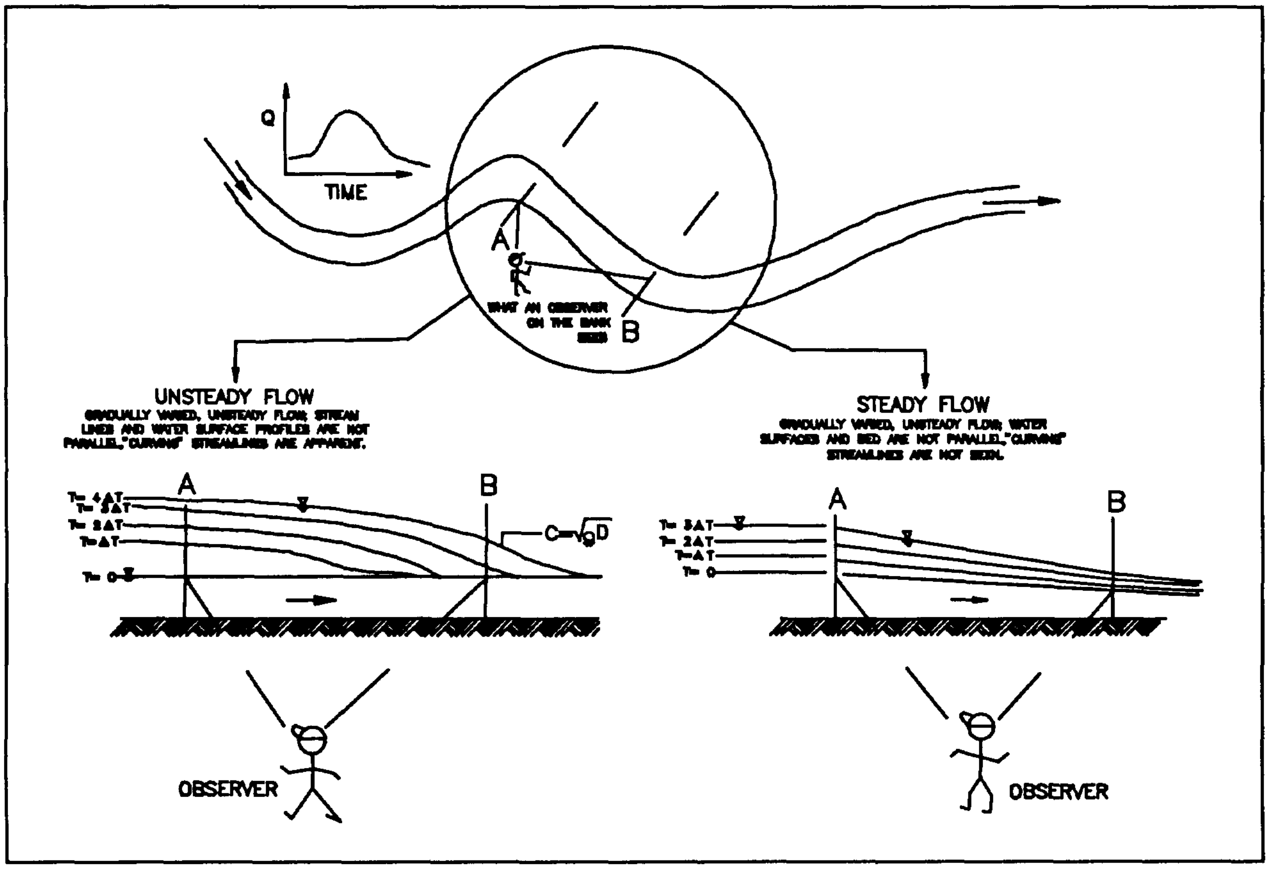
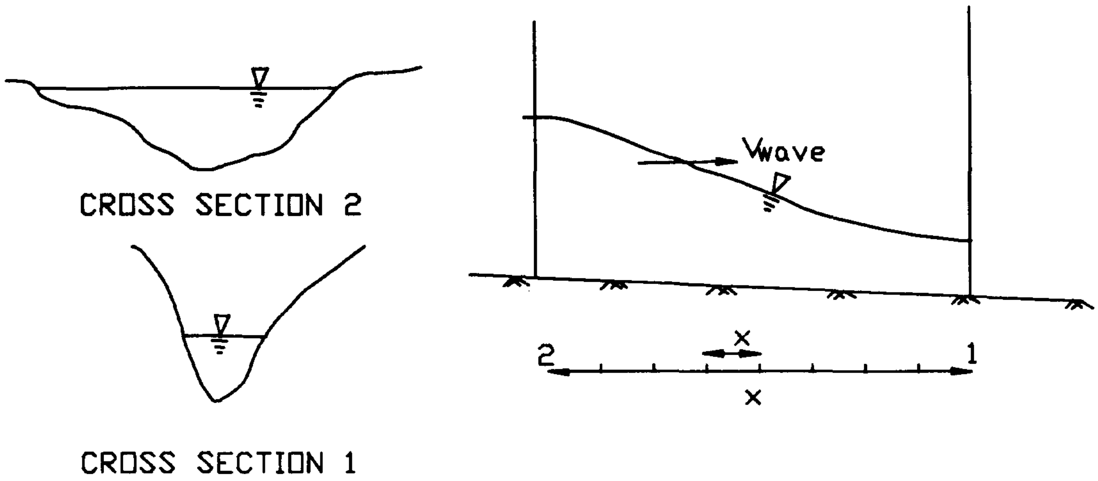


Figure 6-4. Visualization of unsteady and steady flow

Equations for a Wide Channel with No Lateral Inflow



Continuity Equation

$$y\,(\partial v/\partial x) + v(\partial v/\partial x) + (\partial y/\partial t) = 0 \quad \text{.............................. Equation 4}$$

Momentum Equation

$$s_f = s_o - (\partial y/\partial x) - \frac{v}{g}(\partial v/\partial x) - \frac{1}{g}(\partial v/\partial t) \quad \text{........................ Equation 5}$$

where

y = depth, feet
v = velocity, feet per second
x = distance, feet
t = time, minutes
g = gravitational constant, feet/second/second
s_o = channel invert slope, feet/foot
s_f = friction slope, feet/foot

∂ = partial derivative, "change with respect to"; i.e., $\partial v/\partial x$ is the change in velocity with respect to distance

Figure 6-5. Unsteady, gradually varied flow equations *c. Data requirements.* Requirements are similar to those of steady, gradually varied methods, with the exception of boundary conditions and calibration data. For hydraulic routing, the boundary conditions must be completely described as a stage or flow hydrograph. It is generally preferable for the stream geometry to be better defined (more cross sections) than for steady-flow methods. Calibration data for unsteady flow are also more extensive, requiring stages and/or discharges at a number of different time periods. This information is usually more costly to obtain than calibration data for steady-flow applications.

d. Applications. The common applications of one-dimensional unsteady flow analysis have been previously stated in paragraph 6-3*e*. A number of unsteady flow models have been developed in recent years and utilization of these types of models has been made easier. A higher level of engineering expertise is still necessary, however, to use these techniques. As most situations for calculating flood elevations use steady, gradually varied flow, fewer individuals are sufficiently knowledgeable and experienced to properly apply and interpret the results of unsteady-flow models.

e. Limitations. Since this method accounts for more of the physical processes that are occurring than steady-flow analysis does, there are fewer limitations. As the next level of analysis is still more complex, situations where one-dimensional unsteady flow solutions are computationally inadequate are fortunately few. Situations requiring a higher level of computational analysis include:

(1) Analysis of flow patterns in bays and estuaries, where velocities and elevations may vary in the horizontal and vertical directions.

(2) Cases in which a one-dimensional assumption cannot model the elevations with sufficient accuracy; i.e., multiple bridge openings across a wide floodplain, major river junctions, etc.

(3) Analysis of flow patterns around dike fields, hydropower plants, and cofferdams.

6-5. Multi-Dimensional River Hydraulics

Although nearly all flood elevation determination requirements can be satisfied with either one-dimensional steady or unsteady flow models, certain specialized problems occasionally require a yet more sophisticated and complex modeling approach. Use of multi-dimensional river hydraulics is necessary when one can no longer assume that a single elevation at each computational point (cross section) is appropriate. This problem requires the use of a two-dimensional (2D) model, where hydraulic properties vary across the section as well as along the length of stream, or of a three-dimensional (3D) model, which would include changes of hydraulic properties in the vertical direction. Three-dimensional computer models are currently under development and testing and are not yet fully available. Three-dimensional efforts have largely been through the application of physical models, the subject of paragraph 6-7. Only 2D modeling is addressed further in this section (EM 1110-2-1415).

a. Principles. Multi-dimensional models are usually applied to evaluate a short reach of river, where average depth is small compared to the average stream width. Because of the relative shallowness compared to length and width dimensions, differences in the vertical for hydraulic properties are often averaged to obtain a 2D solution. This greatly simplifies the work effort. The basic equations to solve 2D unsteady-flow problems are lengthy and are not included here. Assumptions inherent in the application of this technique include: gradually varied flow, constant water density, and a rigid boundary (or one that is changing insignificantly).

b. Data.

(1) General. Data requirements are considerably greater than for previous methods. It is normally insufficient to utilize a data set developed for steady or one-dimensional unsteady flow in a multi-dimensional model.

(2) Geometry. Geometry is usually derived from map data. Close interval contour mapping is most desirable, with 0.5-foot intervals often used. Since most applications of 2D models are for detailed analysis of a short reach of stream, this type of topographic information is usually feasible.

(3) Turbulent exchange coefficients. Turbulent exchange coefficients, used for modeling eddy losses, are required in addition to other coefficients such as Manning's n.

(4) Velocity. Velocity and velocity direction measurements are needed. As vertical velocities in a 2D model are depth averaged, these prototype measurements also must be depth averaged. Depth, water surface elevation, and velocity data at many points in the distance-time grid must be obtained.

(5) Acquisition. Data required for 2D models are site-specific and usually developed through a data collection program. Data acquisition is a considerable cost for 2D modeling.

c. Applications. Often, use of a multi-dimensional model requires contracting with a Corps Lab or a private consultant to develop the input data and operate the model. Considerable start-up expense and time are required to educate a new user of a multi-dimensional model, although if additional applications in the near future are foreseen, in-house capability should be further investigated. Figure 6-6 shows a typical application of 2D modeling. Other examples were indicated in paragraph 6-4*e* with additional applications, including the following:

(1) Channel deepening. Investigating the effects of deepening a ship channel on velocity patterns and shoaling.

(2) Encroachment. Investigating the effects of major encroachment into a river channel on flow patterns and water surface elevations.

(3) Velocity and flow patterns. Investigating the velocity and flow patterns of water entering and leaving a wide floodplain from the river channel.

d. Limitations.

(1) Practical limitations. The practical limitations of 2D models are in their application and in the user skills required. Because of input data needs and computational requirements, applications are normally for a short (1 mile or less) reach of river. Qualified personnel skilled in utilizing 2D models are often more difficult to obtain.

(2) Technical limitations. Technical limitations include the necessary assumptions of gradually varied flow and of insignificant changes caused by sedimentation

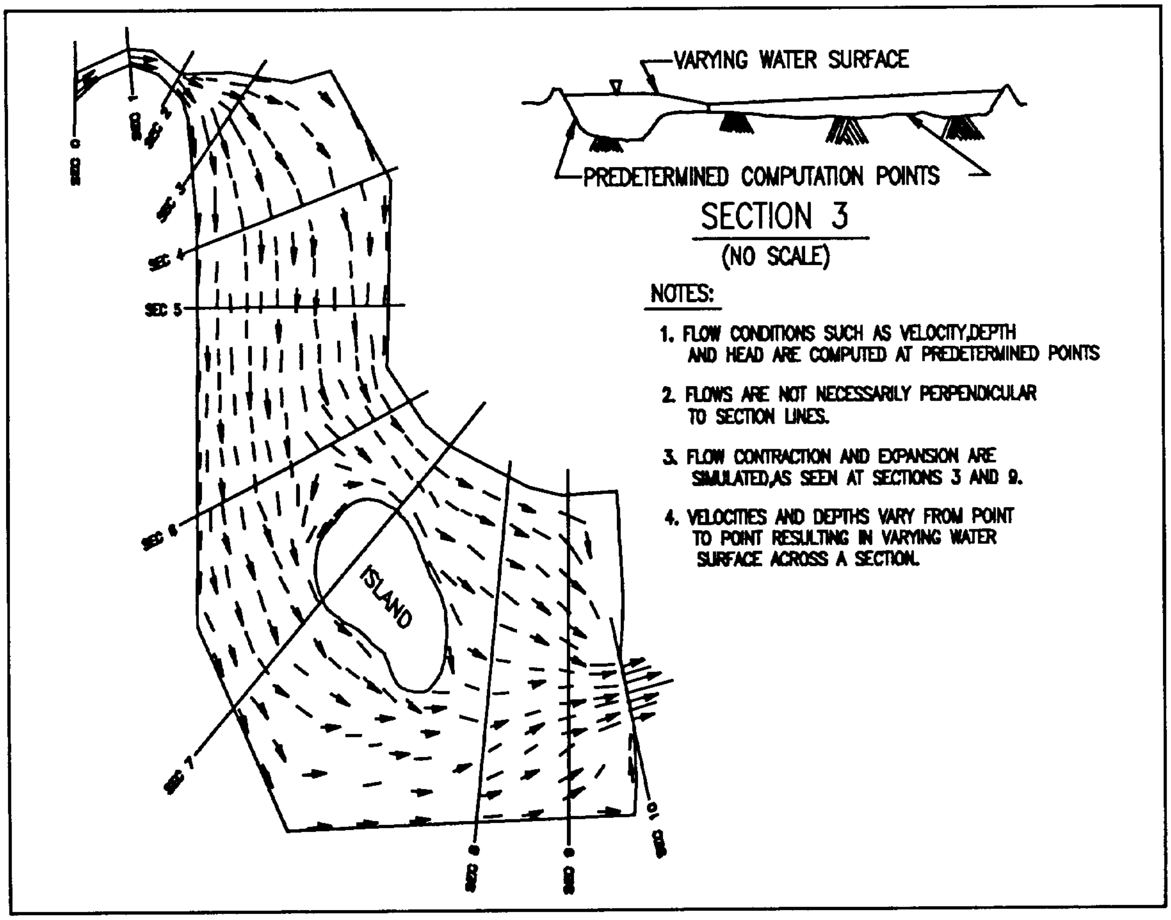


Figure 6-6. 2D flow representation in Cache Creek settling basin

or erosion. If the vertical depth components cannot be averaged and a 3D simulation is necessary, physical models must be employed.

6-6. Mobile Boundary Hydraulics

Mobile boundary analysis is necessary when the assumptions of a rigid boundary are no longer valid. For most streams, a rigid boundary assumption is acceptable for a design flood, as channel and overbank geometry are typically slow to change in response to the sediment transport characteristics causing scour and deposition. Over time, however, the stream does respond to changes in its sediment regime by adjusting its cross-sectional geometry, stream slope, bed material composition, sediment load, etc. Mobile boundary analysis is thus generally concerned with the longer-term trends over the life of a flood reduction or navigation project. The complexity level of a mobile boundary analysis is similar to that of a one-dimensional unsteady flow analysis (USACE 1991a).

a. Basic principles.

(1) Assumptions. The most common mobile boundary analysis incorporates a number of important assumptions, including:

(a) The analysis is one-dimensional (single water surface elevation at each point).

(b) The channel slope is small.

(c) Sediment-water density is constant.

(d) Manning's n value applies.

(e) Gradually varied flow occurs along the stream channel.

(2) Models. These assumptions may be incorporated into mobile boundary models. The models commonly combine a gradually varied steady flow analysis with sediment transport calculations at the end of each flow period. Changes in channel geometry are calculated before starting the next computation period. Computations would normally take place over several years of discharge data to identify long-term trends occurring in channel geometry and water surface elevations.

(3) Results. The model may be calibrated and operated to give information like channel invert and water surface profiles. The most valuable information is the identification of trends and the comparison of the effects of a flood mitigation component on the sediment regime. Model results can be used to evaluate and compare with- and without-project conditions. A typical application would determine how fast a channel improvement, or a reservoir, loses capacity due to sediment deposition.

b. Basic data. A mobile boundary analysis typically requires the most data of any of the methods of analysis described in this chapter, necessitating hydrologic, geometric, and sediment information.

(1) Hydrologic data. Discharge data are needed for all flow periods, from flood to drought. The time duration associated with each of the actual discharges is also necessary. The discharge and time data are often converted from a continuous, smooth hydrograph to a histogram, or bar graph, averaging smaller flows over long time periods. The water temperature is also important, as it has a significant effect on how fast small particles settle in the water column.

(2) Geometric data. Channel cross sections and reach lengths are required, similar to the information necessary for a gradually varied steady flow model. Geometric data are normally less extensive than for a water surface profile analysis, however. Longer distances between sections are tolerable, and bridge sections are not normally included. Manning's n values are used for boundary friction estimates.

(3) Sediment data. The sediment composition of the channel section at each point is needed, with this data coming from borings and/or "grab" samples by the engineer in the field. The amount and composition of sediment flowing in the water column for a wide range of discharges must be determined for the main channel and any significant tributaries. This information is best obtained from actual measurements of sediment load at gage locations, but may be derived in the absence of any real data. The unmeasured, or bed load (that moving within a few inches of the channel surface) must be estimated and included. Geometric and channel sediment composition data require measurement at two or more widely separated time periods to provide calibration information for the sediment transport model.

c. Applications. The primary application of sediment transport models is to evaluate with-project against without-project conditions to determine long-term trends affecting project design and operation and maintenance of the project. Typical applications include:

(1) Determining sediment rate in reservoirs and length of useful life.

(2) Determining rate and location of deposition in channel modifications to estimate frequency of dredging and sediment removal, thereby maintaining design channel capacity.

(3) Determining deposition along a levee over time and the corresponding effects of this deposition on increasing flood heights, thereby decreasing the levee protection.

(4) Maintaining adequate depth at all times at locations where this is important, such as for navigation channels.

(5) Monitoring locations where great changes in channel geometry occur during a flood, such as flow across an alluvial fan.

d. Limitations. Limitations for one-dimensional sediment transport analysis are the same as for one-dimensional unsteady-flow problems. Sediment scour and deposition that cannot be assumed reasonably uniform at a channel section require multi-dimensional or physical model testing. Scour evaluations around cofferdams, navigation locks, or similar structures usually require a higher level of analysis.

6-7. Use of Physical Models

Physical models are employed when mathematical models cannot adequately simulate the full range of effects caused by the component or problem under study. Three-dimensional analysis most often results in physical model testing. These models are normally expensive to build and operate, and require particular engineering expertise to utilize. Typical applications of physical modeling include:

a. Analysis of river navigation improvements on channel geometry and sediment characteristics.

b. Verification/modification of hydraulic design of flood reduction components to minimize operational problems and optimize performance under all adverse conditions.

c. Simulation of navigation through potential hazardous river reaches.

d. Water quality simulations, dispersal of pollutants, and temperature stratification in reservoirs.

6-8. Comparison of Flood Elevation Determination Methods

Although comparisons between the various methods have been made throughout this chapter, additional comparisons are provided in the following tables. Table 6-1 illustrates when the various methods are usually appropriate for different reporting levels, while Table 6-2 gives a rather subjective appraisal of the differences in experience level, time, money, data needs, and computer requirements for the various techniques.

Table 6-1
Model Usage for Hydrologic Engineering Studies

Study Stage	Existing Data & Criteria(1)	GVSF	MB	GVUSF	Multi-D	Physical
Reconnaissance	X	X				
Feasibility		X	X	X(2)	?(3)	
Reevaluation		X	X	X(2)	?(3)	?
DM					X(4)	X(5)

(1) Existing data and criteria = available reports, U.S. Army Engineer Waterways Experiment Station (WES) criteria, regional relationships for depth frequency, normal depth rating relationships, etc.; GVSF = gradually varied steady flow; MB = mobile boundary analysis; GVUSF = gradually varied unsteady flow, multi-dimensional analysis, Physical = physical models (by WES or similar agency).

(2) Use is possible, but unlikely, on most flood control studies.

(3) ? Possible, but very unusual--very dependent on problem being analyzed.

(4) Typically employed to evaluate design performance for a short reach of river, or in the immediate vicinity of a specific project component, or refine the hydraulic design of a project component.

(5) Typically performed to evaluate 3D or other specific conditions where mathematical modeling results are considered inaccurate.

Table 6-2
Qualitative Comparison of Different Analysis Technique Requirements[1]

Analysis Technique	Hydraulic Engineer's Time	Special Technical Expertise Requirements	Computer Requirement	Data Requirement	Study Cost
Existing or simplified criteria	1	None[2]	0,1	1	1
Gradually varied, steady flow	10	None[2]	10	10	50
Gradually varied, unsteady flow	30	Some[3]	20	20	100
Mobile boundary analysis	30	Some[3]	40	30	150
Multi-dimensional analysis	40	Many[4]	100	50	200
Physical modeling	100	Severe[5]	---	100	500

[1] Comparisons among techniques would be as follows: multi-dimensional analysis would require four times the amount of engineer time and five times the amount of data compared to the gradually varied, steady-flow technique.

[2] "Average" hydraulic engineer can adequately handle this technique.

[3] "Average" hydraulic engineer has limited experience in these techniques.

[4] "Average" hydraulic engineer has no experience in this technique, specialized training/assistance by consultants may be necessary.

[5] Would require the use of WES or similar consultant.

Chapter 7
Hydrologic Engineering Requirements for Flood Damage Reduction Measures

7-1. Overview

This chapter provides an overview of the hydrologic engineering analyses necessary for the major structural and nonstructural measures of flood damage reduction studies. The types of analyses and hydrologic methods described in the previous chapters are used to show the analysis requirements for different types of measures.

7-2. Without-Project Conditions

a. Flood damage analysis. Corps of Engineers flood damage reduction analyses for different projects, both structural and nonstructural, are similar in method. The first step is the analysis of the discharge or stage versus frequency of flooding relationships at key points in the stream system for the existing or base project conditions. This step is repeated for at least one time in the future, assuming future land use conditions will result in changing discharge/stage versus frequency relationships. The development of existing and future, without-project hydrologic and hydraulic relationships is critical to establish the magnitude of the flood problem so that flood damage analyses may be performed. The flood damage analysis provides insight as to the location and the amount of existing and future expected damage, and therefore the amount of project costs that one could spend to mitigate the flood damage.

b. Hydrologic engineering studies. Hydrologic engineering studies normally require considerable time establishing the existing and future without-project relationships by performing rainfall-runoff, frequency, river hydraulics and reservoir operation studies. Specific methods used during the analysis of each flood damage reduction measure are based on the amount of data available, the complexity of the study area, and the needs of the Interdisciplinary Planning Team (IPT).

7-3. Screening of Alternatives

a. Structural measures. Following development of without-project conditions, analysis of different structural and nonstructural flood damage reduction measures is performed. Not all measures presented in this chapter would likely be evaluated in a specific study. Rather, the IPT, including representatives of various Corps disciplines and the local cost-share partner, would identify one or more likely feasible measures and plans to evaluate for the study area. Reservoirs are practical because they reduce flooding at downstream locations; however, they are often the most costly alternative and the most difficult to economically justify. If flood damage reduction is for a single site along a stream, a local protection project (channel modification, levee, or diversion) would be examined. These projects are normally less costly than a reservoir and provide site-specific protection to a single area. However, local protection projects can have adverse effects on flooding elsewhere.

b. Nonstructural measures. Nonstructural measures are required to be analyzed as a means of reducing flood damage (Section 73 of Public Law 93-251). Nonstructural alternatives may be examined with structural solutions, or by themselves. These solutions are typically the least expensive, but often provide the least flood damage reduction to the area. If the existing/future without-project damages are small, nonstructural solutions may be the only ones feasible.

7-4. Reservoirs

The intent of flood control reservoirs is to store and gradually release upstream flood runoff after downstream flooding is over. Reservoirs are practical flood damage reduction solutions because they reduce flooding throughout the downstream river system, although the effects of the reservoir decrease as the distance from the reservoir increases. A flood control reservoir is analyzed to accomplish flood damage reduction and to ensure safety of the structure in extreme floods.

a. Flood control.

(1) Flood control analysis determines the storage volume in the reservoir that should be reserved to control flooding. The hydrologic modeling effort requires varying magnitudes of floods to be routed through the reservoir and to downstream damage centers. The analysis yields with- and without- reservoir discharge-frequency relationships. Figure 7-1 illustrates this analysis.

(2) Historic data for the routings are preferred and are usually available for sites in larger rural areas. Urban reservoirs usually have little or no data and synthetic rainfall-runoff modeling is normally employed. Discharge is converted to stage at downstream locations to determine project damage. The difference between with- and without-project damage is the flood inundation reduction benefits attributed to the project.

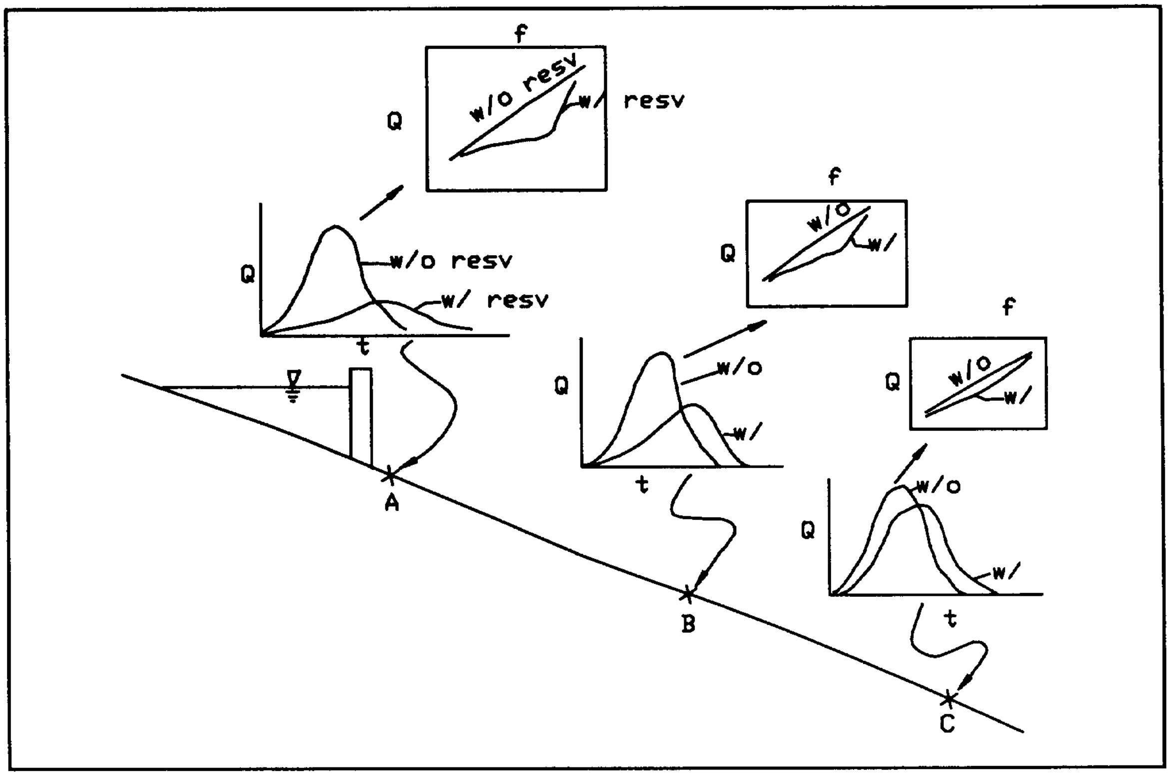


Figure 7-1. Effects of a reservoir

b. Safety. A safety analysis specifically determines the height of dam and size of spillway necessary to ensure that essentially no risk of dam overtopping exists. For high hazard dams, where overtopping would cause a downstream catastrophe, a very high safety design standard - typically the Probable Maximum Flood should be selected.

7-5. Local Protection Projects

Channels, levees, and diversions are considered local protection projects. Protection of a specific damage center is accomplished with each, although channelization, levee systems, and major diversions have been constructed to protect a series of damage centers. Each project reduces the severity and frequency of flooding to the protected area. They may, in unusual circumstances, also increase flooding immediately adjacent to the protection area.

a. Channels.

(1) New channels or modifications to existing channels attempt to decrease flood stage by increasing channel efficiency. The effect of a channel project is illustrated in Figure 7-2.

(2) Channelization is a typical measure for urban flooding situations. An improved channel can provide a smoother flow path (less boundary friction), increase the cross-sectional area of the channel, improve the efficiency of the channel, or combinations of these changes. If an extensive reach of channelization is to be constructed, the effects of these changes will be to increase the severity of downstream flooding by accelerating the flood hydrograph through the reach, causing higher peak discharges downstream. The hydrologic analysis must address this problem, as well as the beneficial effects of channelization. River hydraulics dominate channelization studies, with storage routing becoming more important in determining adverse effects as the channel reach becomes longer.

b. Levees and floodwalls.

(1) Levees and floodwalls prevent floodwaters from entering the protected area until the design event is exceeded and the levee or floodwall is overtopped. Figure 7-3 illustrates the usual effect of a levee or floodwall for the area protected and for unprotected areas upstream.

(2) River hydraulics is the major analysis component for evaluating levee grade and alignment, as well as

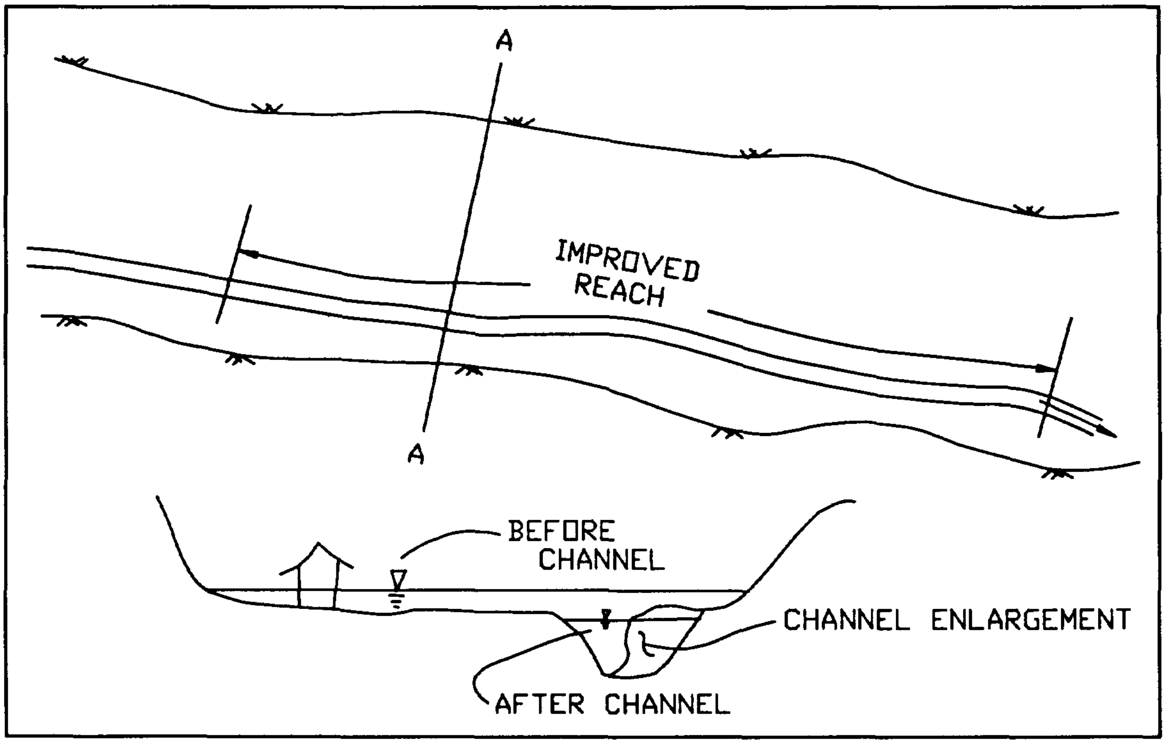


Figure 7-2. Effects of a channel

certain adverse effects. Upward shifts in river stage for the same discharge may occur when the levee or floodwall confines the flood to areas outside the protected area. This effect may extend upstream from the levee unit, inducing additional flooding to unprotected areas.

(3) An extensive levee project or a system of levee projects can remove significant floodplain storage. This lost storage can result in increases in peak discharge downstream of the levee(s). Hydraulic routing is required to satisfactorily evaluate storage effects on flood magnitude.

(4) Levees have the potential in unusual circumstances for inducing flooding, both upstream and downstream of the protected area. Thus, the hydrologic design should minimize these adverse effects as much as practical.

(5) Levees and floodwalls greatly reduce the direct threat of flooding by the main river or lake. However, the nature of this solution may introduce a secondary flood problem, which is the remaining or residual interior area flooding. This flooding results from interior ponding by rainfall on the leveed interior, blockage of existing flow paths, and seepage water through the levee during high interior stages. During lengthy high exterior stages, interior flooding caused by interior ponded water could negate much of the damage prevented by the levee or floodwall. Therefore, an interior flood control analysis is an integral part of any levee or floodwall project. Rainfall-runoff analysis and storage operations are the dominate features of interior flood control analyses. These analyses are complex because they must address the joint probability of high river stages and of significant interior runoff. Period-of-record analysis is preferred, but gaged data are seldom available for an accurate application. Hypothetical events are often used. Interior flood control studies are among the most difficult hydrologic engineering analysis. EM 1110-2-1413 provides additional information on these complex studies.

c. Diversions. These components remove water from the main channel upstream of the area to be protected, and usually reintroduce the diverted water downstream of the area. Figure 7-4 illustrates the impact of a diversion. River hydraulics is the dominate means of analysis. The potential exists for adverse effects on flood heights downstream of the diversion reentrance. This problem would be analyzed through storage operations.

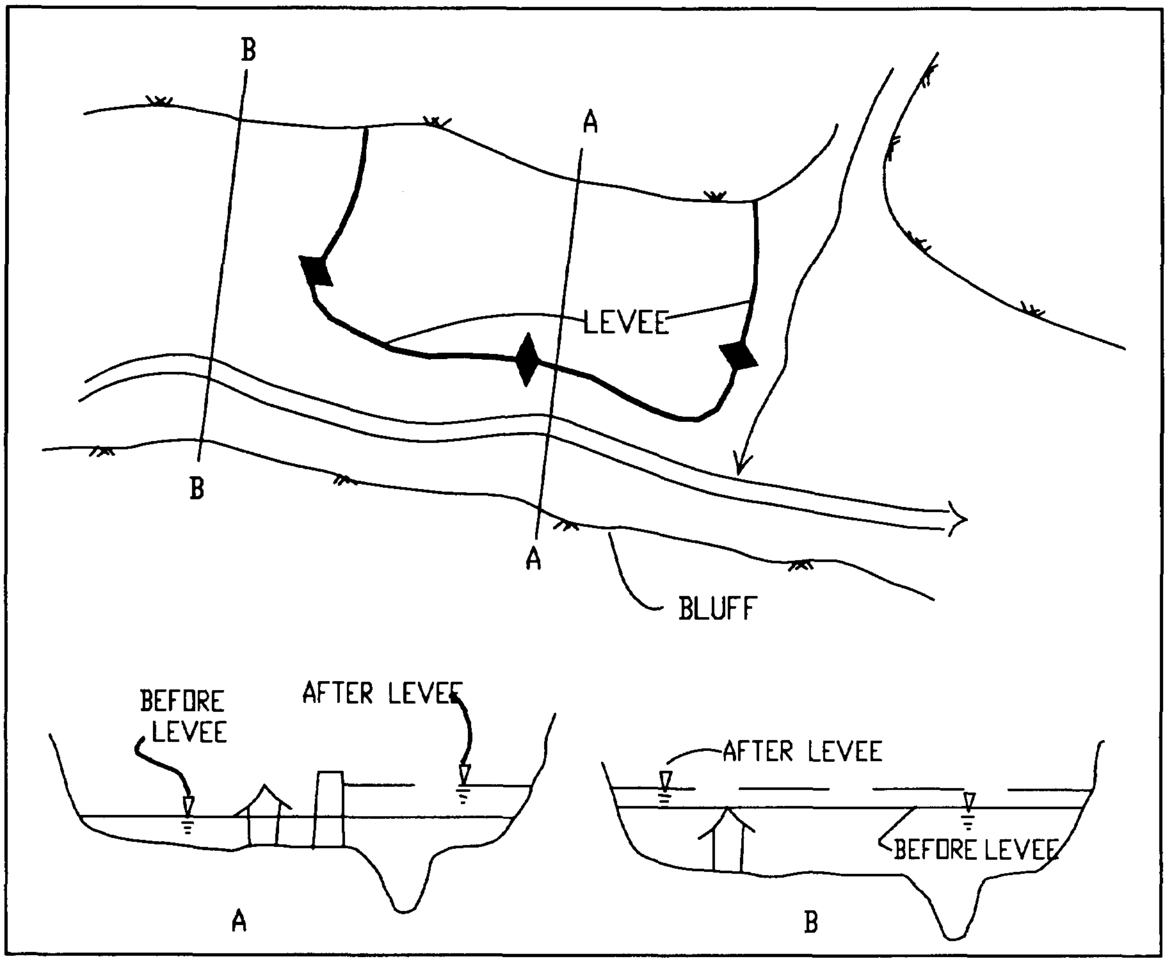


Figure 7-3. Effects of a levee/floodwall

7-6. Nonstructural Measures

Structural solutions modify the watershed's hydrology/hydraulics to reduce flood damage to the protected area(s). Nonstructural measures operate in a reverse fashion, by reducing the damage potential in the flood-prone area without changing the hydrology and hydraulics of the watershed. Rainfall-runoff, frequency, river hydraulics, and storage operations may be utilized in development of existing hydrologic/hydraulic conditions. Nonstructural measures include: floodplain management and flood insurance, floodproofing, relocations, and flood warning-preparedness planning. A reference (Hydrology Sub-Committee 1985) further describes nonstructural analyses.

a. Floodproofing. This alternative minimizes damage by raising the elevation where floodwaters first enter a structure. Usual means of floodproofing are the installation of waterproof shields to doorways and basement windows. Two feet is the practical maximum depth for floodproofing before the pressure of water on exterior walls could result in structural failure. Floodproofing applications are most suitable when first-floor flooding is more frequent than a 5-percent chance event, and the difference between frequent and infrequent flood elevations is 1 to 2 feet.

b. Relocation. This alternative refers to permanently moving flood-damageable items to a higher elevation (second floor, etc.) or moving the entire structure to higher ground. Moving the structure is most feasible when flooding of the first floor is more frequent than a 5-percent chance (20-year) event and the structure has sufficient value for relocation to be economically justified.

c. Flood warning-preparedness planning. This alternative refers to a formal system and plan for ascertaining that a flood threat is imminent and ensuring that

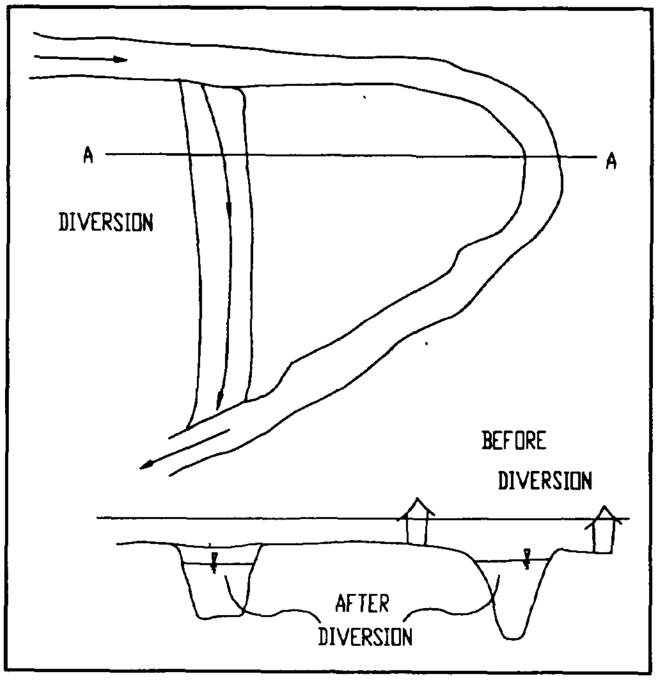


Figure 7-4. Effects of a diversion

appropriate actions are taken to minimize the threat to human life and decrease flood damage. The system usually includes rainfall and river gages upstream of the damageable area, a communication network to get the measured information to the appropriate personnel, a forecast model or other indicator to estimate flood severity and a detailed response plan to address all necessary actions. In addition to the need to identify flood stages at key locations for different flood events, adequate warning time is needed to take the appropriate actions. The measured information and the forecasting model must be accurate to minimize the threat of false alarms and resulting loss of confidence in the system. The complexity of the system must be commensurate with the ability of the sponsor to operate and maintain the system.

7-7. Floodplain Management Studies (FPMS)

These studies include floodplain management reports, flood hazard reports, and flood insurance studies. The FPMS program is intended to provide flood information for wise land use planning by local communities. Knowledge concerning future flood levels is instrumental in preventing development of flood-prone land. The hydrology and hydraulics performed for flood insurance studies also provide the technical basis for the purchase of flood insurance by individuals already occupying the floodplain. Flood insurance studies also require additional river hydraulics studies to establish a floodway, normally for the 1-percent chance event. The floodway specifies the portion of the floodplain that can be encroached without adversely affecting upstream flood heights more than a specified amount, normally 1 foot.

7-8. Hydrologic Analysis Requirements Summary

The type of technical studies required to analyze specific flood damage reduction measures are shown in Table 7-1. The information presented in Table 7-1 should be considered typical and may vary depending on specific study conditions and requirements.

Table 7-1
Hydrologic Analysis Needs for Flood Damage Reduction Measures[1]

Measure	Rain-Runoff(1)			Frequency			River Hydraulics		Storage Operations		
	A(2)	B	C	D	E	F	G	H	I	J	K
Reservoirs											
Flood Control	Y(3)	Y	Y	Y	Y	X	Y	Y	Y	Y	X
Safety	X	Y	X	N	N	N	Y	N	Y	Y	N
Channels	Y	Y	Y	X	Y	N	Y	X	X	X	N
Levees	Y	Y	Y	X	Y	N	Y	X	X	X	N
Interior Flood Control	X	Y	Y	N	Y	N	X	N	Y	Y	X
Diversions	Y	Y	Y	X	Y	N	Y	X	X	X	N
Floodplain Management	X	Y	N	X	Y	N	Y	N	X	N	N
Nonstructural	X	Y	N	X	Y	N	Y	N	X	N	N

(1) Dominate analysis types but not necessarily done for every case. For instance, historic frequency analysis would be done for interior flood control studies if the data were available.

(2) (A) Reconstitute historic floods, (B) develop hypothetical floods, (C) analyze the changed discharge/stage-frequency, (D) develop historic data, (E) develop from hypothetical events, (F) volume-duration studies, (G) elevation (stage) conversion from discharge, (H) sediment transport/deposition analyses, (I) routing operations, (J) facility sizing by routing, (K) sequential (period of record) routing.

(3) _Y_ Usually done (major part of study), _X_ Done less often (not a major part of study), _N_ not usually done.

[1] In general, not a detailed specification.

Chapter 8
Hydrologic Engineering Studies

8-1. General

Hydrologic engineering studies are a key part of an overall Corps flood damage reduction analysis. These studies form the technical basis to define: existing, or base without-project conditions; future without- project conditions; and the same conditions with project. The best technical hydrologic engineering analysis cannot be done independent of input by others. Two-way communications with members of the study team and all other concerned individuals and groups are important. This chapter briefly describes some of the input of others necessary to perform hydrologic engineering studies.

8-2. Study Design and Management

The hydrologic engineering study must be planned and detailed to allow the effective and efficient management of the technical work. Before any hydrologic modeling or analytical calculations are undertaken, considerable planning efforts should be performed (ER 1110-2-1460).

a. Scope of study. The overall scope of the study should be resolved early, ideally while preparing the Initial Project Management Plan during the reconnaissance study, through meetings with the entire IPT and the local sponsor. The time and cost required are a direct function of the study scope and amount of detail necessary to fully evaluate the range of problems and potential solutions. The hydrologic engineer should formalize these scoping meetings and ideas on addressing the problems through preparation of a Hydrologic Engineering Management Plan (HEMP). This plan would be reviewed and approved by the technical supervisor and furnished to the study manager. Unusual problems or solutions would make it wise to receive division review also. The HEMP is especially important to develop immediately after the reconnaissance-phase report has identified the problems for further analysis in (and prior to initiating) the feasibility-phase study.

b. Study team coordination. Every cost-shared feasibility study has an IPT, headed by a study manager. The team consists of working-level members from the areas of economics, hydraulics, geotechnical, design, real estate, environmental, and cost estimating. The local sponsor is also a member, although the sponsor may not wish to attend all IPT meetings. Frequent meetings of the IPT should be held (once a week to once a month), depending on the level of study activity and complexity. The advantage of frequent meetings lies in communication and the exchange of ideas between team members. The most successful studies are those having free and easy communication among team members.

c. Technical procedures. General technical procedures have been addressed throughout this document. The hydrologic engineer should select those procedures which adequately address the problem(s) under study. Choose the simplest technical methods that will do this---usually hydrologic modeling. Where more difficult methods appear necessary; i.e., 2D unsteady flow analysis, etc., these methods should be presented in the reconnaissance-phase report for higher level technical review and concurrence.

d. Quality control and review. The assurance of quality work and an adequate review come from both the technical supervisor and the IPT. The development of the HEMP and the supervisor's concurrence in the methods and procedures for study analysis give the hydrologic engineer a road map for the entire study. Frequent updates and consultations between the engineer and technical supervisor are important. With these steps followed, technical quality should be acceptable for the final report. Similarly, scoping of the problems and necessary hydrologic information supplied to other IPT members will be accomplished through IPT meetings and discussions. Unusual technical problems or policy issues may require the review of higher level authority.

e. Relationship with cost-share partner. The cost-share partner is a full member of the IPT and often provides technical assistance in many areas of the study. The partner also has valuable insights on the study area and its problems that may not be apparent to the study team. The cost-share partner should have as much (or as little) input and access to the planning and technical analysis as desired. All hydrologic engineering negotiations with the cost-share partner must involve the hydrologic engineer.

8-3. Level of Detail/Completeness

This subject was more fully addressed in earlier chapters and is only summarized here. For feasibility reports, hydrologic engineering must fully address the hydrology of the study watershed and the level of flooding throughout. Feasible solutions are formulated and evaluated. These requirements necessitate that hydrologic engineering be complete and final upon completion of the feasibility report. Refined hydraulic design should be the primary effort for the hydrologic engineer after the feasibility phase.

8-4. Documentation and Reporting

The technical analysis should be fully and completely presented in the portion of the feasibility report dealing with hydrologic engineering. A separate appendix for the hydrologic engineering effort is normally prepared. The appendix should present a complete and accurate description of the hydrologic engineering studies. A reviewer should be able to follow the logic and thought processes of the technical engineer and be able to reach the same conclusions concerning the make-up of the recommended plan. The appendix should describe the methods used, input parameters, calibration and verification processes, assumptions made, sensitivity tests performed, alternatives analyzed, plan selected, consequences of design exceedance of the recommended plan, and overall conclusions on project effectiveness. The complete, recommended project must be presented, including work required by other Federal and non-Federal agencies necessary to allow full functional operation of the recommended plan. The hydrologic engineer must also prepare the necessary technical studies outline and time and cost estimates for the Project Management Plan, which must also accompany the completed feasibility report.

8-5. Local Sponsor Coordination

Sponsor participation in the study process should be continuous. Study layout and scoping, IPT meetings and decisions, alternative evaluation and project selection, and report recommendations and review should all involve the local cost-share partner.

8-6. Summary

The Corps of Engineers has moved into a new era of feasibility planning, requiring a local partner to participate financially in the study process. These fiscal requirements by the Corps on the partner must also allow more participation of the partner in the study evaluation process. Further understanding of the hydrologic engineering analysis requirements during the feasibility phase by the local sponsor and others should allow for a better final product. This document is intended to provide an initial step in this direction.

Appendix A
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