
Hydrologic Engineering Requirements for Reservoirs – Part 2

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NOTE:

PART-2 IS BASED ON CHAPTERS 5 TO 9 OF THIS DOCUMENT



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**US Army Corps
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ENGINEERING AND DESIGN

Hydrologic Engineering Requirements for Reservoirs

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Engineering and Design
HYDROLOGIC ENGINEERING REQUIREMENTS FOR RESERVOIRS

Purpose. This manual provides guidance to field office personnel for hydrologic engineering investigations for planning and design of reservoir projects. This document is an update to the 1997 version. Changes were made to reflect models currently used by the USACE, updated sources of hydrologic and meteorological data, climate change requirements, clearer guidance on wave run-up and wind setup determination, and updated reference to other U.S. Army Corps of Engineers guidance. Refer to Section 1-4, Manual Updates, for more information.

Applicability. This manual applies to all Headquarters, U.S. Army Corps of Engineers (HQUSACE) elements and U.S. Army Corps of Engineers (USACE) commands having civil works responsibilities.

Distribution Statement. Approved for public release; distribution is unlimited.

References. Referenced publications are listed in Appendix A.

Discussion. This manual provides guidance to field office personnel for hydrologic engineering investigations for the planning and design of reservoir projects. The manual presents typical study methods; however, the details of procedures are only presented if there are no convenient references describing the methods. Also, publications that contain the theoretical basis for the methods are referenced. Many of the computational procedures have been automated, and appropriate references are provided.

FOR THE COMMANDER:



KIRK E. GIBBS
Colonel, EN
Chief of Staff

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Engineering and Design
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CHAPTER 1

Introduction

1-1. Purpose. This manual provides guidance to field office personnel for hydrologic engineering investigations for the planning and design of reservoir projects. The manual presents typical study methods; however, the details of procedures are only presented if there are no convenient references describing the methods. Also, publications that contain the theoretical basis for the methods are referenced. Many of the computational procedures have been automated, and appropriate references are provided. This manual incorporates by reference other USACE guidance documents, but unless expressly stated, this manual does not alter or supersede other USACE guidance. Additionally, this manual does not alter or supersede any law or binding regulation, or determine the authorized purposes of any USACE reservoir project; nor does it impose legal requirements on any entity.

1-2. Applicability. This manual applies to all HQUSACE elements and U.S. Army Corps of Engineers (USACE) commands having civil works responsibilities.

a. Scope. This manual provides information on hydrologic engineering studies for reservoir projects. These studies may also use many of the hydrologic engineering methods described in the manuals listed in paragraph 1-5. The hydraulic design of project features (e.g., spillways and outlet works) is not included here but is presented in a series of hydraulic design manuals.

b. Organization. This manual is divided into four parts. Part 1 (Chapters 1-4) provides basic hydrologic concepts for reservoirs including reservoir purposes and basic hydrologic concerns and methods. Part 2 (Chapters 5-9) describes hydrologic data and analytical methods. Part 3 (Chapters 10-13) covers storage requirements for various project purposes, and Part 4 (Chapters 14-18) covers hydrologic engineering studies.

1-3. References. Additional reference publications are listed in Appendix A.

1-4. Manual Updates. This EM supersedes EM 1110-2-1420 published on 31 October 1997. Major updates to this document are the following.

a. Computer software packages were updated to reflect models currently used by the USACE (e.g., HEC-1 updated to HEC Hydrologic Modeling System [HEC-HMS]). Other models approved by the USACE may be used but proprietary software will not be included in this document.

b. Methodologies were updated to include geographic information system (GIS) data and computer programs.

c. Sources of meteorological, GIS, and hydrologic data are referenced.

- d. Reservoir reallocations are mentioned and applicable guidance is referenced.
- e. Inflow design flood (IDF) development is discussed and the Engineering Regulation 1110-8-2, Inflow Design Floods for Dams and Reservoirs, is referenced as the primary guidance document.
- f. General background on site-specific probable maximum precipitation (PMP) is provided as well as sources for the development of these storms.
- g. Special reservoir operations and adaptive management are included.
- h. Climate change is considered and applicable guidance referenced.
- i. Figures have been updated using actual project data where possible.
- j. The wave run-up and wind setup analysis for dam freeboard calculations have been updated to include guidance from the EM 1110-2-1100 Coastal Engineering Manual and previously published engineering technical letters (ETLs).
- k. An appendix of websites for data references and hydrologic tools has been added.

1-5. Related Hydrologic and Hydraulic (H&H) Guidance.

a. USACE Engineer Manuals (EMs) and Engineer Regulations (ERs) can be obtained from the U.S. Army Corps of Engineers Official Publications website. The link to the website is provided in Appendix B (Entry 29, p. B-4).

b. Engineer Manuals. This EM relies on, and references, technical information presented in other guidance documents. Some of the key EMs for reservoir studies are listed below. Additionally, there are related documents on hydraulic design for project features (e.g., spillways and outlet works) associated with reservoir projects. This document does not present hydraulic design concepts.

- EM 1110-2-1100, Coastal Engineering Manual
- EM 1110-2-1201, Reservoir Water Quality Analysis
- EM 1110-2-1406, Runoff from Snowmelt
- EM 1110-2-1411, Standard Project Flood Determinations
- EM 1110-2-1415, Hydrologic Frequency Analysis
- EM 1110-2-1416, River Hydraulics
- EM 1110-2-1417, Flood-Runoff Analysis

- EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works
- EM 1110-2-1603, Hydraulic Design of Spillways
- EM 1110-2-1612, Ice Engineering
- EM 1110-2-1701, Hydropower
- EM 1110-2-3600, Management of Water Control Systems
- EM 1110-2-4000, Sedimentation Investigation of Rivers and Reservoirs

(1) These manuals provide the technical background for study procedures that are frequently required for reservoir analysis. Specific references to these EMs are made throughout this document.

c. Engineer Regulations.

(1) Several ERs prescribe necessary studies associated with reservoir projects. The most relevant ERs are listed below.

- ER 405-1-12, Real Estate Handbook
- ER 1105-2-100, Planning Guidance Notebook
- ER 1110-2-240, Water Control Management
- ER 1110-2-1156, Safety of Dams – Policy and Procedures
- ER 1110-2-1451, Acquisition of Lands Downstream from Spillways for Hydrologic Safety Purposes
- ER 1110-2-1460, Hydrologic Engineering Management
- ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs

(2) These and other regulations should be consulted prior to performing any hydrologic engineering study for reservoirs. The most recent regulations are published on the USACE Publications website.

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PART 1:

HYDROLOGIC ENGINEERING CONCEPTS

FOR RESERVOIRS

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CHAPTER 2

Reservoir Purposes

2-1. Congressional Authorizations.

a. Authorization of Purposes. The United States Congress authorizes the purposes served by U.S. Army Corps of Engineers (USACE) reservoirs at the time the authorizing legislation is passed and signed into law by the President. Congress commonly authorizes a project “substantially according to the recommendations of the Chief of Engineers,” as detailed in a separate document referenced in the authorizing legislation. Additional purposes are sometimes added, deleted, or original purposes modified later, by subsequent congressional action.

b. The USACE constructs and operates reservoir projects according to the congressionally-authorized purposes. There are also non-USACE owned projects in which USACE is responsible for developing water control plans and manuals. These projects include those in which storage is operated and managed for flood control and navigation and subject to USACE direction per Section 7 of the Flood Control Act of 1944 or other law. While this manual does not determine or define the authorized purposes or legal operating requirements of any USACE owned or non-USACE owned reservoir project, a summary of generally applicable legislation and of common purposes for which these reservoirs are planned, designed, authorized, and operated follows below. Congress has enacted general legislation that may apply to many projects.

c. Authorizations have increased the number of purposes that the USACE considers when planning and managing water resource development projects. The first authorizations were principally for improved navigation, the control and construction of hydroelectric dams, and flood risk management. Later authorizations covered a variety of conservation purposes and programs. During drought when there is a water shortage, multiple purposes (with the possible exception of flood risk management) compete for available water and are affected by the shortage. The more purposes and programs there are to serve, the greater the potential for conflict, and the more complex the task of managing existing supplies.

2-2. Reservoir Purposes.

a. A reservoir, as defined by ER 1110-2-1156, Engineering and Design Safety of Dams, is a body of water impounded by a dam in which water can be stored. Congressionally authorized purposes of reservoirs in the USACE include flood risk management, navigation, hydropower, municipal and industrial (M&I) water supply, water quality, irrigation, fish and wildlife, low flow augmentation, and recreation. The following sections provide an overview of reservoir storage and how it is used for authorized purposes.

b. Storage capacity. A cross section of a typical reservoir is shown in Figure 2-1. The storage capacity is divided into pools and/or zones. In some cases these are: flood control, conservation, and inactive. The nomenclature of these pool zones vary from District to District within the USACE and have changed over time. The nomenclature shown in Figure 2-1 is used here for illustration. For example, the flood control zone is also called the surcharge zone and the conservation zone is also called the normal or multipurpose pool. While each USACE reservoir is unique both in its allocation of storage space and in its operation, the division of storage illustrated by Figure 2-1 is relatively typical.

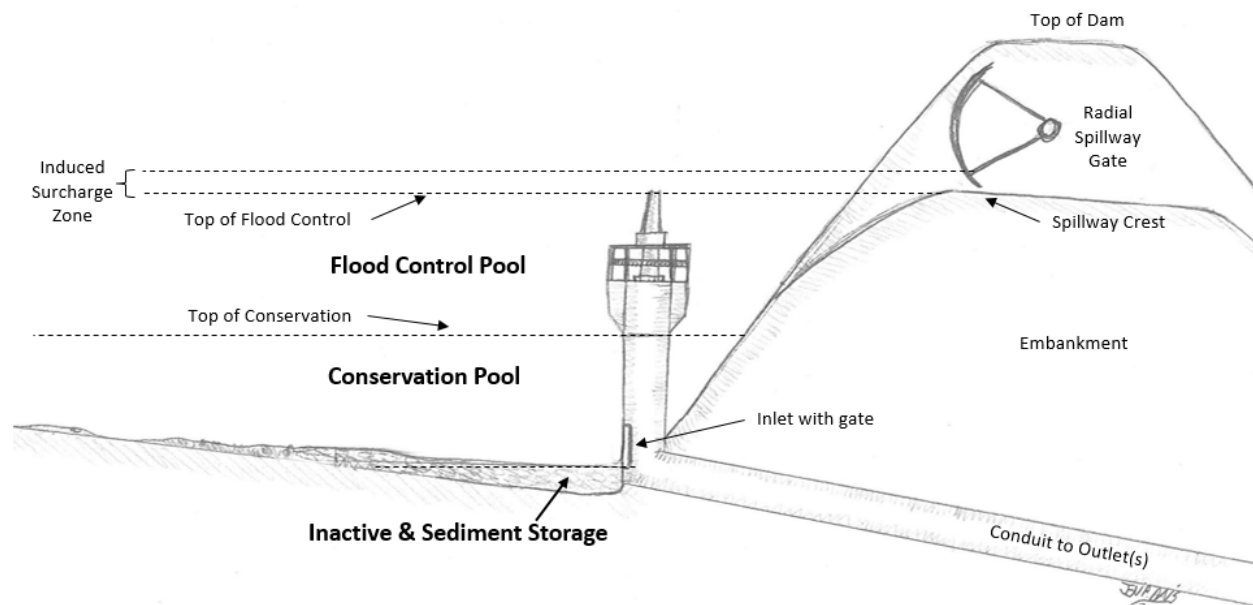


Figure 2-1. Typical Storage Allocation in Reservoirs.

c. Induced surcharge zone. Induced surcharge is water stored above the top of the flood pool or above the spillway crest that allows maximum use of the available flood storage space in the reservoir and allows evacuation of flood waters in a planned manner as the reservoir rises.

d. Flood Control. The flood control pool is typically reserved for the control of large in-flow events and is managed with the goal of being empty prior to flood season.

e. Conservation Pool. Conservation storage, sometimes referred to as normal or multipurpose storage, serves a variety of purposes, including but not limited to navigation, hydroelectric power, water supply, irrigation, fish and wildlife, recreation, and water quality. Multipurpose storage may also include seasonal flood risk management storage, in addition to the flood control storage pool or zone.

f. Inactive Capacity. Inactive storage is used to designate a volume for sediment accumulation within some reservoirs. Sediment accumulation, however, is usually not isolated to only

the inactive storage zone. Sediment deposition may occur at different rates within the various pool zones of the reservoir. Sediment deposition within the flood storage zone can impact reservoir operations. Typical sediment processes that affect reservoir storage include headwater delta formation, shoreline erosion, and tributary inflow locations. See Chapter 9 of this manual for additional information on reservoir sediment deposition. Inactive storage is dead storage and cannot be relied upon for emergency water supply. Further uses of inactive storage can be addressed in a drought contingency plan.

g. Buffer Storage. Buffer storage or buffer zones are regions within the conservation pool where operational criteria guiding releases from the reservoir change. These are discussed in greater detail later in this document.

h. The boundaries between the storage zones and operational boundaries within the zones may be fixed throughout the year, or they may vary from season to season as shown in Figure 2-2. The varying boundaries usually offer a more flexible operational plan, which may result in higher yields for all purposes, although an additional element of chance is often introduced when the boundaries are allowed to vary. A discussion of reservoir operating procedures is found in EM 1110-2-3600, Management of Water Control Systems.

i. Reservoir storage space may be allocated to specific purposes or its storage requirements may change based on seasonal needs. Examples of seasonal needs include flood storage availability at the beginning of the runoff year and storage for additional water for other purposes during the summer. In addition, reservoir releases can serve several purposes such as meeting environmental needs in one reach and irrigation needs in a reach farther downstream. However, the amount of water needed to serve each purpose varies. During drought, with limited multiple-purpose storage available, the purposes requiring greater releases begin to compete with purposes requiring less. In this case, if the greater releases are not made, the storage would last longer for the purposes served by the lesser releases.

2-3. Flood Risk Management.

a. Operation of Storage Space. Flood control reservoirs are designed to minimize flooding downstream by capturing upstream runoff and releasing it from storage at a controlled rate. Each USACE reservoir has a water control plan that defines its purposes and management goals. Reservoirs designed for seasonal flood risk management often vary their storage space allocations seasonally. In some cases, pool elevations are drawn down through the fall and winter to provide more storage in early spring for the runoff season and are then allowed to increase to provide water for other purposes in the late summer. In special circumstances (e.g., snowmelt runoff), forecasting tools and expertise are available to adjust the timing of releases with the prediction of future inflow volumes and the peak timing. The timing of seasonal drawdowns for flood risk management differs based on the reservoir location.

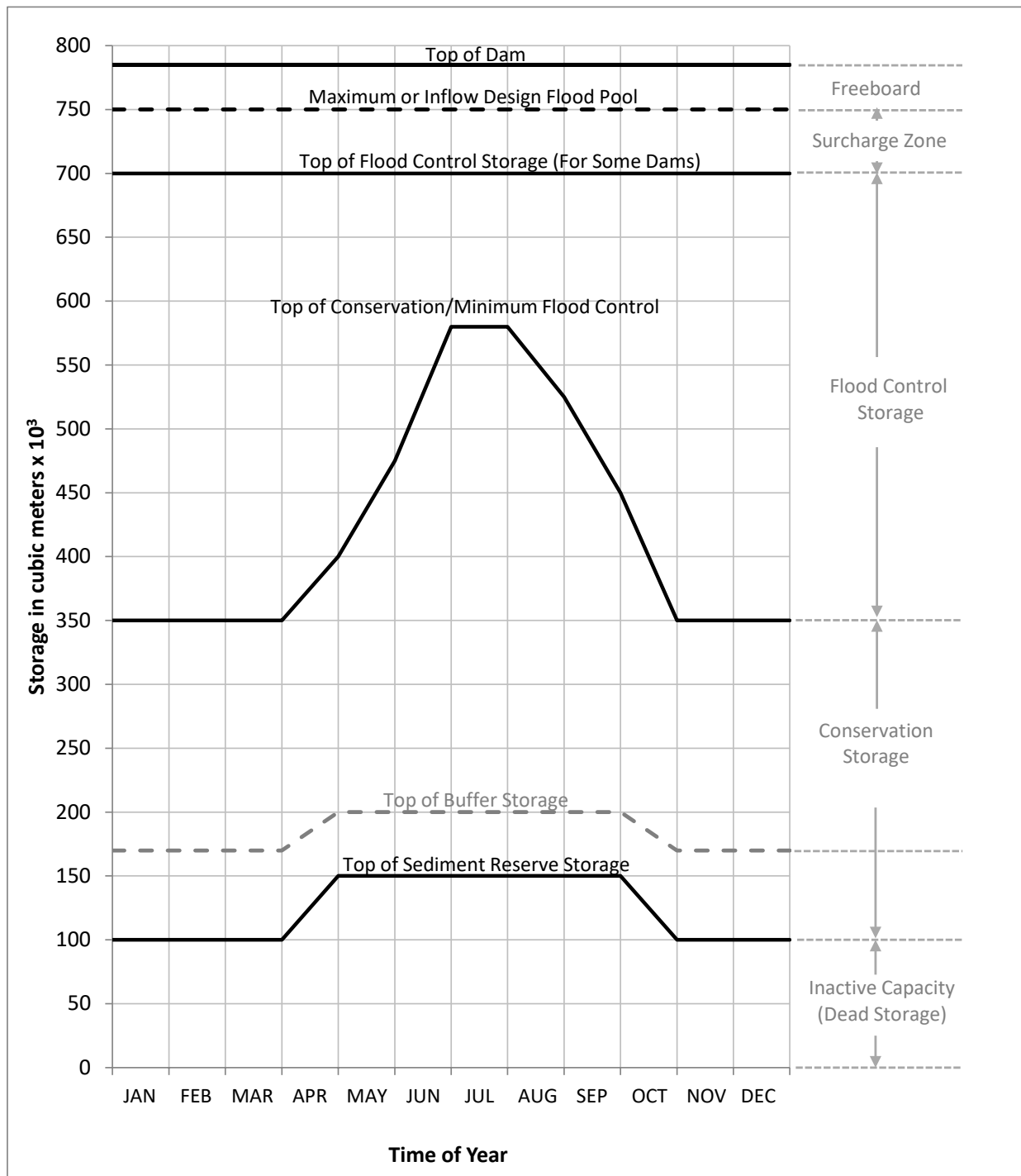


Figure 2-2. Seasonally Varying Storage Boundaries.

b. Releases. Flood control releases are guided by predetermined flow limits at downstream locations (control points) to help prevent damages. Downstream control points are typically stream gauges located in communities along the river. Water management must consider the travel times caused by storage effects in the river system and the timing and magnitude of local inflows from tributaries between the reservoir and downstream control points to avoid releases that adversely impact flooding.

c. Intervening Tributary and Downstream Damage Areas. A multiple reservoir system is generally managed to provide flood risk management at both intervening tributary areas and at downstream main stem damage areas. The extent of reservoir management needed to mitigate risk in these areas depends on local flood conditions, uncontrolled tributary drainage, reservoir storage capacity, and the volume and time distribution of reservoir inflows. Upstream or downstream impacts may influence reservoir management. Typically, optimum management is based on a combination of the two.

d. Coordinated Reservoir Management. A series of reservoirs is operated as a coordinated system to achieve their authorized purposes. This can entail a balanced reservoir management approach, in which some reservoir pools are drawn down and others increased, while also considering expected inflows and downstream channel capacities. The evacuation of flood water stored in a reservoir system is guided by conditions at control points along the river and available flood storage within each reservoir. Releases of water from the various zones of reservoir storage are determined to minimize the risk of encroaching into the flood control storage and to meet other project requirements. Sometimes the lower portion of the flood control pool must be evacuated at a slower rate to minimize bank caving and allow channel recovery.

2-4. Navigation.

a. Navigational Requirements. Navigational requirements are integrated with other water uses in multiple-purpose water resource systems to help provide sufficient water for channel navigation during dry periods. Navigational requirements involve controlling water levels in the reservoirs and at downstream locations to help provide the quantity of water necessary for navigation and the operation of locks. Constraints in the rates of change of water surface elevations and outflows may also exist. There are numerous special navigational considerations that may involve water control, such as ice, undesirable currents and water flow patterns, emergency precautions, boating events, and launchings.

b. Flow Requirements. Navigation locks located at dams on major rivers generally have sufficient in-stream flows to supply the required water flow for lockage. Navigation requirements for downstream use in open-river channels may require larger quantities of water over a long period of time (from several months to a year) to maintain water levels for boat or barge navigation. Usually, water released from reservoirs for navigation is also used for other purposes, such as hydroelectric power, low-flow augmentation, water quality, enhancement of fish

and wildlife, and recreation. Seasonal or annual water management plans are prepared to define the use of water for navigation. The amount of stored water to be released depends on the conditions of water storage in the reservoir system and downstream requirements or goals for low-flow augmentation, as well as factors related to all uses of the water in storage.

c. **Water for Lockage.** In some rivers, the supply of water for lockage is a significant problem, particularly during periods of low flow or droughts. In critical low-water periods, a curtailment of water use for lockage may be instituted by restricting the number of locks used, thereby conserving the use of water through a more efficient use of the navigation system. Water requirements for navigation canals are sometimes based on lockage and in-stream flows as necessary to preserve water quality in the canal.

2-5. Hydroelectric Power.

a. **Reservoir Project Categories.** Reservoir projects with hydropower generally fall into two distinct categories: storage reservoirs with sufficient capacity to regulate flow on a seasonal, daily or hourly basis, and run-of-river projects in which storage capacity is minor relative to the volume of flow. Most storage projects are multiple-purpose. Normally, the upstream reservoirs include provisions for power production at the site, as well as for release of water for downstream control. Run-of-river hydropower plants are usually developed in connection with navigation projects.

b. **Integration and Control of a Power System.** The management of hydropower systems requires coordination between both power authorities and USACE Divisions and Districts. Integration and management of a major power system involving hydropower is generally accomplished by a centralized power dispatching or real-time scheduling facility. The dispatching center is continuously staffed, and operators monitor and manage the flow of power through the system, rectify outages, and perform all the necessary steps to assure power system operation meets system loads. Operating project personnel are often the points-of-contact for dispatchers and real-time power schedulers, but USACE water managers may also need to be in contact with dispatchers regarding regulation schedules, transmission and generation constraints, emergencies, or to provide project updates. This includes dispatchers for agencies transmitting power from USACE-owned dams, as well as dispatchers for dams operated by other entities in which the USACE directs flood risk management operations or has reservoir regulation responsibility. See EM 1110-2-3600, Management of Water Control Systems, for detail on hydropower system management.

2-6. Irrigation.

a. **Irrigation Diversion.** Irrigation water diverted from reservoirs, diversion dams, or natural river channels is controlled to help meet the relevant water demands of irrigation users. The requirements vary seasonally, and in most irrigated areas in the western United States the agri-

cultural growing season begins in the spring months. The diversion requirements gradually increase as the summer progresses, reaching their maximum amounts in July or August. They then recede to relatively low amounts by late summer. By the end of the growing season, irrigation diversions are terminated, except for minor amounts of water that may be necessary for domestic use, stock water, or other purposes.

b. Irrigation as a Project Purpose. Some USACE reservoir projects include storage for irrigation, which could be in conjunction with U.S. Department of the Interior, Bureau of Reclamation authorized diversion projects. Corps of Engineers reservoir projects have been authorized and operated primarily for flood risk management, navigation, and hydroelectric power. However, several major USACE multipurpose reservoir projects include irrigation as a project purpose. Where authorized, water for irrigation may be supplied from reservoir storage to meet irrigation demands in downstream areas. In some cases, water is diverted from the reservoir by gravity through outlet facilities at the dam that feed directly into irrigation canals. In other cases, water is pumped directly from the reservoir for irrigation purposes.

2-7. Municipal and Industrial Water Supply.

a. Municipal and Industrial Water Supply Storage. Municipal and industrial (M&I) water supply may be an originally authorized purpose of a USACE reservoir project, or may be added by the inclusion of storage per the Water Supply Act of 1958 or other project-specific authority. Reservoir management for M&I water supply is performed in line with congressionally authorized purposes and/or contractual arrangements, consistent with applicable project authorities. Per agreement, the water user typically has the right, subject to any relevant conditions, of withdrawing water from the lake or to request releases through the outlet works, subject to the availability of water.

b. Temporary Withdrawal. In times of drought, special considerations may guide the management of projects with regard to water supply. Drought and other emergencies affecting M&I water supplies may generate requests for water available at USACE reservoirs. When these situations occur, requests may require immediate action. See ER 1105-2-100, Planning Guidance Notebook for policies that may be applicable in such circumstances.

2-8. Water Quality.

a. Goal and Objective. Water quality includes the physical, chemical, and biological characteristics of water and abiotic and biotic interrelationships. Water quality is an integral consideration during all phases of a project's life, from planning through operation. The minimum goal is to meet current applicable State and Federal water quality standards for lakes and tail waters. The operating objective is to maximize beneficial uses through enhancement and non-degradation of water quality.

b. **Release Requirements.** Water quality releases for downstream targets may have both qualitative and quantitative requirements. With respect to water quality, projects are operated according to all USACE policy, guidance, and in compliance with applicable legislation and any executive orders. An important measure of water quality is the quantity of available water, as many USACE reservoir projects may have minimum flow requirements at some downstream control point for water quality. Other common objectives include water quality monitoring in reservoirs for characteristics such as temperature, dissolved oxygen, and turbidity targets at downstream locations.

c. **Coordinated Management.** Coordinated management of multiple reservoirs in a river basin is required to maximize benefits beyond what might be achievable with individual project management. System management for quantitative aspects, such as flood risk management and hydropower generation, is a widely accepted and established practice, and the same principle applies to water quality concerns. Water quality maintenance and enhancements may be possible through coordinated system management. This applies to all facets of quality including water quantity, water temperature, and dissolved oxygen content.

d. **System Management.** System management for water quality often is of most value during low-flow periods when available water can be used with the greatest efficiency to avoid degrading lake or river quality. Seasonal water control plans are formulated based on current and forecasted basin hydrologic, meteorologic, and quality conditions; reservoir status; quality objectives; and the knowledge of water quality characteristics within the system. Required flows and qualities are then apportioned to the individual projects, resulting in a quantitatively and qualitatively balanced system.

2-9. **Fish and Wildlife.** Project management can influence fisheries both in the reservoir pool and downstream. The USACE Environmental Operating Principles described in ER 1110-2-240 are integrated into all water control management activities with respect to authorized or approved purposes.

2-10. **Recreation.**

a. **Reservoir Level.** The recreational use of reservoirs may extend throughout the entire year. The optimum pool elevation for recreational use under most circumstances is a full or near full reservoir conservation pool during the recreation season. The degree to which this objective can be met varies widely depending upon the regional characteristics of water supply, runoff, and the basic objectives of water management for the various project purposes, consistent with authorizing legislation. Facilities constructed to enhance the recreational use of reservoirs may be designed to be operable under the planned reservoir guide curves, which reflect the ranges of reservoir levels that are to be expected during the recreation season.

b. **Downstream River Levels.** The management of project outflows, to the extent consistent with authorizing legislation and the approved water control manual, may include requirements for criteria to enhance the use of the rivers downstream of the projects for recreation and

to protect the safety of the general public, which could include streamflow augmentation during low flow periods.

2-11. Water Management Goals and Objectives.

a. Water Management. ER 1110-2-240, Water Control Management, defines the goals and objectives for water management by the USACE. Basically, the objective is to conform with the provisions of the project authorizing legislation and water management criteria defined in Corps of Engineers reports prepared in the planning and design of the project or system. Beyond this, the goals for water management will include applicable statutory and regulatory provisions relating to operations of USACE facilities.

b. Water Control Systems Management. EM 1110-2-3600, Management of Water Control Systems, provides guidance on water control plans and project management. A prime requirement in project management is the safety of users of the facilities and the general public, both at the project and at downstream locations. The development of water control plans and the scheduling of reservoir releases must be coordinated with appropriate agencies, or entities, as necessary to meet commitments made during the planning and design of the project. Additionally, water control plans should be reviewed and adjusted, as appropriate, to meet changing local conditions, as provided in ER 1110-2-240, Water Control Management.

c. Regional Management. Regional water management should consider the interaction of surface-groundwater resources. HEC Research Document 32, Importance of Surface-Groundwater Interaction to Corps Total Water Management: Regional and National Examples, provides examples for several regions in the United States (HEC 1991b).

d. Reservoir Reallocations. Existing USACE projects may reallocate water storage from one reservoir purpose to another reservoir purpose. Specific policies and procedures applicable to reallocations of storage are discussed in ER 1105-2-100, Planning Guidance Notebook.

2-12. Special Reservoir Operations and Adaptive Management.

a. Special Operations. Reservoir operations are guided by the needs of the congressionally authorized purposes. Many of these operations are consistent from year to year, and have been so historically. However, other operations may be dependent on annual conditions such as reservoir storage, forecasted inflows, and river stages and change from year to year. These operations are sometimes called special operations.

b. Adaptive Management. Special operations are added or changed over time as new information becomes available. Adaptive management is the ability to adjust management or operation of a system based on best available science and acquired knowledge about current conditions and the effects of management actions, as opposed to following a rigid set of rules. Adaptive management, where appropriate and consistent with authorized purposes and applicable law,

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may provide greater flexibility in system operations to account for the variability and unpredictability of future conditions.

CHAPTER 3

Multipurpose Reservoirs

3-1. Hydrologic Studies for Multipurpose Projects.

a. Multipurpose reservoirs were originally conceived as projects that served more than one purpose and effect savings through the construction of a single large project instead of two or more smaller projects. Even uses that might appear to have competing objectives—such as flood risk management, which focuses on releasing water to provide storage for future inflows, and water supply, which stores water for future use during droughts—could use the same reservoir space at different times during the year in a multipurpose project. In all cases, reservoir operations and uses of storage are dependent upon the relevant project authorization and other applicable legislation.

b. The feasibility of a multiple purpose reservoir is dependent upon its ability to serve several purposes simultaneously. To demonstrate that multipurpose operation is feasible, detailed analyses of the effects of various combinations of stream flows, storage levels, and water requirements are required. Detailed analyses of these factors may be deferred during the planning phase depending upon the complexity of the analysis. However, simplifying multipurpose operation in the planning phase is risky because the operation criteria are critical in determining the feasibility of serving several purposes simultaneously.

c. The determination of a reservoir project's storage-yield relationship through detailed sequential analysis is difficult during the planning phase because data on water demands are either not available or not of comparable quality for all purposes. To adequately define the multipurpose operation, the analyses must include information on the magnitude and seasonal variations of each demand, long-term changes in demands, and shortage tolerances. See Chapter 12 of this EM for guidance on developing storage-yield relationships.

d. Successful multipurpose operation also depends on the formulation of operational rules that ensure that water is available in the proper quantities and qualities for each of the purposes at the proper time and place. Techniques for formulating operational rules are not fixed, but the logical approach involves determining the seasonal variation of the flood control space requirement, the seasonal variation of conservation requirements, the formulation of general operational rules that satisfy these requirements, and detailed testing of the operational rules to ascertain the adequacy of the plan for each specific purpose.

e. The judgment of an experienced hydrologic engineer is invaluable in the initial formulation and subsequent development and testing of operational rules. Although the necessary rules cannot be completely developed until most of the physical dimensions of the reservoir are known, any tendency to discount the importance of operational rules as a planning variable should be resisted due to their importance in determining the feasibility of multipurpose projects.

As a minimum, the operational rules used in a planning study should be sufficiently refined to assist the engineer in evaluating the suitability of project alternatives to satisfy water demands for specified purposes.

3-2. Balance of Multiple Objectives.

a. For multipurpose reservoirs, the projects are designed and operated to balance multiple objectives. In general, the goal of water control management is to conform a project's operation to its authorizing legislation, to criteria defined in USACE reports prepared in the planning and design of a particular project or system, and to applicable congressional acts relating to the purpose of federal facilities or systems.

b. A water control plan will need to strike a balance among the use of water storage for all such purposes including, for example: flood risk management, municipal and industrial water supply, navigation, hydroelectric power, water quality, fish and wildlife protection, ecosystem management, and recreation. For multipurpose projects, operational priorities among these purposes under particular conditions, such as drought or high water, may need to be defined.

c. The basic objectives of water control management can be summarized as: (1) operate according to authorized purposes and applicable law, (2) maintain the structural and operational integrity of the project, and (3) avoid risk to public health and safety, life, and property.

3-3. Managing Competitive and Complementary Functions.

a. Identifying Interactions between Purposes. Adverse (competitive) and beneficial (complementary) interactions between reservoir purposes must be identified before operation rules can be formulated. The time of occurrence of the interactions is often as important as the degree of interaction, particularly if one or more of the water uses has significant variations in water demand. For example, it is possible for the complementary purposes of a single multipurpose reservoir to become competitive at times due to differences in their seasonal water requirements.

b. Allocating Storage Space. Storage space within a reservoir can be allocated to specific purposes. This practice evolved from projects that served only flood risk management and one conservation purpose because it was necessary to reserve a portion of the reservoir storage for floodwater storage. It is still necessary to have storage allocated for flood control storage (although the storage can vary seasonally) due to the conflict between reserving empty storage space for regulating potential floods and filling that space to meet conservation storage requirements. Setting specific storage allocations for different conservation purposes should be kept to a minimum, however, because it may reduce the operational flexibility of the reservoir.

c. Operational Conflicts. The allocation of specific storage space to several purposes within the conservation pool can result in operational conflicts that may make it more difficult to provide water for the various purposes in the quantities and at the time they are needed. Using

commingled or joint-use conservation storage for all conservation purposes with operational criteria to maximize the complementary effects and minimize the competitive effects is far easier to manage and, if carefully designed, will provide better service for all purposes. When joint-use storage is used, the operational criteria should be studied in the planning process with an emphasis on making sure that all of the various authorized purposes are accomplished. This allows careful evaluation of a number of operational plans.

3-4. Operating Concepts. Reservoir operations vary with the available storage in the reservoir. The highest zones, in a reservoir with flood risk mitigation as a purpose, are the flood control pool and surcharge zone. These pool zones are reserved for the management of floods and passage of spillway flows. Whenever water is in the flood control zone it must be released consistent with flood control requirements. Water in the conservation pool of a multipurpose reservoir serves a variety of purposes and releases are affected by the available storage. The top zone of conservation storage may include storage that is not required to satisfy the firm conservation demands. Water in this space can be released to serve needs or uses that exceed the basic requirements of authorized purposes. The middle zone of conservation storage is that needed to store water to supply firm water needs. The bottom zone of the conservation storage space may contain a buffer zone. When operation is in the buffer zone, services to authorized purposes may be curtailed to prevent a more severe shortage later.

a. Storage Boundaries. The boundaries between storage zones may be fixed at a constant level or they may vary seasonally. In general, the seasonally varying boundaries offer the potential for a more flexible operating plan that can result in higher yields for all purposes. However, the proper location of the seasonal boundaries requires more study than the location of a constant boundary. This is discussed in more detail in Chapter 11. Furthermore, an additional element of chance is introduced when the boundaries are allowed to vary, because the joint use of storage might endanger firm supplies for one or more specific purposes. The location of the seasonally varying boundaries is determined by a process of formulating a set of boundaries and attendant operational rules, testing the scheme by a detailed sequential routing study, evaluating the outcome of the study, changing the rules or boundaries if necessary, and repeating the procedure until a satisfactory operation results. Sequential routing is used to determine storage-yield relationships and is discussed in Chapter 12.

b. Demand Schedules. Expressing demand schedules as a function of the relative availability of water is another means of incorporating flexibility in operational rules. For example, it might be possible to have two or more levels of navigation service or lengths of navigation season with the actual level of service or length of season being dependent upon the availability of water in the reservoir. By regulating the level of supply to the available water in the reservoir, users can plan emergency measures that will enable them to withstand partial reductions in service and thereby avoid complete cessation of service, which might be disastrous. Terms such as “desired flow” and “minimum required flow for navigation” can be used to describe two levels of service.

c. Levels of Service. There can be as many levels of service as a user desires, but each level requires criteria for determining when the level is to be initiated and when it is to be terminated. The testing and development of the criteria for operating a multipurpose project with several purposes and several levels of service are accomplished by detailed sequential routing studies.

d. Buffer Storage. Buffer storage or buffer zones are regions within the conservation storage where operational rules create a temporary reduction in firm services. The two primary reasons for temporarily reducing services are to ensure service for a high priority purpose while eliminating or curtailing services for lower-priority purposes, and to change from one level of service for a given purpose to a lower level of service for that same purpose when storage levels are too low to ensure the continuation of firm supplies for all purposes. As with the other techniques for implementing multipurpose operation, the amount of buffer storage and the location of the boundaries cannot be determined accurately except by successive approximations and testing by sequential routing studies.

3-5. Construction and Physical Operation. In addition to hydrologic determinations discussed above, a number of important hydrologic determinations are required during project construction and during project operation for ensuring the integrity of the project and its operation.

a. Cofferdams. If a cofferdam used for dewatering a construction work area is overtopped, serious delays and additional construction costs can result. In the case of high cofferdams where substantial poundage occurs, it is possible that failure could cause major damage in downstream areas. Cofferdams should be designed on the same principles as are permanent dams, generally on the basis of balancing incremental costs against incremental benefits of all types. This will require flood-frequency and hypothetical flood studies, as described in Chapters 6 and 7 of this manual.

b. Overtopping. If a major dam embankment may be overtopped during construction, the diversion conduit capacity must be sufficient to regulate floods that might occur with substantial probability during the critical construction period. It is not necessary that the regulated releases be nondamaging downstream, but it is vital that the structure remain intact. An explicit evaluation of risk of embankment failure and downstream impacts during construction should be presented in the design document. See ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs, for more information.

c. Conduits, Spillways, and Gates. Conduits, spillways, and all regulating gates must be functionally adequate to accomplish project objectives. Their sizes, dependability, and speed of operation should be tested using recorded and hypothetical hydrographs and anticipated hydraulic heads to ensure that they will perform properly. The nature of stilling facilities might be dictated by hydrologic considerations if frequency and duration of high outflows substantially influence their design. Detailed reservoir stratification studies under combinations of hydrologic and

reservoir conditions can be used to assess the necessity for multilevel intakes to control the quality of reservoir releases. Techniques for conducting reservoir stratification studies are discussed in EM 1110-2-1201, Reservoir Water Quality Analysis.

d. **Hydropower Design.** The design of power facilities can be greatly influenced by hydrologic considerations, as discussed in Chapter 11 of this manual and EM 1110-2-1701, Hydropower. General considerations in the hydrologic design of spillways are discussed in Chapter 10 of this EM and more detailed information is presented in Chapter 14.

e. **Extreme Floods.** Regardless of the reservoir purposes, it is imperative that spillway facilities will ensure the integrity of the project in the event of extreme floods. Whenever the operation rules of a reservoir are substantially changed, spillway facilities should be reviewed to ensure that the change in project operation does not adversely alter the spillway's capability to pass extreme floods without endangering the structure. The capability of a spillway to pass extreme floods can be adversely affected by changes in operation rules that affect the flood operation itself or by changes that result in higher pool stages during periods of high flood potential. Extreme floods are addressed in more detail in ER 1110-8-2, Inflow Design for Dams and Reservoirs, and ER 1110-2-1156, Safety of Dams – Policy and Procedures.

f. **Special Operating Rules.** A number of situations might require special operating rules. For example, operating rules are needed for the period during which a reservoir is initially filling, for emergency dewatering of a reservoir, for interim operation of one or more components in a system during the period while other components are under construction, and for unanticipated conditions that seem to require deviation from established operating rules. The need for operation rules during the filling period is especially important. Many dams have special geotechnical monitoring requirements during first filling, based on a history of dam failures during their initial filling, and many decisions must be based on the filling plan. Among the important factors that are dependent upon the filling schedule are the online date for power-generating units, the in-service dates for various purposes such as water supply and navigation, and the effective date for legal obligations such as recreation concessions.

g. **Monitoring Facilities.** The installation of monitoring equipment at a reservoir is important for the hydrologic analysis and monitoring of the system. Data collected can include stream flow, rainfall, reservoir stage, and other hydrologic data. Monitoring equipment serves two basic purposes: it records all operations and provides information for operation decisions. The former purpose, monitoring, satisfies legal requirements and provides data for future studies. The latter purpose may greatly increase the project effectiveness by enabling the operating agency, through reliable forecasts of hydrologic conditions, to increase operation efficiency.

h. **Stream Gauges.** Special care should be taken in choosing the location of a gauge as stream gauges are important during flood events and should be accessible and high enough that they are unlikely to be washed out during high flows. Stream gauges should not be located on

bridges or other structures that are subject to being washed out. To the extent possible, the gauges should be capable of working up through extreme flood events, and stage-discharge relationships should be developed up to that level. The gauge should have reasonable access for quality checks and repair during flood events. Reservoir spilling, local flooding, and backwater effects from downstream tributaries should all be considered when finding a suitable location. More detailed information on stream gauges can be found in the U.S. Geological Survey (USGS) publications on their Techniques of Water-Resources Investigations Reports website (see Appendix B [Entry 35, p B-4] for website information). Refer to Chapter 5 in this EM for sources of streamflow data.

3-6. General Study Procedure. As indicated earlier, there is no fixed procedure for developing reservoir operational plans for multipurpose projects; however, the general approach that should be common to all cases would include the following steps:

- a. Survey the potential water uses to be served by the project to determine the magnitude of each demand and the seasonal and long-term variations in the demand schedule.
- b. Develop a relative priority for each purpose and determine the levels of service and required priority that will be necessary to serve each purpose. If necessary, make sequential studies illustrating the consequences of various alternative priority systems.
- c. Establish the seasonal variation of flood control space required, using procedures discussed in Chapter 10.
- d. Establish the total power, water supply, and low-flow management requirements for competitive purposes during each season of the year.
- e. Establish preliminary feasibility of the project based on physical constraints.
- f. Establish the seasonal variation of the storage requirement to satisfy these needs, using procedures described in Chapter 11.
- g. Determine the amount of storage needed as a minimum pool for power head, recreation, sedimentation reserve, and other purposes.
- h. Using the above information, estimate the size of reservoir and seasonal distribution of space for the various purposes that would satisfy the needs. Determine the reservoir characteristics, including flowage, spillway, power plant, and outlet requirements.
- i. Test and evaluate the operation of the project through the use of recorded hydrologic data in a sequential routing study to determine the adequacy of the storage estimates and proposed rules with respect to the operational objectives for each purpose. If the record is short,

supplement it with synthetic floods to evaluate flood storage reserves. If necessary, make necessary changes and repeat testing, evaluating, and changing until satisfactory operation is obtained.

j. Test proposed rules of operation by using sequential routing studies with stochastic hydrologic data to evaluate the possibility of historical bias in the proposed rules.

k. Determine the needs for operating and monitoring equipment required to ensure proper functional operation of the project.

l. As detailed construction plans progress, evaluate cofferdam needs and protective measures needed for the integrity of project construction, particularly diversion capacity as a function of dam construction stage and flood threat for each season.

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CHAPTER 4

Reservoir Systems

4-1. Introduction. Water resource systems should be designed and operated for the most effective and efficient accomplishment of overall objectives. A system may consist of reservoirs, power plants, diversion structures, channels, and conveyance facilities that are each constructed for specific objectives and operated together as a system. There may be considerable latitude in developing an operational plan for any water resource system, but system operations must be consistent with congressional authorizations and all applicable law.

a. Mathematical Modeling. Water resource system operation is usually modeled mathematically with software packages, rather than with physical models. The mathematical representation of a water resource system can be extremely complex. The Hydrologic Engineering Center Reservoir System Simulation (HEC-ResSim) software package simulates reservoir operations for flood management, low-flow augmentation and water supply for planning studies, detailed reservoir regulation plan investigations, and real-time decision support. HEC-ResSim can represent both large and small scale reservoirs and reservoir systems through a network of elements (junctions, routing reaches, diversions, reservoirs) that the user builds. The software can simulate single events or a full period of record using available time steps. HEC-ResSim is a decision support tool that meets the needs of modelers performing reservoir project studies as well as meeting the needs of reservoir regulators during real-time events. In addition, the Corps Water Management System (CWMS) is able to link a suite of models' results and inputs so the models can be run simultaneously. This suite of models may include HEC-HMS, HEC-ResSim, the Hydrologic Engineering Center River Analysis System (HEC-RAS), and the Hydrologic Engineering Center Flood Impact Analysis (HEC-FIA) computer program.

b. Inputs and Requirements. A factor that greatly complicates the simulation and evaluation of reservoir system outputs is the stochastic nature of both the inputs (stream inflows) and of the requirements (demands) on the system. Historically, it has been customary to evaluate reservoir system operation on the assumption that a repetition of historical streamflow and requirements (adjusted to future conditions) would adequately represent future values. However, this assumption has been demonstrated to be deficient. It is desirable to test any proposed system operation under a great many sequences of inputs and requirements. This requires a mathematical model that will define the frequency and correlation characteristics of inputs and requirements and that is capable of generating a number of long sequences of these quantities. In addition, the possible effects of climate change must now be addressed based on USACE policy. Concepts for accomplishing stochastic streamflow modeling are discussed in Chapter 5. Emerging guidance for climate change analysis is presented in Engineering and Construction Bulletins, and will be transferred to permanent guidance.

4-2. System Description.

a. System Operation Simulation. Water resource systems may consist of reservoirs, power plants, diversion structures, channels, and conveyance facilities. To simulate system operation, the system must be completely described in terms of the location and functional characteristics of each facility. The system should include all components that affect the project operation and provide the required outputs for analysis.

(1) Reservoirs. Reservoir storage and behavior is described through elevation-storage-discharge or elevation-area-discharge relationships. Characteristics of the control gates on the outlets and spillway must be known to determine constraints on operation. The top-of-dam elevation must be specified and the ability of the structure to withstand overtopping must be assessed.

(2) Downstream Channels. The physical characteristics of downstream channels must be defined. Example characteristics include estimates of Mannings roughness, length, and channel cross sections. Maximum and minimum-flow targets are required. For short-interval simulation, the translation of flow through the channel system is modeled by routing criteria. The travel time for flood flow is important in determining reservoir releases and potential limits for flood control operation to distant downstream locations.

(3) Power Plants. The relationship between turbine and generation capacity to head must be determined for power plants at storage reservoirs. To compute the head on the plant, the relation of tail water elevation to outflow must be known. Also, the relation of overall power plant efficiency to head is required. Other characteristics such as turbine leakage and operating efficiency under partial load are also important.

(4) Diversion Structures. Maximum diversion and delivery capacity must be established for diversion structures. The demand schedule is required, and the consumptive use and potential return flow to the system may be important for the simulation.

b. Preparing Data. While reservoir system data must be defined in sufficient detail to simulate the essence of the physical system, preparing the required hydrologic data may require far more time and effort. The essential flow data are required for the period of record, for major flood events, and in a consistent physical state of the system. However, flow records are usually incomplete, new reservoirs in the system change the flow distribution, and water usage in the watershed alters the basin yield over time. Developing a consistent hydrologic data series, making maximum use of the available information, is discussed in Chapter 5.

4-3. Operating Objectives and Criteria.

a. Services. Projects are constructed and operated to provide services for authorized purposes, consistent with the authorizing legislation and applicable law.

b. Rules for Services. Shortages in many of the services of a reservoir can be very costly, whereas surpluses are usually of minor value. Accordingly, a set of rules for reservoir operation based on prioritized objectives and the current conditions of the system are followed. These are expressed in terms of operational rules that specify quantities of water to be released and diverted, quantities of power to be generated, reservoir storage to be maintained, and flood releases to be made. These quantities will normally vary seasonally and with the amount of storage water in the system. Operational rules can be entered into the HEC-ResSim software package through mathematical expressions. Operational rules are sometimes expressed through guide curves. Guide curves are developed based on historic streamflow and adjusted to forecasted future conditions or they are based on synthetic stream flows representative of forecasted future runoff potential. Guide curves are discussed in Section 4-6.

4-4. Reservoir System Simulation.

a. Computation Interval. The evaluation of reservoir system operation under specified operation rules and a set of input quantities is complex and requires detailed simulation of the operation over long periods of time. This is accomplished by assuming that steady-state conditions prevail for successive intervals of time. The time interval must be short enough to capture the details that affect system outputs. For example, average monthly flows may be used for most conservation purposes. However, for small reservoirs, the flow variation within a month may be important so data and computations at smaller time intervals (e.g., days or hours) may be needed. For hydropower reservoirs, the average monthly pool level or tail water elevation may not give an accurate estimate of energy production.

b. Process. To simulate the operation during each interval, the simulation solves the continuity equation with the reservoir release as the decision variable. The system is analyzed in an upstream-to-downstream direction. At each pertinent location, requirements for each service are noted, and the reservoirs at and above that location are operated in such a way as to serve those requirements (e.g., water demands), subject to system constraints such as outlet capacity, channel capacity, and reservoir storage capacity. As the computation procedure progresses to downstream locations, the tentative release decisions made for upstream locations become increasingly constraining. It often becomes necessary to assign priorities among services that conflict. Where power generation causes flows downstream to exceed channel capacity, for example, a determination must be made as to whether to curtail power generation. If there is inadequate water at a diversion to serve the canal and river requirements, a decision must be made.

c. Software Package. The modeling approach and software package (or packages) used should consider the best method to realistically and practicably represent the system. For systems with adequate precipitation and streamflow data and manageable complexity, continuous rainfall-runoff simulation may be the best method. Refer to the Hydrologic Engineering Center (HEC) website (Appendix B, Entry 14, p B-2) for available software packages and manuals.

4-5. Flood Risk Management Simulation. Flood discharge can change rapidly with time. Therefore, steady-state conditions cannot be assumed to prevail for long periods of time (such as 1 month). Also, physical constraints such as outlet capacity and the ability to change gate settings are important. The time translation for flow and channel storage effects cannot ordinarily be ignored. Consequently, the problem of simulating the flood-control operation of a system can be more complex than for conservation.

a. **Computation Interval.** The computation interval necessary for flood operations is usually on the order of a few hours to one day at the most. Sometimes intervals as short as 15 or 30 minutes are necessary. It is usually not feasible to simulate conditions for long periods of time, such as the entire period of record, using such a short computation interval. However, the simulation of the full period of record may be unnecessary because most of the flows are of no consequence from a flood control standpoint. Accordingly, simulation of flood control operation is usually made only for important flood periods.

b. **Starting Conditions.** The antecedent reservoir pool elevation at the beginning of a reservoir simulation depends upon the operation of the system for conservation purposes prior to the flood. The antecedent pool elevation used also depends on the purpose of the modeling.

(1) If the reservoir model is being used for dam and spillway design, antecedent pools must be determined based on the guidance in ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs. Dam and spillway design often involves an iterative approach where multiple designs are possible depending on the variation of both the height of the top of dam and the width of the spillway cross section. Geotechnical and cost considerations can also be involved through the balancing of earthwork cut and fill.

(2) If the reservoir model is being used for purposes other than spillway design and IDF analysis, conservation operation could be simulated first to establish the state of the system at the beginning of the month during which the flood occurred as the initial conditions for the flood simulation. The starting storage for flood operation should be based on a realistic assessment of likely future conditions. If it is likely that the conservation pool is full when a flood occurs, then that would be a better starting condition to test the flood-pool capacity. It may be possible that the starting pool would be higher if there were several storms in sequence, or if the flood operation does not start the instant excessive inflows raise the pool level into flood control space.

c. **Historic and Synthetic Sequences.** The analysis of flood operations should use both historic and synthetic floods. While simulating historic sequences are important, future floods will be different and occur in different sequences. The possibility of multiple storms, changes in the upstream catchment, and realistic flood operation should be included in the analysis. Chapter 7 presents flood-runoff analysis and Chapter 10 presents flood control storage requirements.

d. **Upstream-to-Downstream Solution.** If the operation of each reservoir in a system can be based on conditions at or above that reservoir, an upstream-to-downstream solution approach

can establish reservoir releases, and these releases can be routed through channel reaches as necessary to obtain a realistic simulation. Under such conditions, a simple simulation model is capable of simulating the system operation with a high degree of accuracy. However, as the number of reservoirs and downstream damage centers increase, the solution becomes far more complex. Priority criteria must be established among the reservoirs to establish which should release water when there is a choice among them.

e. **Combination Releases.** The Reservoir System Simulation (HEC-ResSim) (HEC 2013) software can solve for the combination of releases at upstream reservoirs that will satisfy channel capacity constraints at a downstream control point, taking into account the time translation and channel storage effects, and that will provide continuity in successive time intervals. The time translation effects can be modeled with a choice of hydrologic routing methods. Reservoir releases are determined for all designated downstream locations, subject to operation constraints. The simulation is usually performed with a limited foresight of inflows and a contingency factor to reflect uncertainty in future flow values. The concept of pool levels is used to establish priorities among projects in multiple-reservoir systems. Standard output includes output of reservoir storage, releases, and downstream flows.

4-6. Conservation Storage Simulation.

a. **Computation Interval.** While the flood control operation of a reservoir system is sensitive to short time variations in system input, the operation of a system for most conservation purposes is usually sensitive only to long-period streamflow variations. Historically, simulation of the conservation operation of a water resource system has been based on a relatively long computation interval such as a month. With the ease of computer simulation and available data, shorter computational intervals (e.g., daily) can provide a more accurate accounting of flow and storage. Some aspects of the conservation operation, such as diurnal variations in power generation in a peaking project, might require even shorter computational intervals for selected typical or critical periods to define important short-term variations.

b. **Hydropower Simulation.** Hydropower simulation requires a realistic estimate of power head, which depends on reservoir pool level, tail water elevation, and hydraulic energy losses. Depending on the size and type of reservoir, there can be considerable variation in these variables. Generally, the shorter time intervals will provide a more accurate estimate of power capacity and energy productions.

c. **Evaporation and Channel Losses.** In simulating the operation of a reservoir system for conservation, the travel time of water between points in the system is usually ignored, because it is small in relation to the typical computation interval (e.g., monthly or weekly). On the other hand, the evaporation from reservoirs can be substantial. Channel losses, as well, may also be very important in highly irrigated areas where the groundwater table is affected and it is sometimes necessary to account for such losses in natural river channels and diversion canals.

d. Guide Curves. The guide curve of a reservoir represents the seasonally varying target pool elevation of the reservoir. In the case of typical flood risk management projects, reservoir storage above the target elevation represented by the guide curve is the flood pool while storage below the guide curve is the conservation pool. The guidelines for determining the release from the reservoir are influenced by where the current pool elevation is in relation to the guide curve. Under basic operation, if the pool is below the guide curve, then the basic objective is to reduce releases to refill the pool. If the pool is above the guide curve, the basic objective is to increase releases to draw down the pool. Deviation from a guide curve is permitted but may require approval from specific agencies/authorities. Guide curves are developed to ensure water is available to meet project purposes. They are based on historic streamflow and are either adjusted to forecasted future conditions or they are based on synthetic stream flows representative of forecasted future runoff potential. Guide curves for the operation of a reservoir system for multiple purposes may consist of standard power generation and water supply requirements that will be served under normal conditions, a set of storage levels that will provide a target for balancing storage among the various system reservoirs, and maximum and minimum permissible pool levels for each season based on flood risk management, recreation, and other project requirements. Often some criteria for decreasing services when the system reservoir storage is critically low will be desirable. Figure 4-1 shows an example guide curve for Tiber Reservoir on the Marias River in Montana.

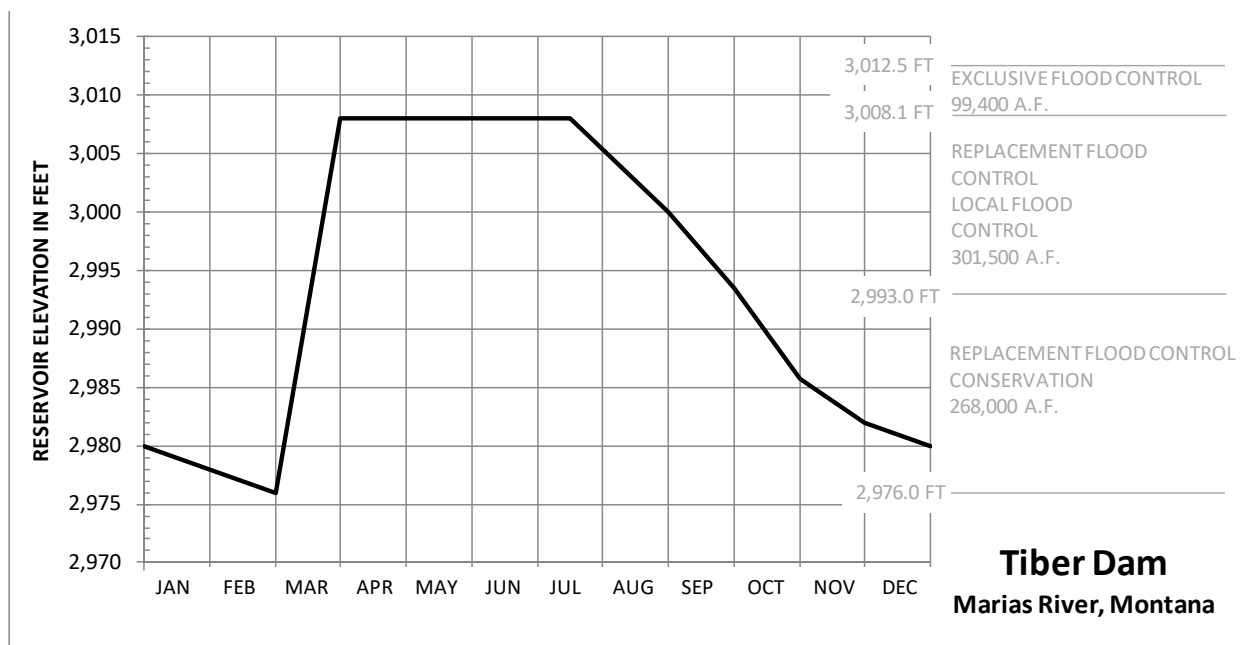


Figure 4-1. Guide Curve for Tiber Dam.

4-7. System Power Simulation. Water resource systems with a number of power plants serving the same system load usually have considerable flexibility in the selection of plants for power

generation at a given time. The overall system requirement and the minimum amount of energy that must be generated at each plant during each month (or other time interval) must be specified to simulate the hydropower generation of the system. Water releases through the powerhouse for other project purposes create hydropower generation that may meet the power required from the system. For this reason, it is first necessary to search the entire system to determine generation that would occur with only minimum power requirements at each plant and with all requirements throughout the system for other purposes. If insufficient power is generated to meet the entire system load in this manner, a search will be made for those power reservoirs where storage is at a higher level, in relation to the guide curves, than at other power reservoirs. The additional power load requirement will then be assigned to those reservoirs in such a manner as to maintain the reservoir storage as nearly as possible in conformance with the guide curves that balance storage among the reservoirs in the most desirable way. This must be done without assigning more power to any plant than it can generate at overload capacity and at the system load factor for that interval. EM 1110-2-1701, Hydropower, describes hydropower system analysis.

4-8. Determination of Hydropower Firm Yield. Hydropower firm yield can be defined as the power supply that can be maintained throughout the simulation period without shortages. If this definition is accepted for the simulation, then the process of computing the maximum yield can be expedited by maintaining a record of the minimum reserve storage (if no shortage has yet occurred) or of the amount of shortage (if one does occur) in relation to the total requirement since the last time that all reservoirs were full. The surplus or shortage that existed at the end of any computation interval can be expressed as a ratio of the supply since the reservoirs were last full, and the minimum surplus ratio (if no shortage occurs) or maximum shortage ratio (if a shortage does occur) that occurs during the entire simulation period can be used to adjust the target yield for the next iteration. This basic procedure for computing firm yield is included in the HEC-ResSim software package. Additionally, the program has a routine to make an initial estimate of the critical period and expected yield. After the yield is determined using the critical period, the program will evaluate the yield by performing a simulation with the entire input flow record. Chapter 12 describes storage-yield procedures.

4-9. Derivation of Reservoir Operating Criteria.

a. A water-resources development plan consists not only of the physical structures and their functional characteristics but also of the criteria by which the system will be operated. To compare alternative plans of development, it is necessary that each plan be operated optimally. The derivation of optimal operation criteria for a water resource system is probably more difficult than the derivation of optimum configuration and unit sizes because any small change in operation rules can affect many functions in the system for long periods of time and in very subtle ways.

b. Simulation. Operation criteria generally consist of release schedules at reservoirs, diversion schedules and minimum flows at control points, and reservoir balancing levels that define the target storage contribution among the various reservoirs in the system. All of these characteristics can vary seasonally. Once the unit sizes and target flows are established for a particular plan of development, a system of balancing levels must be developed. The system response to a change in these reservoir balancing levels is a complicated function of many system, input, and requirement characteristics. For this reason, the development of a set of balancing levels is an iterative process, and a complete system simulation must be undertaken for each iteration.

(1) It is best to establish preliminary estimates of reservoir balanced pool levels through the simulation of only the most critical periods of historic stream flows in the reservoir system. This solution should then be checked by the simulation of longer periods of time. The balancing levels defining the flood control space are first tentatively established on the basis of minimum requirements for flood control storage that will provide the desired degree of flood risk management. Preliminary estimates of other levels can be established on the basis of reserving the most storage in the smaller reservoirs, in those reservoirs with the least amount of runoff, and in those reservoirs that supply operation services not producible by other reservoirs.

(2) After a preliminary set of balancing levels is established, they should be defined approximately in terms of a minimum number of variables. The general shape and spacing of levels at a typical reservoir might be defined by the use of four or five variables, along with rules for computing the levels from those variables. Variations in levels among reservoirs should be defined by one or two variables, if possible, to reduce the amount of work required for optimization to an acceptable quantity.

(3) Optimization of a set of balancing levels for operational guide curves can be accomplished by successive approximations using a complete system simulation computation for critical drought periods. However, the procedures are limited to the input specifications of demands and storage allocation. While one can compare simulation results and conclude one is better than another based on performance criteria, there is no way of knowing that an optimum solution has been achieved.

c. Optimization and Stochastic Analysis Techniques. While water resource agencies have historically focused on simulation models for system analysis, the academic community and research literature have emphasized optimization and stochastic analysis techniques. Research performed at the HEC (1991a) found a proliferation of papers on optimization of reservoir system operations. Current state-of-the-art has expanded to include a variety of newer optimization techniques, such as the use of genetic or swarm algorithms. Reservoir operations optimization has proven useful for improving reservoir management by identifying the best possible operations and areas of practice with greatest potential improvement. Studies show that optimization is most useful when paired with simulation modeling, which can be used to test the operating schemes suggested by the optimal results.

d. Prescriptive Reservoir Model (PRM).

(1) HEC has developed a system analysis tool based on network flow programming. While many reservoir optimization models are uniquely formulated to solve a specific problem for a given system, this generalized model allows for the application of reservoir operations optimization to any system. HEC-ResPRM (2011) performs deterministic network flow optimization of reservoir system operations. The computational core of the more user-friendly HEC-ResPRM is the Hydrologic Engineering Center legacy software the Prescriptive Reservoir Model (HEC-PRM).

(2) HEC-PRM prescribes optimal values of flow and storage over time by formulating the operating problem as a minimum-cost network flow problem. The objective function of this network problem is the sum of user-defined penalty functions representing different interests in the system. The multi-objective nature of water resource problems is addressed by allowing any number of penalty functions (including those with differing units) to be added at any network location. Penalties can be quantified using monetary or nonmonetary units, with exchange rates and priorities reflected using weights. A modified simplex algorithm is used to determine the optimal allocation of water within the system.

(3) The simplicity and versatility of the HEC-ResPRM model formulation has enabled its use in a variety of studies. Fields of application include development of reservoir system operating rules, shared vision planning, multi-objective management, hydrologic-economic modeling, conflict resolution, climate change impact assessment, and trans-boundary cooperation. HEC-PRM has been applied to several large and complex water systems including the Columbia River System (HEC1991d), the Missouri River System (HEC 1991c, 2003), the Mississippi Headwaters, and California's water resource system. There are multiple techniques for deriving utility from optimization models, though continued application experience is required to define generalized procedures for these types of analysis.

(4) The primary advantages of using HEC-ResPRM are the ease with which it balances multiple objectives, identifies tradeoffs, and determines the best possible operations. This information can then be used to identify the optimal long-term operational strategy or to improve upon rules developed over years of experience and observation. It also reveals tradeoffs between different interests in the watershed.

(5) Data inputs include maximum and minimum storage for the reservoirs, period of record monthly flows, and the linking of the system through the network of channels and diversions. There are no guide curves or details of storage allocation, only basic physical constraints are defined. The other primary input data are the penalty functions. The development of penalty functions requires an economic evaluation of the values to be placed on flow and storage in the system. Alternatively the values can be represented in nonmonetary units. The process requires local expertise and collaboration, particularly if values are nonmonetary. There are multiple approaches to defining and reviewing the purposes and their relative values.

(6) The primary disadvantages of the HEC-ResPRM software are the monthly time step and lack of channel routing, which limit its application for short-interval simulation, such as flood risk management and peaking hydropower. HEC would like to improve the software by linking PRM to a linear programming solver (rather than the current network flow programming solver), which can eliminate those limitations and further expand functionality.

(7) It is also important to consider that a PRM compute optimizes over the entire time window and network simultaneously, resulting in perfect foresight and perfect system coordination. Perfect foresight of the entire time window tends to produce operations that are not achievable in reality (where foresight is not perfect) and makes the software less useful on a real-time basis. However, foresight can be limited by the option to optimize only a year at a time across the period of record. Overall, the software is best used as a planning model. Results can be reviewed and post-processed to develop improved operating policies, which can then be tested with a simulation model.

4-10. System Formulation Strategies.

a. **Alternative Evaluation.** An alternative is generally considered best in terms of the national income criteria if it results in a value for system net benefits that exceeds that of any other feasible alternative. Except where noted, the following discussion was developed in a paper presented at the International Commissions on Large Dams Congress (Eichert and Davis 1976). The number of alternatives that are feasible to evaluate are limited based on the number of reservoir system components evaluated. Exhaustive evaluation provides the strategy for determining the best system, however, when the number of components is more than just a few, then the exhaustive evaluation of all feasible alternatives cannot practically be accomplished. In this instance, a screening strategy is needed to reduce the number of alternatives evaluated to a manageable number while providing a good chance of identifying the best system. System analysis (maximum net benefit system) does not permit for reasonably complex systems even with all hydrologic-economic data known. An acceptable strategy does not need the absolute guarantee of economic optimum because seldom will the optimum economic system be selected as best.

b. **Incremental Test.** The incremental test of an individual reservoir system component value provides the basis for several alternative formulation strategies. If existing reservoir components are present in the system, then they define the base conditions. If no reservoirs exist, the base condition represents natural conditions. The strategies described below are extensions of currently used techniques and are based upon the concept of examining the performance of a selected few alternative systems in detail. The performance of each alternative is typically evaluated by traditional simulation methods, like the use of the HEC-ResSim (HEC 2013) software package.

c. **Reasoned Thought Strategy.** Reasoned thought strategy assumes that the most reasonable alternative systems can be reasoned out using judgment and other criteria. The strategy reduces alternatives to a manageable number through rational thought, sampling, public opinion,

literature search, and brainstorming. No more than 15 to 20 alternative systems could be evaluated by detailed simulation in a practical sense.

(1) The total performance of alternative in terms of economic (net benefit) and performance criteria is evaluated by a system simulation. A system (or systems if more than one have very similar performance) is selected that maximizes the contribution toward the formulation objectives (those that exhibit the highest value of net benefits while satisfying the minimum performance criteria). To confirm the incremental justification of each component, the contribution of each system component in the last added position is evaluated. The last added value is the difference between the value (net benefits) of the system with all components in operation and the value (net benefits) of the system with the last added component removed. If each component is incrementally justified, as indicated by the test, the system is economically justified, and formulation is complete. If any components are not incrementally justified, they should be dropped and the last added analysis repeated.

(2) The system selected by this strategy will be a feasible system that is economically justified. Assuming the method of devising the alternative systems is rational, the chances are good that the major worthwhile projects will have been identified. On the other hand, the chances that this system provides the absolute maximum net benefits is relatively small. This strategy would require between 30 and 60 system evaluations for a moderately complex (15 component) system.

d. First Added Strategy. This strategy is designed such that its successive application will yield the formulated system. The performance of the systems, including the base components (if any), are evaluated with each potential addition to the system in the “first added” position. The component that contributes the greatest value (net benefit) to the system is selected and added to the base system.

(1) The analysis is then repeated for the next stage by computing the first added value of each component to the system again, the base now including the first component added. The strategy is continued to completion by successive application of the first added analysis until no more component additions to the system are justified.

(2) The strategy does have a great deal of practical appeal and probably would accomplish the important task of identifying the components that are clearly good additions to the system and that should be implemented at an early stage. The strategy, however, ignores any system value that could be generated by the addition of more than one component to the system at a time, and this could omit potentially useful additions to the system. For example, the situation sometimes exists where reservoirs on, say, two tributaries above a damage center are justified, but either one analyzed separately is not (i.e., the system effect is great enough to justify both). The number of system analyses required to formulate a system based on this strategy could range upwards to 120 for a moderately complex (15 component) system, which is probably close to being an unmanageably large number of evaluations.

e. Last Added Strategy. This strategy, similar to first added strategy, is designed such that successive application yields the formulated system. Beginning with all proposed components to the system, the value of each component in the last added position is computed. The project whose deletion causes the value (net benefit) of the system to increase the most is dropped out. The net benefits would increase if the component is not incrementally justified. The strategy is continued through successive staged applications until the deletion of a component causes the total system value (net benefits) to decrease.

(1) The last added strategy will also yield a system in which all components are incrementally justified and in which the total system will be justified. This strategy would probably identify the obviously desirable projects, as would the others. However, its weakness is that it is slightly possible, though not too likely, that groups of projects that would not be justified are carried along because of their complex linkage with the total system. For example, the situation sometimes exists where reservoirs on two tributaries above a damage center are not justified together, but deletion of each from a system that includes both results in such a great loss in system value that individual analysis indicates neither should be dropped individually.

(2) The number of systems analyses required for this strategy would be similar to the first added strategy requiring perhaps 10-20% more evaluations. Twenty-two last added analyses were made in the four stages required to select four new projects out of seven alternatives. This strategy is more efficient than the first added if the majority of the potential system additions are good ones.

4-11. General Study Procedure.

a. There is no single approach to developing an optimum plan of improvement for a complex reservoir system. Ordinarily many services are fixed and act as constraints on system operation for other services. In many cases, all but one service is fixed, and the system is planned to optimize the output for one remaining service, such as power generation. It should also be recognized that most systems have been developed over a long period of time and that many services are in fact fixed, as are many system features. Nevertheless, an idealized general study procedure is presented below:

b. Prepare regional and river-system maps showing locations of hydrologic stations, existing and contemplated projects, service and damage areas, and pertinent drainage boundaries. Obtain all precipitation, evaporation, snowpack, hydrograph timing and runoff data pertinent to the project studies. Obtain physical and operational data on existing projects. Construct a normal seasonal isohyetal map for the river basin concerned.

c. Estimate the approximate nondamaging flow capacity that exists or could be ensured with minor channel and levee improvements for each location where flood risk management is to be provided. Estimate also the amount of storage (in addition to existing storage) that would be needed to provide a reasonable degree of flood risk management, using procedures described in Chapter 10. Distribute this storage in a reasonable way among contemplated reservoirs to obtain

a first approximation of a plan for flood management. Include approximate guide curves for releasing some or all of this storage for other uses during the nonflood season where appropriate.

d. Determine the approximate total water needed each month for all conservation purposes for each tributary, where appropriate, and the attendant losses. Using procedures described in Chapter 11, estimate the storage needed on each principle tributary for conservation services. Formulate a basic plan of development including detailed specification of all reservoir, canal, channel, and power plant features and operation rules; all flow requirements; benefit functions for all conservation services; and stage-damage functions for all flood damage index locations. Although this part of plan formulation is not entirely a hydrologic engineering function, a satisfactory first approximation requires good knowledge of runoff characteristics, hydraulic structure characteristics and limitations, overall hydroelectric power characteristics, engineering feasibility, and costs of various types of structures, and relocations.

e. Using the general procedures outlined in Part 2 of this EM, develop flood frequencies, hypothetical flood hydrographs, and stage-discharge relations for unregulated conditions and for the preliminary plan of development for flood risk management. It may be desirable to do this for various seasons of the year to evaluate seasonal variation of flood control space. Evaluate the flood control adequacy of the plan of development, using procedures described in paragraph 4-5 and Chapter 10, modify the plan, as necessary, to improve the overall net benefits for flood risk management while preserving basic protection where essential. Each modification must be followed by a new evaluation of net benefits for flood risk management. Each iteration is costly and time consuming; consequently, only a few iterations are feasible, and considerable thought must be given to each plan modification.

f. For system analysis to determine the best allocation of flow and storage for conservation purposes, consider optimization using the HEC-ResPRM (paragraph 4-9c). The program outputs can then be analyzed to infer an operation policy that could be defined for simulation and more detailed analysis. The alternative is to repeatedly simulate with critical low-flow periods to develop a policy to meet system goals and then perform a period of record simulation to evaluate total system performance.

g. Consider generating synthetic sequences of flow to evaluate the system's performance with different flow sequences (see paragraph 5-5). Projected changes in the basin should be factored into the analysis as well as the effects of climate change. Typically, future conditions are estimated at several stages into the future. The system analysis should be performed for each stage. While these analyses will take additional time and effort, they will also provide some indication of how responsive the system results are to changing conditions.

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PART 2:

HYDROLOGIC ANALYSIS

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CHAPTER 5

Hydrologic Engineering Data

5-1. Meteorological Data.

a. General. Meteorological data typically recorded at weather stations include air (sometimes water) temperature, precipitation, wind, and evaporation. As indicated below, more extensive recording of various types of data is often made for special purposes. The primary source of historic meteorological data for the United States is the National Oceanic and Atmospheric Administration (NOAA) National Centers for Environmental Information (NCEI) in Asheville, NC (NOAA 2013a, App B-20). NCEI was formerly known as the National Climatic Data Center (NCDC).

b. Data Sources. The NCEI website has access to meteorological data from many sources including land-based stations, satellites, radars, and weather balloons. Data for meteorological modeling can be accessed through NOAA's National Operational Model Archive & Distribution System (NOMADS) (NOAA 2013b, Appendix B, Entry 23, p B-3). Publications are also available online through the NOAA Images and Publications System (IPS) (NOAA 2013c, Appendix B, Entry 17, p B-3). Real-time meteorological data is available from the National Weather Service (NWS). NOAA's Advanced Hydrologic Prediction Service also includes real-time observations (Appendix B, Entry 19, p B-3). Specific website links are provided in Appendix B and referred to throughout the text. Another source of meteorological data are local rain observation records.

c. Storm Meteorology. Runoff potential, particularly flood potential, in areas where hydrologic data are scarce can be based on knowledge of storm meteorology. Derivation of hydrologic quantities associated with various storms must take into consideration the type of storm, its path, potential moisture capacity and stability of the atmosphere, isobar, wind and isotherm patterns, and the nature and intensity of fronts separating air masses. These are usually described adequately in the synoptic charts that are prepared at regular (usually 6-hour) intervals for weather forecasting purposes and associated upper-air soundings. Where such charts are available, it is important that they be retained as a permanent record of meteorological activity for use in supplementing information contained in the regularly prepared hemispheric charts. Hemispheric charts summarize the daily synoptic situation throughout the hemisphere but do not contain all of the data that are of interest or that would have direct bearing on the derivation of design criteria.

(1) The NOAA publication Storm Data (1959-present) documents the time, location, meteorological characteristics, storm paths, deaths, injuries, and property damage of all reported severe storms or unusual phenomena. These publications are available on the NCDC IPS website (NOAA 2013c, Appendix B, Entry 17, p B-3). Another source of extreme events is the NCEI Storm Event database (NOAA 2013d, Appendix B, Entry 25, p B-3).

(2) USACE is currently building its own Extreme Storm Database, which is available on the web (see Appendix B, Entry 4, p B-1 for website). The database includes a list and map of historic

storms with links to products specific to the events. Storm data products include storm location, start and end dates, total rainfall, area, dew point, radar data, precipitation data, and depth-area duration curves. Products available depend on the specific storm.

(3) Synoptic charts can be accessed from the NCEI Climate Data Online section of the Land-Based Station Data. The link to Legacy Access leads to Analysis and Forecast Charts where United States Analysis can be selected. Choose United States Surface Analysis from the list and enter up to 3 days for which synoptic charts are wanted. Other sources of meteorological data include the National Hurricane Center in Coral Gables, FL, and state climatologists as well as U.S. Geological Survey and USACE flood reports.

d. Precipitation.

(1) Monthly summaries and datasets of observed hourly and daily precipitation data are available on the NOAA NCEI website (NOAA 2013a, Appendix B, Entry 18, p B-3). Many of these observations are currently collected by data collection platforms but historic records should be interpreted and used with care as measurements might have been manually recorded by an observer. Frequency precipitation data are also available for the updated NOAA Atlas 14 Rainfall Frequency publications through the Hydrometeorological Design Studies Center Precipitation Frequency Data Server (PFDS) (NOAA 2015a, Appendix B, Entry 20, p B-3). Sources of gridded precipitation data include the Parameter-elevation Regressions on Independent Slopes Model (PRISM) Grids developed by Oregon State (Oregon 2015, Appendix B, Entry 10, p B-2) and the NWS River Forecasting Center next-generation radar (NEXRAD) gridded precipitation for 1995 to the present (NOAA 2015b, Appendix B, Entry 24, p B-3).

(2) Local and state agencies collect precipitation data for their own use. These data could provide additional storm information. However, precipitation measurements at remote, unattended locations may not be consistently and accurately recorded, particularly where snow and hail frequently occur. For this reason, records obtained at unattended locations must be interpreted with care. When an observer is regularly on site, the times of occurrence of snowfall and hail should be noted to make accurate use of the data. The exact location and elevation of the gauge are important considerations in precipitation measurement and evaluation. For uniform use, this is best expressed in terms of latitude and longitude and in meters or feet of elevation above sea level. Of primary importance in processing the data is tabulating precipitation at regular intervals. This should be done daily for non-recording gauges with the time of observation stated. Continuously recording gauges should be tabulated hourly or at 15-minute intervals.

(3) Procedures to develop PMP estimates are presented in NWS Hydrometeorological reports and technical memorandum (Appendix B, Entry 21, p B-2). Chapter 7 of this manual provides an overview of hypothetical storms and their application to flood-runoff analysis. The Engineering Manual for developing the precipitation for Standard Project Floods (SPFs) is EM 1110-2-1411, Standard Project Flood Determinations.

e. Snow Packs. In watersheds that have a significant snowpack, the development of hydrologic design criteria requires insight into the snowpack characteristics. The two primary characteristics are the distribution of snow water equivalent (SWE) and the distribution of the snow covered area (SCA) throughout the watershed. SWE is the amount of water that the snowpack will produce per unit area when the snowpack melts. The depth of the snowpack at a location is related to SWE by the snow density, the ratio of SWE to the snow depth. Unfortunately the snow density changes continuously throughout the winter season as the snow pack compacts. As a result, snow depth measurements have to be carefully evaluated before being used to estimate SWE. SCA is often easily estimated in mountainous watersheds where there is a strong relationship between SCA and elevation as discussed further below. In watersheds without much of an elevation range it may be quite difficult to determine SCA without resorting to remote sensing or other means.

(1) Measurements of SWE. Typically SWE is found by weighing snow. This is done automatically at over 700 Snowpack Telemetry (SNOTEL) sites throughout the western United States using snow pillows. This data is available from the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service, generally on a daily basis. SWE measurements are also made in manual snow surveys at snow courses. The frequency and timing of snow surveys varies considerably with locality and snowpack conditions. Often surveys are made on the 1st and 15th of the months that are expected to include the maximum SWE of the winter season. A broad range of agencies conduct snow surveys. The NWS no longer collects SWE data at their stations, although a few Cooperative Observer Network stations, also known as COOP stations, continue to make measurements. SWE can also be measured remotely, from aircraft using gamma radiation surveys and from satellites with sensors that measure the passive microwaves emitted by the earth. Airborne surveys are conducted by the NWS National Operational Hydrologic Remote Sensing Center (NOHRSC, Appendix B, Entry 22, p B-3) along flight lines in preselected locations throughout the Northern United States. These surveys are considered accurate to within known standards. Typically measurements are made once each winter season. Satellite estimation of SWE is based on observing changes in the microwave spectrum emitted by the earth caused by the obscuration created by the snowpack. The pixel size of the SWE estimates is typically about 25 kilometers (15.5 miles). Studies have shown that passive microwave estimation of SWE is most accurate in watersheds with less than 20% forest cover and less than 200 mm (7.87 in.) of SWE, which limits its usefulness in mountainous regions of the United States.

(2) Observations of SCA. In mountainous watersheds the SCA may be easily estimated by visual observations from the ground in areas where the snowline (the lowest elevation where snow exists) is clearly visible. In watersheds with heavy vegetation or limited sight lines it may be difficult to see the snow line. More and more, satellite imagery is being used to determine SCA. Satellites with optical sensors (sensors that measure the radiance of the earth surface in the wavelengths that are used in human sight) can easily detect snow because of its high contrast with the surrounding ground. However, clouds easily confound snow detection, and heavy forest vegetation can obscure

snow on the ground. At this time satellite detection of SCA for use in hydrologic studies is not routine, and is a subject of much research.

f. Using Models to Estimate SWE and SCA. Numerical modeling is a technique that can produce estimates of SWE and SCA if the meteorological conditions in the watershed are known or can be estimated. Essentially precipitation that falls when air temperatures are near or below freezing will accumulate as SWE. Air temperatures above freezing will melt the snowpack. Snow models differ on the sophistication of the techniques used to estimate snow melt and the internal conditions within the snowpack.

(1) HEC-HMS includes a temperature-index snow model that can be used to estimate snow accumulation and melting. This model requires only air temperature and precipitation as input, and has shown good results in many watersheds.

(2) The National Weather Service NOHRSC produces daily estimates of SWE throughout the United States at a 1 kilometer scale using the Snow Data Assimilation System (SNODAS), a numerical model driven by a NWS Mesoscale meteorology model. SNODAS assimilates SWE observations made throughout the United States to correct for errors, especially problems with precipitation, which is notoriously difficult to model correctly at a fine scale. In regions where ground-based SWE observations are available, SNODAS can produce accurate results, but its accuracy is reduced in regions with few SWE observations.

(3) The NOHRSC website (NOHRSC 2013) (Appendix B, Entry 22, p B-3) includes interactive maps of the contiguous United States and Alaska showing the model results. The interactive maps allow the selection of a date, time, and physical element (i.e., SWE) to view. Archive data of past model results are available. Both vector and raster datasets are also available for download and viewing in GIS systems.

g. Temperatures. In most hydrologic applications of temperature data, maximum and minimum temperatures for each day at ground level are very useful. Continuous records of diurnal temperature variations at selected locations can be used to determine the daily temperature pattern fairly accurately at nearby locations where only the maximum and minimum temperatures are known. In applying temperature data to large areas, it must be recognized that temperatures normally decrease with increasing elevation and latitude. It is also important to preserve all of the original temperature records. Summaries of daily maximum and minimum temperatures should be maintained and, where feasible, published. Monthly, daily, and hourly station records and normal temperature data are available online on the NCEI website (NOAA 2013a) (Appendix B, Entry 18, p B-3).

h. Moisture. Atmospheric moisture is a major factor influencing the occurrence of precipitation. This moisture can be measured by atmospheric soundings that record temperature, pressure, relative humidity, and other items. Total moisture in the atmosphere can be integrated and

expressed as a depth of water. During storms, the vertical distribution of moisture in the atmosphere ordinarily follows a rather definite pattern. Total moisture can therefore be related to the moisture at the surface, which is a function of the dew point at the surface. Accordingly, a record of daily dew points is of considerable value. Here, again, the elevation, latitude and longitude of the measuring station must be known. The NOAA publication Local Climatological Data contains observed dew points, pressure, and temperature data and is available online through the NCDC IPS (NOAA 2013c, Appendix B, Entry 17, p B-3).

i. Winds. Probably the most difficult meteorological element to evaluate is wind speed and direction. Quite commonly, the direction of surface wind reverse diurnally, and wind speeds fluctuate greatly from hour to hour and minute to minute. There is also a radical change of wind speed and direction with altitude. The speed and direction at lower levels is greatly influenced by obstructions such as mountains, and locally by small obstructions such as buildings and trees. Accordingly, it is important that great care is exercised in selecting a location and altitude for wind measurement. For most hydrologic applications, wind measurements at elevations of 5 to 15 meters above the ground surface are satisfactory. The best elevation for wind measurements is the standard level of 10 meters above the ground surface. It is important to preserve all basic records of winds, including data on the location, ground elevation, and the height of the anemometer above the ground. An anemometer is an instrument for measuring and indicating the force or speed of the wind. Where continuous records are available, hourly tabulations of speed and direction are highly desirable. Total wind movement and the prevailing direction for each day are also useful data. Wind data are summarized in the Local Climatological Data and Climatological Data publications available on the NCDC IPS (NOAA 2013c, Appendix B, Entry 18, p B-3). They are also available through the GIS-Based Map Interface (NOAA 2018, Appendix B, Entry 21, p B-2).

j. Evaporation. Evaporation data is usually required for reservoir studies, particularly for low-flow analysis. Reservoir evaporation is typically estimated by measuring pan evaporation or computing potential evaporation. There are several methods of estimating potential evaporation, based on meteorological information. Pruitt (1990) reviewed various approaches in an evaluation of the methodology and results published in A Preliminary Assessment of Corps of Engineer Reservoirs, Their Purposes and Susceptibility to Drought, (HEC 1990b).

(1) Evaporation is usually measured by using a pan about 4 feet (1.2 meter) in diameter filled with water to a depth of about 8 in. (0.2 m). Daily evaporation can be calculated by subtracting the previous day's reading from today's reading and adding the precipitation for the intervening period. The pan should be occasionally refilled and this fact noted in the record. This volume of added water, divided by the area of the pan, is equal to the daily evaporation amount expressed in inches or millimeters. A tabulation of daily evaporation amounts should be maintained and, if possible, published. It is essential that a rain gauge be maintained at the evaporation pan site, and it is usually desirable that temperature, dew point (or wet-bulb temperature) and low-level wind measurements also be made at the site for future study purposes.

(2) NOAA Technical Report NWS 33, Evaporation Atlas for the Contiguous United States (Farnsworth, Thompson, and Peck 1982), provides maps showing annual and May-October evaporation in addition to pan coefficients for the contiguous United States. Companion report NWS 34, Mean Monthly, Seasonal, and Annual Pan Evaporation for the United States documents monthly evaporation data that were used in the development of the evaporation atlas. Daily observed evaporation data are published for each state in NOAA publications Local Climatological Data and in Climatological Data publications available on the NCDC IPS website (Appendix B, Entry 17, p B-3).

(3) The Cold Regions Research and Engineering Laboratory (CRREL) and the Engineering and Research Development Center (ERDC) have developed a new method to automate reservoir evaporation rates. The methodology requires no new instrumentation, works within the “CWMS framework,” and is physically based. Contact CRREL for references.

k. Upper-Air Soundings. Upper-air soundings are available from the NOAA NCEI in Asheville, NC. The soundings provide atmospheric pressure, temperature, dew point temperature, wind speed, and direction data from which lapse rate, atmospheric stability and jet stream strength can be determined. These meteorological parameters are necessary to a comprehensive storm study. Upper-Air data are available on the NWS Telecommunication Operations Center’s website that publishes standard barotropic level fax charts (NWS 2013) (Appendix B, Entry 27, p B-4).

5-2. Topographic Data.

a. Geographic Information Systems (GISs). GISs are essential sources of digital spatial and topographic data. These systems link land attributes, topographic data, and other information concerning processes and properties to geographic locations. HEC software packages used to process geographic information include the Hydrologic Engineering Center Geospatial Hydrologic Modeling Extension (HEC-GeoHMS) and HEC-GeoRAS. One form of digital topography used as input into the listed software packages are digital elevation models (DEMs).

b. Digital Elevation Models. DEMs are rasters, or grids, containing elevation data. The raw data used to generate DEMs are collected from a variety of sources including aerial surveys, digitized cartographic map contour overlays, and National Aerial Photography photographs (NAPP) (USGS 2013a). The USGS is the main source of DEMs in the form of the National Elevation Dataset (NED). DEMs are produced from the Light Detection and Ranging (LiDAR) method or other remote sensing technology such as Interferometric synthetic aperture radar (IfSAR or InSAR). LiDAR can be topographic or bathymetric. Only bathymetric LiDAR penetrates water while topographic is reflected by the water surface.

(1) The vertical and horizontal resolutions of DEMs vary and metadata will need to be referenced. Common horizontal DEM resolutions are the 10 meter (1/3 arc-second, 30 feet) and 30 meter (1 arc-second, 100 feet). The average vertical resolution of the USGS NED is 2.44 meters (8 feet) (USGS 2013c). DEMs produced by LiDAR data can have a horizontal resolution of 1 meter. It is

important to check the metadata of a DEM for its horizontal and vertical resolutions to determine the accuracy of results processed from the data.

(2) It is also important to check the coordinate system of the data and determine if it is a geographic or projected coordinate system. The USGS NEDs are distributed with geographic coordinates (latitude and longitude) and must be transformed into a projected coordinate system (e.g., Albers Equal Area, Universal Transverse Mercator, State Plane, etc.) before accurate information (e.g., overland slopes) can be generated through GIS processing. The Albers Equal Area projection is standard for CWMS modeling work. This projection preserves area, which is important when applying rainfall to the ground surface.

(3) DEMs are the basic input data used to calculate overland slopes, generate contours, and delineate watersheds in HEC-GeoHMS. DEMs may be downloaded using the USGS National Map Viewer and Download Platform (USGS 2013b, Appendix B, Entry 6 p B-1) or other service and are often available internally in USACE. The DEM grid size resolution used in HEC-GeoHMS is a balance between watershed size and DEM file size as GeoHMS has a limit on DEM size for processing.

c. Stream Patterns and Profiles. Stream patterns can be delineated from aerial photography, estimated through the use of HEC-GeoHMS, and/or obtained from an organization or vendor. One source of river and stream coverages for the United States is the National Hydrologic Dataset (NHD) (Appendix B, Entry 31, p B-4). In the case of processing stream profiles through HEC-GeoHMS, the resolution of the base DEM data is very important and results should be compared with aeriels or other sources. Where detailed studies of floodplains are required, computation of water surface profiles is necessary. Basic data needed for this computation include detailed cross sections of the river and overbank areas at frequent intervals. These are usually obtained by special field surveys, aerial photography, and/or GIS processing. When these surveys are made, it is important to document and date the data and resulting models, then permanently preserve the information so it is readily available for future reference. Observations of actual water surface elevations during maximum flood stages (high-water marks) are invaluable for calibrating and validating models for profile computations.

d. Watersheds. A Watershed is the land area that drains to a common waterway or point location. Watershed boundaries are often produced using HEC-GeoHMS in USACE projects. These boundaries are then compared with published watershed boundaries such as the Hydrologic Unit Codes (HUCs) published by the USGS. HUCs can be downloaded from several sources such as the USGS National Map Viewer and Download Platform, the NOHRSC, and the U.S. Department of Agriculture Geospatial Data Gateway (Appendix B, Entry 6 [p B-1], 23 [p B-3], and 13 [p B-2]).

e. Lakes and Swamps. The rate of runoff from any watershed is greatly influenced by the existence of lakes, swamps, and similar storage areas. It is therefore important to indicate these

areas on project maps. The National Land Cover Database available through the Multi-Resolution Land Characteristics Consortium (MRLC 2006, Appendix B, Entry 7, p B-1) is one source of this information. Data on the outlet characteristics of lakes are important because, in the absence of outflow measurements, the outflow can often be computed using the relationship between the amount of water stored in the lake and its outlet characteristics.

f. Soil and Geology. Certain maps of soils and geology can be very useful in surface water studies if they show characteristics that relate to the perviousness of the basin. These can be used for estimating loss rates during storms. Of particular interest are areas of extensive sandy soils that do not contribute to runoff and areas of limestone and volcanic formations that are highly pervious and can store large amounts of water beneath the surface in a short time. Additionally, watershed sediment yield estimates will depend on similar information.

(1) Three sources of soil data include the Soil Survey Geographic (SSURGO) database, the State Soil Geographic (STATSGO) database, and the National Soil Geographic (NATSGO) database. The three sources vary in their level of detail. SSURGO has the most detailed soil data and is intended for township, county, or parish scale planning and management. STATSGO data is intended for planning and management on the regional, state, or river basin scale. NATSGO data is the most generalized of the three sources and is intended for planning on the national scale (NRCS 1995).

(2) SSURGO and STATSGO data can be accessed through the U.S. Department of Agriculture Natural Resources Conservation Service Soil Data Mart (USDA 2013) (see Appendix B, Entry 13, p B-2 for website address).

(3) Gridded SSURGO data (gSSURGO) has been developed in 10-meter grid sizes and can be used in place of the vector polygons for determining important soil parameters like hydraulic conductivity and available water storage at different soil depths. A source for these data is the U.S. Department of Agriculture Natural Resources Conservation Service GeoSpatialDataGateway (USDA 2015) (see Appendix B, Entry 12, p B-2 for website address).

g. Land Use. Land use type affects the hydrologic response of a watershed especially if a historically rural area has been urbanized. The National Land Cover Database (NLCD) categorizes 16 land uses: open water, perennial ice/snow, developed-open space, developed-low intensity, developed-medium intensity, developed-high intensity, barren land, deciduous forest, evergreen forest, mixed forest, shrub/scrub, grassland/herbaceous, pasture/hay, cultivated cropland, woody wetlands, and herbaceous wetlands (PE&RS 2006). Land uses within the raster are given numeric class codes (i.e., 11 means open water, 21 means developed-open space, 90 means woody wetlands, etc.) whose land types can be determined from the USGS Land Cover Institute (LCI) website (USGS 2013d). Land uses can be generalized into broader categories with less data (i.e., urban development, wetlands, etc.) using tools in GIS systems. The resolution of the

2006 data is 30-meters. The data can be downloaded from the MRLC NLCD website (MRLC 2006, Appendix B, Entry 7, p B-3).

h. Existing Improvements. Streamflow at any particular location can be greatly affected by hydraulic structures located upstream. It is important, therefore, that essential data be obtained on all significant hydraulic structures located in and upstream from a study area. For diversion structures, detailed data are required on the size of the diversion dam, capacity of the diversion canal, and the probable size of flood required to wash out the diversion dam. In the case of storage reservoirs, detailed data on the relation of storage capacity to elevation, location, and size of outlets and spillways, types, sizes, and operation of control gates, and sizes of power plant and penstocks should be known. Bridges can produce backwater effects that will cause upstream flooding. This flooding may be produced by the approach roads, constriction of the channel and floodplain, pier shapes, the angle between the piers and the streamflow, or the pier length-width ratios. Sources of these data depend upon the geographic area of the study, the owner agencies of the structures, when the structure was designed and built, and the importance of the structure and its documentation. Operation and Maintenance Manuals are a good source of these data for Corps of Engineers structures. The National Inventory of Dams (NID) is also an informative source (USACE 2015a, Appendix B, Entry 16, p B-2). A reference for information on bridges is the National Bridge Inventory from the Federal Highway Administration (FHWA 2015, Appendix B, Entry 15, p B-2).

i. Other Topological Data. Many types of spatial data at different resolutions are available online. Some websites are provided in Appendix B.

5-3. Streamflow Data. The availability of streamflow data is a significant factor in the selection of an appropriate technical method for reservoir studies. It is important to be cognizant of the nature, source, reliability, and adequacy of available data. If estimates are needed, the assumptions used should be documented, and the effect of errors in the estimates on the technical procedure and results should be considered.

a. Measurement. Streamflow data are usually best obtained by means of a continuous record of river stage, supplemented by frequent measurements of flows that can be related to corresponding river stages. It is important that stage measurements be made at a good control section, even if a weir or other control structures must be constructed. Each meter measurement should consist of velocity measurements within each of several (6-20, where practical) subdivisions within the channel cross section. Velocity for a subdivision is usually taken as the velocity at a depth of 60% (0.6) of the distance from the surface to the streambed or as the average of velocities taken at 20 and 80% (0.2 and 0.8) of the depth at the middle of the subdivision. River stage readings should be made immediately before and after the cross section is metered. The average of these two stages is the stage associated with the measurement. The measurement is computed by integrating the rates of flow in all subdivisions of the cross section.

(1) Measurements of stream velocity and computed streamflow are usually recorded on standard forms. When measurements have been made for a sufficient range of flows, the rating curve of flow versus stage can be developed. The rating curve can be used to convert the continuous record of stage into a continuous record of flow. The flows should be averaged for each day to construct a tabulation of mean daily flows. This constitutes the most commonly published record of runoff.

(2) For flood studies, it is particularly important to obtain accurate records of short-period variations during high river stages and to obtain meter measurements at or near the maximum stage during as many floods as possible. Where the river profile is very flat, as in estuaries and major rivers, it is advisable to obtain measurements frequently on the rising and on the falling stage to determine if a looped, or hysteresis, effect exists in the rating. The reason for this is that the hydraulic slope can change greatly, resulting in different rating curves for rising and falling stages.

b. Streamflow Data Sources. The USGS is the primary agency for documenting and publishing flow data in the United States. Flow data for each state are available from the U.S. Geological Survey's National Water Information System (NWIS) website (USGS 2013e, Appendix B, Entry 33, p B-4). NOAA's Advanced Hydrologic Prediction System is also an excellent source for real-time stream data for flood monitoring (see Appendix B, Entry 19, p B-2 for the website address). Streamflow is not measured directly but determined from a series of concurrent streamflow measurements using a current meter and water stage readings. A rating curve is then developed relating the water stage at the site with streamflow. Rating curves are often shifted to account for changes in the measured velocity of the cross section due to ice affects and changes in geomorphology. In addition, rating curves are developed by agencies other than the USGS (e.g., NWS and state agencies) so streamflow discharges recorded by various agencies will likely vary due to differences in rating curves.

c. Flow Modifications with Time. Reservoirs substantially alter the distribution of flow in time. Many other developments, such as urbanization, diversions, or cultivation and irrigation of large areas can also have a significant effect on watershed yield and the distribution of flow in time. The degree that flows are modified depends on the scale and manner of the development, as well as the magnitude, time, and areal distribution of rainfall (and snowmelt, if pertinent). Most reservoir evaluations require an assessment based on a consistent flow data set. Various terms are used to define what condition the data represent:

(1) Natural conditions in the drainage basin are defined as the hydrologic conditions that would prevail if no regulatory works or other development affecting basin runoff and streamflow were constructed. The effects of natural lakes and swamp areas are included.

(2) Present conditions are defined as the conditions that exist at, or near, the time of study. If there are upstream reservoirs in the basin, the observed flow record would represent "regulated flow." Flow records, preceding current reservoir projects, would be adjusted to reflect those project operations to have consistent "present conditions" flow.

(3) Unregulated conditions reflect the present (or recent) basin development, but without the effect of reservoir management. Unlike natural conditions, which are difficult to determine, only the effect of reservoir operation and major diversions are removed from the historic data.

(4) Without-project conditions are defined as the conditions that would prevail if the project under consideration were not constructed but with all existing and future projects under construction assumed to exist.

(5) With-project conditions are defined as the conditions that will exist after the project is completed and after completion of all projects having an equal or higher priority of construction.

5-4. Adjustment of Streamflow Data. The adjustment of recorded streamflow is often required before the data can be used in water-resources development studies. This is because flow information usually is required at locations other than gauging stations and for conditions of upstream development other than those under which flows occurred historically. In correlating flows between locations, it is important to use “natural” flows (unaffected by artificial storage and diversion) that correlation procedures will apply logically and efficiently. In generating flows, natural flows should be used because general frequency functions, characteristic of natural flows, are employed in this process.

a. Natural Conditions. When feasible, flow data should be converted to natural conditions. The conversion is made by adding historical storage changes (plus net evaporation) and upstream diversions (less return flows) to historical flows at the gauging stations for each time interval in turn. Under some conditions, it may be necessary to account for differences in channel and over-bank infiltration losses, flow diversions, travel times, and other factors.

b. Unregulated Conditions. It is not always feasible to convert flows to natural conditions. Often, required data are not available. Also, the hydrologic effects and timing of some basin developments are not known to sufficiently define the required adjustments. An alternative is to adjust the data to a uniform basin condition, usually near current time. The primary adjustments should remove special influences, such as major reservoirs and diversions that would cause unnatural variations of flow.

c. Reservoir Holdouts. The primary effect of reservoir operation is the storage of excess river flow during high flow periods, and the release of stored water during low-flow periods. The flow adjustment process requires the addition of the change in water stored (holdouts) in each time step to the observed regulated flow. Holdouts, both positive and negative, are routed down the channel to each gauge and algebraically added to the observed flow. Hydrologic routing methods, typically used for these adjustments, are described in Chapter 9 of EM 1110-2-1417, Flood-Runoff Analysis. The HEC Data Storage System Visual Utility Engine (HEC-DSSVue) software (HEC 2009) provides a convenient data management system and utilities to route flows and add, subtract, or adjust long time-series flow data.

d. **Reservoir Losses.** The non-project inflow represents the flow at the project site without the reservoir and includes runoff from the entire effective drainage area above the dam, including the reservoir area. Under non-project conditions, runoff from the area to be inundated by the reservoir is ordinarily only a fraction of the total precipitation that falls on that area. However, under project conditions infiltration losses over the reservoir area are usually minimal during a rainfall event. Thus, practically all the precipitation falling on the reservoir area will appear as runoff. Therefore, the inflow will be greater under project conditions than under non-project conditions, if inflow is defined as total contribution to the reservoir before evaporation losses are considered. To determine the amount of water available for use at the reservoir, evaporation must be subtracted from project inflow. In operation studies, non-project inflow is ordinarily converted to available water in one operation without computing project inflow as defined above. This is done in one of two ways: by means of a constant annual loss each year with seasonal variation or with a different loss each period, expressed as a function of observed precipitation and evaporation. These two methods are described in the following paragraphs.

(1) The constant annual loss procedure consists of estimating the evapotranspiration and infiltration losses over the reservoir area for conditions without the project, and the evaporation and infiltration losses over the reservoir area with the project. Non-project losses are usually estimated as the difference between average annual precipitation and average annual runoff at the location, distributed seasonally with precipitation and temperature variations. These are expressed in millimeters of depth. Under project conditions, infiltration losses are usually ignored, and losses are considered to consist of only direct evaporation from the lake area, expressed in millimeters for each period. The difference between these losses is the net loss due to the project. Figure 5-1 illustrates the differences between non-project and project losses.

(2) The variable loss approach uses historical records of long-term average monthly precipitation and evaporation data to account for the change in losses due to a reservoir project. This is accomplished by estimating the average runoff coefficient, the ratio of runoff to rainfall, for the reservoir area under pre-project conditions and subtracting this from the runoff coefficient for the reservoir area under project conditions. The runoff coefficient for project conditions is usually 1.0, but a lower coefficient may be used if substantial infiltration or leakage from the reservoir is anticipated. The difference between preproject and project runoff coefficients is the net gain expressed as a ratio of precipitation falling on the reservoir. This is often estimated to be 0.7 for semi-arid regions. This increase in runoff is subtracted from gross reservoir evaporation, often estimated as 0.7 of pan evaporation, to obtain a net loss.

e. **Other Losses.** In final project studies it is often necessary to consider other types of project losses that may be of minor importance in preliminary studies. Often, these losses cannot be estimated until a project design has been adopted. The importance of these losses is dependent upon their relative magnitude. That is, losses of 5 cubic meters per second (m^3/s) might be considered unimportant for a stream that has a minimum average annual flow of 1,500 m^3/s . Such losses, though, would be significant from a stream with a minimum average annual flow of 25 m^3/s . Various types of losses are discussed in the following paragraphs.

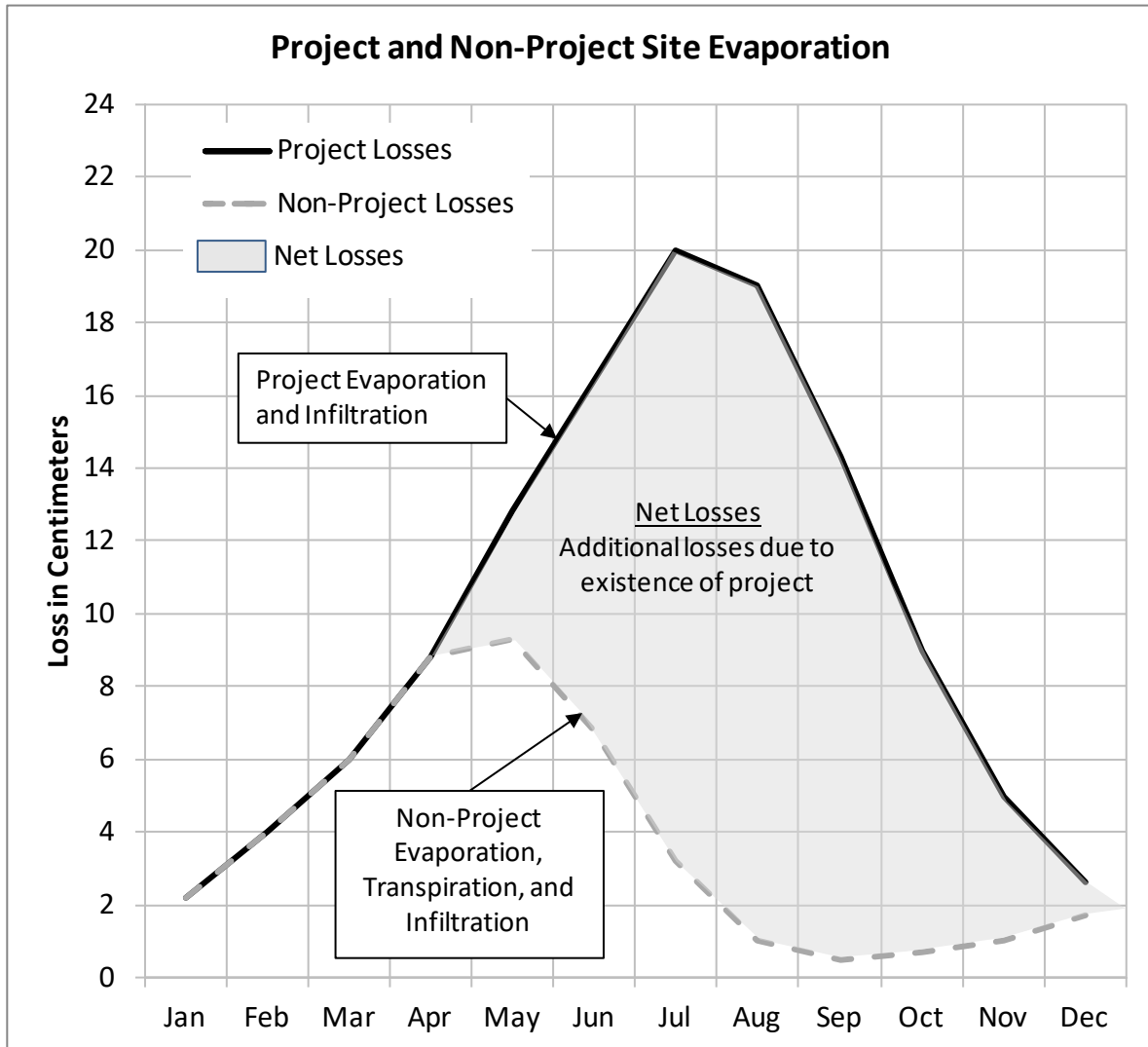


Figure 5-1. Project and Non-project Reservoir Losses.

(1) The term “losses” may not actually denote a physical loss of water from the system as a whole. Usually, water unavailable for a specific project purpose is called a “loss” for that purpose although it may be used at some other point or for some other purpose. For example, water that leaves the reservoir through a pipeline for municipal water supply or fish hatchery requirements might be called a loss to power. Likewise, leakages through turbines, dams, conduits, and spillway gates are considered losses to hydropower generation, but they are ordinarily not losses to flow requirements at a downstream station. Furthermore, such losses that become available for use below the dam should be added to inflow at points downstream from the project.

(2) Leakage at a dam or in a reservoir area is considered a loss for purposes that depend on availability of water at the dam or in the reservoir itself. These purposes include power generation, pipelines from the reservoir, and any purpose that uses pump intakes located at or above the dam. As a rule, leakage through, around, or under a dam is relatively small and is difficult to quantify before a project is actually constructed. In some cases, the measured leakage at a similar dam or geologic area may be used as a basis for estimating losses at a proposed project. The amount of leakage is a function of the type and size of dam, the geologic conditions, and the hydrostatic pressure against the dam.

(3) Leakage from conduits and spillway gates is a function of gate perimeter, type of seal, and head on the gate, and it varies with the square root of the head. The amount of leakage may again be measured at existing projects with various types of seals, and a leakage rate computed per meter of perimeter for a given head. This rate may then be used to compute estimated leakage for a proposed project by using the proposed size and number of gates and the proposed head on the gates.

(4) If a proposed project will include power, and if the area demand is such that the turbines will sometimes be idle, it is advisable to estimate leakage through the turbines when closed. This leakage is a function of the type of penstock gate, type of turbine wicket gate, number of turbines, and head on the turbine. The actual leakage through a turbine may be measured at the time of acceptance and during annual maintenance inspections, or the measurements of similar existing projects may be used to estimate leakage for a proposed project. An estimate of the percent of time that a unit will be closed may be obtained from actual operational records for existing units in the same demand area. The measured or estimated leakage rate is then reduced by multiplying by the proportion of time the unit will be closed. For example, suppose that leakage through a turbine has been measured at $0.1 \text{ m}^3/\text{s}$, and the operation records indicate that the unit is closed 60% of the time. The average leakage rate would be estimated at 0.1×0.6 or $0.06 \text{ m}^3/\text{s}$.

(5) The inclusion of a navigation lock at a dam requires that locking operations and leakages through the lock be considered. The leakage is dependent upon the lift or head, the type and size of lock, and the type of gates and seals. Again, estimates can be made from observed leakage at similar structures. Water required for locking operations should also be deducted from water available at the dam site. These demands can be computed by multiplying the volume of water required for a single locking operation times the number of operations anticipated in a given time period and converting the product to a flow rate over the given period.

(6) Water use for purposes related to project operations is often treated as a loss. Station use for sanitary and drinking purposes, cooling water for generators, and water for condensing operations have been estimated to be about $0.06 \text{ m}^3/\text{s}$ per turbine at some stations in the southwestern United States. Examining operation records for comparable projects in a given study area may also be useful in estimating these losses. If house units are included in a project to supply the project's power requirements, data should be obtained from the designer to estimate water used by the units.

(7) The competitive use of water should also be considered when evaluating reservoir losses. When initially estimating yield rates for various project purposes at a multiple-purpose project, competitive uses of water are often treated as losses. For example, consider a proposed project on a stream with an average minimum usable flow of $16 \text{ m}^3/\text{s}$. The reservoir of this project is to supply $1.5 \text{ m}^3/\text{s}$ by pipeline for downstream water supply and $2.0 \text{ m}^3/\text{s}$ for a fish hatchery in addition to providing for hydroelectric power production. The minimum average flow available for power generation is thus, $16 - (1.5 + 2.0) = 12.5 \text{ m}^3/\text{s}$. Care should be exercised in accounting for all such competitive uses when making preliminary yield estimates.

f. Missing Data.

(1) After recorded flows are converted to uniform conditions, flows for missing periods of record at each pertinent location should be estimated by correlation with recorded flows at other locations in the region. Usually, only one other location is used, and linear correlation of flow logarithms is used. It is more satisfactory, however, to use all other locations in the region that can contribute independent information on the missing data. Although this would require a large amount of computation, the stochastic analysis models can be used.

(2) Flow estimates for ungauged locations can be estimated satisfactorily on a flow per basin area basis in some cases, particularly where a gauge exists on the same stream. In most cases, however, it is necessary to correlate mean flow logarithms (and sometimes standard deviation of flow logarithms) with logarithms of drainage area size, logarithm of normal seasonal precipitation, and other basin characteristics. Correlation procedures and suggested basin characteristics are described in Chapter 9 of EM 1110-2-1415, Hydrologic Frequency Analysis.

g. Preproject Conditions. After project flows for a specified condition of upstream development are obtained for all pertinent locations and periods, they must be converted to preproject (non-project) conditions. Non-project conditions are those that would prevail during the lifetime of the proposed project if the project was not constructed. This conversion is made by subtracting projected upstream diversions and storage changes and by accounting for evaporation, return flows, differences in channel infiltration, and timing. Where non-project conditions will vary during the project lifetime, it is necessary to convert to two or more sets of conditions, such as those at the start and end of the proposed project life. Separate operation studies would then be made for each condition. This conversion to future conditions can be made simultaneously with project operation studies, but a separate evaluation of non-project flows is usually required for economic evaluation of the project.

5-5. Simulation of Streamflow Data.

a. Introduction. The term “simulation” has been used to refer to both the estimation of historic sequences and the assessment of probable future sequences of streamflow. The former reference refers to the use of continuous precipitation-runoff models to simulate streamflow based on meteorological input such as rainfall and temperature. The latter reference refers to the use of

stochastic (probabilistic) models that employ Monte Carlo simulation methods to estimate the probable occurrence of future streamflow sequences. Assessment of the probable reliability of water resource systems can be made given the assessment of probable future sequences of flow. Statistical methods used in stochastic models can also be employed to augment observed historic data by filling in or extending observed streamflow records.

b. **Historic Sequences from Continuous Precipitation-Runoff Models.** Many different types of continuous simulation runoff models have been used to estimate the historic sequence of streamflow that would occur from observed precipitation and other meteorological variables. Among the most prominent is HEC-HMS with the Soil Moisture Accounting loss method used. Fleming and Neary (2004) reports on continuous hydrologic modeling using HEC-HMS. Other continuous simulation models have also been used including the various forms of the Stanford Watershed Model (Mays and Tung 1992) by the North Pacific Division (USACE 2002). For a further description of the application of the models see EM 1110-2-1417 Section 8.

c. **Stochastic Streamflow Models.** Stochastic streamflow models are used to assess the probable sequence of future flows. As with any model, a model structure is assumed, parameters are estimated from observed data, and the model is used for prediction (Salas et al. 1980). Typically, stochastic streamflow models are used to simulate annual and/or monthly streamflow volumes.

(1) Although many different methods have been proposed in the research literature, regression is most commonly used as the basis for stochastic streamflow models. These regressions involve both correlation between flows at different sites and correlation between current and past flow periods, termed serial correlation. The correlation between sites is useful in improving parameters estimates from regional information. The serial correlation between periods is important in modeling the persistence or the tendency for high flow or drought periods. A random error component is added to the regression to provide a probabilistic component to the model.

(2) The model parameters are estimated to preserve the correlation structure observed in the observed data. If the appropriate correlation method is preserved, then the regression residuals should closely approximate the behavior that would be expected from a random error component.

(3) Model prediction is performed via the application of Monte Carlo simulation. Monte Carlo simulation is a numerical integration technique. This numerical technique is necessary because the stochastic model effectively represents a complex joint probability distribution of stream flows in time and space that cannot be evaluated analytically. The simulation is performed by producing random sequences of flows via a computer algorithm that employs random number generators. These sequences of flows are analyzed to assess supply characteristics, for example the probability for a certain magnitude or duration of drought. The number of flow sequences generated is sufficient when the estimated probabilities do not change significantly with the number of simulations.

d. **Assessment of Reliability with Stochastic Streamflow Models.** The advantage of using a stochastic streamflow model over that of employing only historic records is that it can be used to provide a probabilistic estimate of a water resource system's reliability. For example, the probability that a particular reservoir will be able to meet certain goals can be estimated by simulating the stochastic flow sequences with a reservoir simulation model. Once again, the number of flow sequences used are sufficient when the estimate of the probabilities stabilize.

e. **Available Software for Stochastic Streamflow Simulation.** Monte Carlo simulation capabilities have been added to a number of HEC software packages that support simulations where thousands of events are generated and each event includes a new sample of initial conditions and model parameters. These software packages include the Watershed Analysis Tool (HEC-WAT) and HEC-HMS version 3.2 and above. The USACE Risk Management Center also has the Reservoir Frequency Analysis (RMC-RFA) model. The Flood Risk Analysis compute option of HEC-WAT uses an inner and outer loop Monte Carlo sampling approach to model parameters associated within HEC-WAT as well as for those model parameters that have uncertainty distributions defined in external models, like HEC-HMS and HEC-ResSim. RMC-RFA is more efficient than HEC-WAT but it requires the use of only one discharge rating curve for reservoir operations while HEC-WAT will iteratively run hydrograph inflows through an embedded HEC-ResSim model. Both RMC-RFA and HEC-WAT use different hydrograph shapes scaled to various event frequencies as stochastic input.

f. **Extending and Filling in Historic Records.** Statistical techniques can be used to augment existing historic records by either "filling in" missing flow values or extending the observed record at a gauge based on observations at other gauges. The statistical techniques used are called maintenance of variance extension (MOVE) and are a modification of regression-based techniques (Alley and Burns 1983 and Salas 1992). MOVE algorithms have been instituted because the variance of series augmented by regression alone is underestimated. The MOVE technology is only generally applicable when serial correlation does not exist in the streamflow records. However, monthly or annual sequences of streamflow volumes usually do exhibit a degree of serial correlation. In these circumstances, the information provided by the longer record station may not be useful in extending a shorter record station. For a discussion of the impact of serial correlation see Matalas and Langbein (1962) and Tasker (1983).

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CHAPTER 6

Hydrologic Frequency Determinations

6-1. Introduction. Frequency curves are commonly used in Corps of Engineers studies to determine the economic value of flood reduction projects. Reservoir applications of frequency curves also include the determination of reservoir stage for real estate acquisition and reservoir-use purposes, the selection of rainfall magnitude for synthetic floods, and the selection of runoff magnitude for sizing flood control storage. EM 1110-2-1415, Hydrologic Frequency Analysis, discusses hydrologic frequency methodologies in more detail. The Hydrologic Engineering Center Statistical Software Package (HEC-SSP) can be used for frequency analysis.

a. Annual and Partial Duration Frequencies. Two basic types of frequency curves are used in hydrologic work, annual and partial durations. Figure 6-1 compares the events collected from a stream gauge for both the annual and partial duration series of stream flows for the USGS gauge at Omaha, Nebraska. Only a portion of the record is shown for the gauge for illustration purposes.

b. A duration series of annual maximum events is typically used when the primary interest is in the very large events (e.g., 1 or 0.2% annual exceedance, 100 or 500 year) or when the second largest event in any year is of minor concern in the analysis. Series of annual maximum events are used in Bulletin 17C analysis of observed annual maximum streamflow for the generation of peak flow frequencies (e.g., 1% exceedance flow, 100-year flow). These peak flow frequencies are used extensively in flood studies. This is discussed further in a later section.

(1) The partial duration series represents the frequency of all independent events above a given arbitrary base value, regardless of whether two or more occurred in the same year. This type of curve is typically not used in flood analysis but is used in economic analysis when there are substantial damages resulting from the second largest and third largest floods in extremely wet years. Damage from floods occurring more frequently than the annual event can occur in agricultural areas, when there is sufficient time between events for recovery and new investment. Working with partial duration series is more involved than working with annual series. Refer to EM 1110-2-1415, Hydrologic Frequency Analysis, for more information.

(2) When both the frequency curve of annual floods and the partial duration curve (Figure 6-1) are used, care must be exercised to assure that the two are consistent.

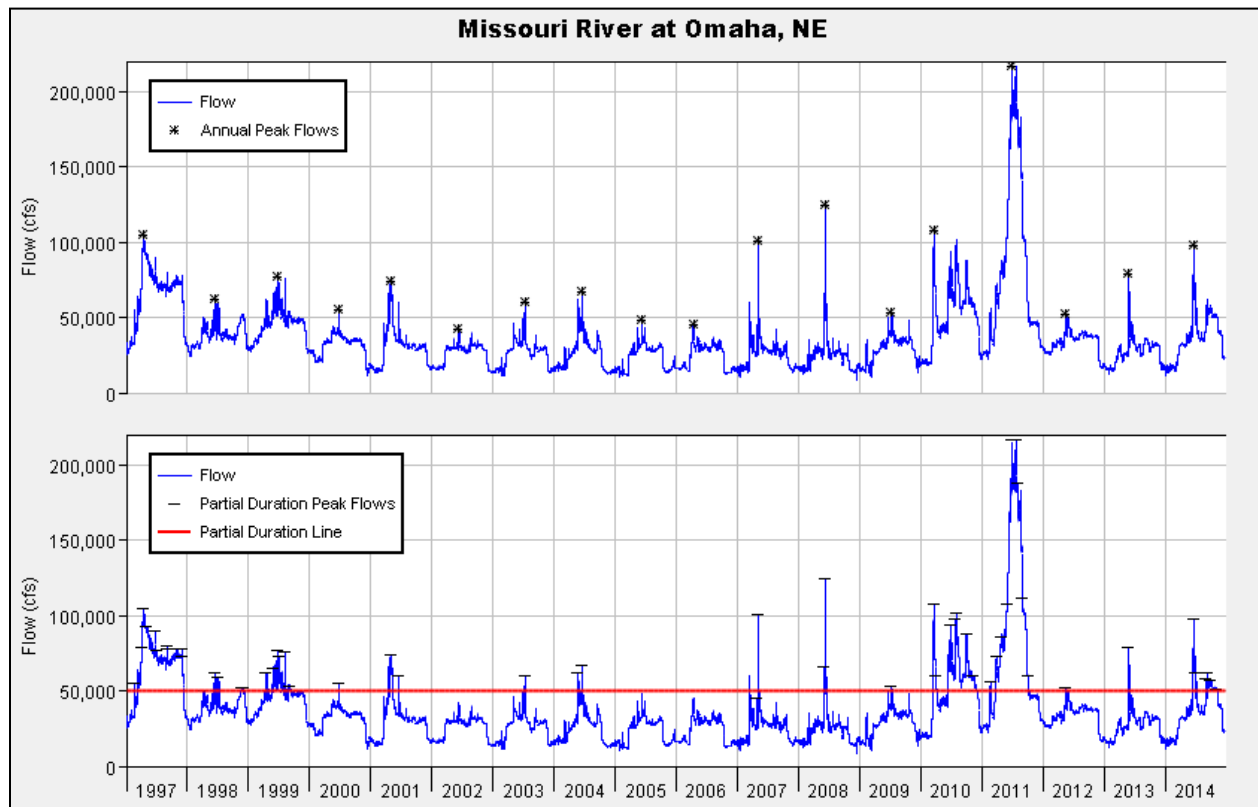


Figure 6-1. Annual and Partial Duration Flows (Sample Time Period).

6-2. Duration Curves.

a. **Flow-Duration Curve.** A flow-duration curve represents the percent of time during which specified flow rates are exceeded at a given location. Unlike flow-frequency curves, which use statistical analysis to extend results beyond the length of the period of record, flow-duration analysis pertains to only the analyzed period of streamflow record. In power studies, for run-of-river plants particularly and in low-flow studies, the flow-duration curve serves a useful purpose. Ordinarily, variations within periods less than 1 day are inconsequential, and the curves are therefore based on observed mean daily flows. For the purposes served by flow-duration curves, the extreme rates of flow are not important, and consequently there is no need for refining the curve in regions of high flow.

b. **Preparing Flow-Duration Curves.** The procedure ordinarily used to prepare a flow-duration curve consists of counting the number of mean daily flows that occur within given ranges of magnitude. The lower limit of magnitude in each range is then plotted against the percentage of days of record where mean daily flows exceed that magnitude. An example of a flood duration curve is shown in Figure 6-2. The HEC Statistical Software Package (HEC-SSP) can be

used to prepare flow-duration curves. Duration analysis can also be accomplished using the math function (statistics) of the Data Storage System (HEC-DSSVue) software package.

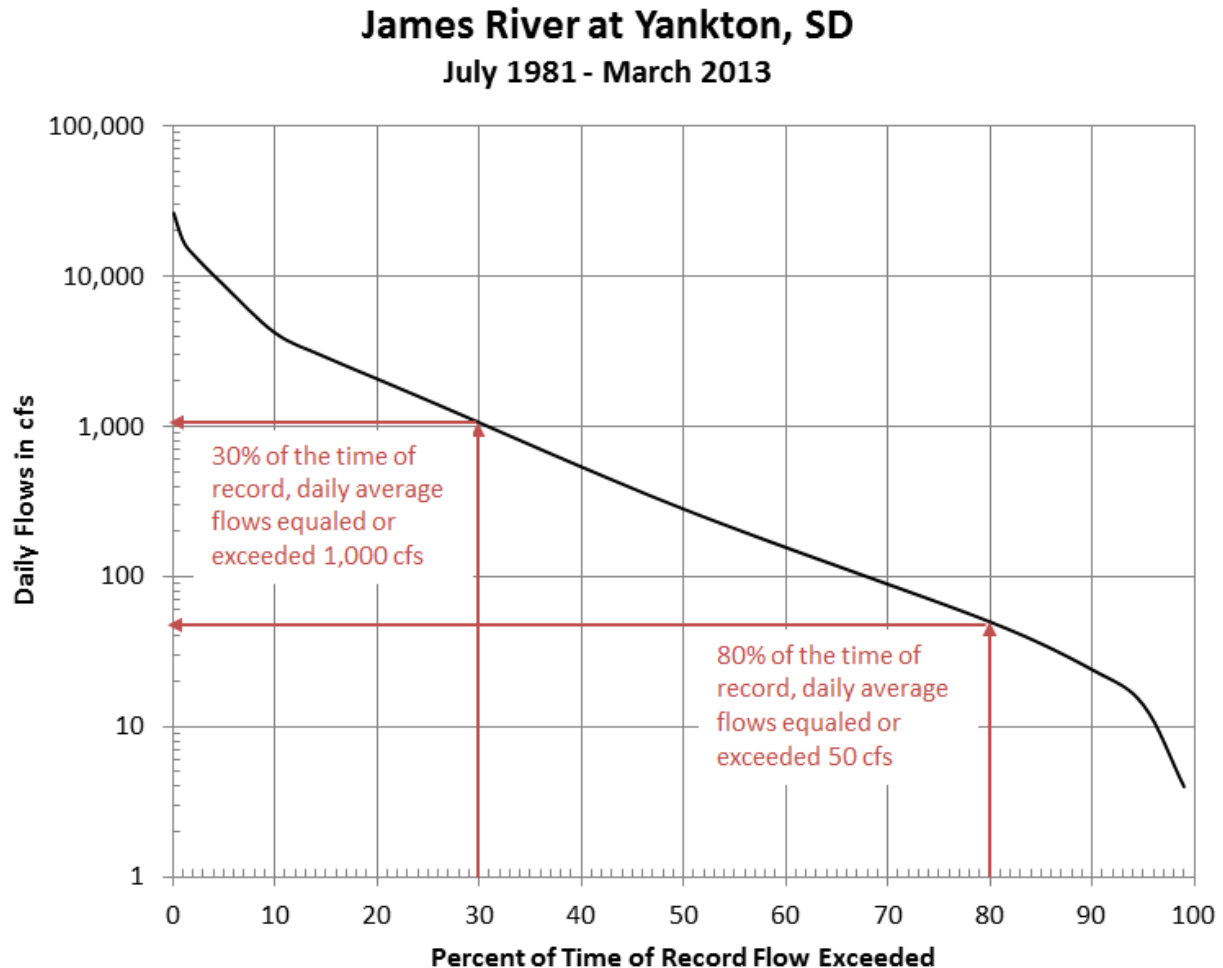


Figure 6-2. Example Flow-Duration Curve.

6-3. Flood-Frequency Curves. A flood-frequency curve is a graphical representation of a frequency distribution and is typically a comparison flood-event magnitude to event probability. Unlike duration curves, which are applicable only to the period of streamflow record used to produce the curve, flood-frequency curves use statistics to provide a probability of an event occurring. At many locations, flood stages are a unique function of flood discharges for most practical purposes. Accordingly, it is usual practice to establish a frequency curve of river discharges as the basic hydrologic determination for risk management project studies. In special cases, factors other than river discharge, such as tidal action or accumulated runoff volume, may greatly influ-

ence river stages. In such cases, a direct study of stage frequency based on recorded stages is often warranted. Refer to EM 1110-2-1415, Hydrologic Frequency Analysis, for additional information.

a. **Determination Made with Available Data.** Where runoff data at or near the site are available, flood-frequency determinations are most reliably made by the direct study of these data. Before frequency studies of recorded flows are made, the flows must be converted to a uniform condition, usually to conditions without major water management alterations or diversion. Developing unregulated flow requires detailed routing studies to remove the effect of reservoir holdouts and diversions. As damaging flows occur during a very small fraction of the total observed record time, only a small percentage of daily flows are used for flood-frequency studies. These consist of the largest flow that occurs each year and the secondary peak flows that cause damage. However, for most reservoir studies, the period of record flow will be required for analysis of nonflood purposes and impacts.

b. **Historical Data.** Flood-frequency estimates are subject to considerable uncertainty, even when fairly long records are available. To increase the reliability of frequency estimates, empirical theoretical frequency relations are used in specific frequency studies. These studies require that a complete set of data be used. To comply with this requirement, the basic frequency study ordinarily is based on the maximum flow for each year of record. Supplementary studies that include other damaging events are ordinarily made separately. The addition of historical information can be very important in verifying the frequency of large recorded events. Historical information on large damaging floods can be obtained through standard sources such as USGS websites or from newspapers and local museums. The latter sources often are more qualitative but give important insight into the relative frequency of recorded events.

c. **Selecting and Computing Frequency Curves.** The underlying general assumption made in all frequency studies is that each observed event represents an approximately equal proportion of the future events that will occur at the location, if controlling conditions do not change. Detailed procedures for selecting data and computing flood-frequency curves are presented in EM 1110-2-1415, Hydrologic Frequency Analysis, and the HEC-SSP manual (HEC 2017a).

d. **Regional Correlation of Data.** Where runoff data at or near the site do not exist or are too fragmentary to support direct frequency calculations, regional correlation of frequency statistics may be used for estimating frequencies. These correlations generally relate the mean and standard deviation of flows to drainage basin characteristics and location. Techniques of regional correlation are presented in Chapter 9 of EM 1110-2-1415, Hydrologic Frequency Analysis.

e. **Extreme Floods.** In the analysis of reservoir projects, the project's performance during floods larger than the maximum recorded events is usually required. Extrapolating derived frequency relations is uncertain, so special studies of the potential magnitudes of extreme flood events are usually required. The most practical approach is through examination of rainstorms that have

occurred in the region and determination of the runoff that would result at the project location if these storms should occur in the tributary area. This subject is discussed in Chapter 7 of this EM.

f. **Seasonal Frequency Curves.** In most locations, there are seasons when storms or floods do not occur or are not severe, and other seasons when they are more severe. Also, damage associated with a flood often varies with the season of the year. In studies where the seasonal variation is of primary importance, it becomes necessary to establish frequency curves for each month or other subdivision of the year. For example, one frequency curve might represent the largest floods that occur each January; a second one would represent the largest floods that occur each February, etc. In another case, one frequency curve might represent floods during the snowmelt season, while a second might represent floods during the rainy season. Occasionally, when seasons are studied separately, an annual-event curve covering all seasons is also prepared. Care should be exercised to assure that the various seasonal curves are consistent with the annual curve.

6-4. Estimating Frequency Curves.

a. **Approaches.** Two basic approaches exist for estimating frequency curves, graphical and analytical. Each approach has several variations, but the discussion herein will be limited to recommended methods. The primary USACE reference for computing frequency curves is EM 1110-2-1415, Hydrologic Frequency Analysis.

(1) **Graphical.** Graphical frequency curves are used when the stream-gauge site of the data is downstream of regulating structures (e.g., dams) or affected by seasonal or land management impacts. Frequencies are evaluated graphically by arranging observed values in the order of magnitude and representing frequencies by a smooth curve through the array of values. Each observed value represents a fraction of the future possibilities and, when plotting the frequency curve, it is given a plotting position that is calculated to give it the proper weight. Every derived frequency relation should be plotted graphically, even though the results can be obtained analytically. See paragraph 2-4 of EM 1110-2-1415 for more information on graphical frequency analysis.

(2) **Analytical.** Analytical frequency curves are used for natural streams that are not impacted by regulating structures upstream. In the case of peak stream flows, Bulletin 17C procedures are used to develop analytical frequency curves. Figure 6-3 shows an example peak flow-frequency curve for U.S. Geological Survey station 11274500 Orestimba Creek near Newman, California. In the application of analytical (statistical) procedures, the concept of theoretical populations or distributions is employed. The events that have been observed are presumed to constitute a random sample of the parent population of all possible flows, and they are used accordingly to make inferences regarding their “parent population” (the distribution from which they were derived). The procedure is applied to annual maxima of unregulated flow, which are assumed to be independent random events. The fact that the set of observations could result from any of many sets of physical conditions or distributions leaves considerable uncertainty in the derived curve. However, using statistical processes,

the most probable nature of the distribution from which the data were derived can be estimated. Because this in all probability is not the true parent population, the relative chance that variations from this distribution might be true must be evaluated. Each range of possible parent populations is then weighted in proportion to its likelihood to obtain a weighted average. A probability obtained from this weighted average is herein referred to as the expected probability P_N . Chapter 3 of EM 1110-2-1415 covers analytical flood-frequency analysis.

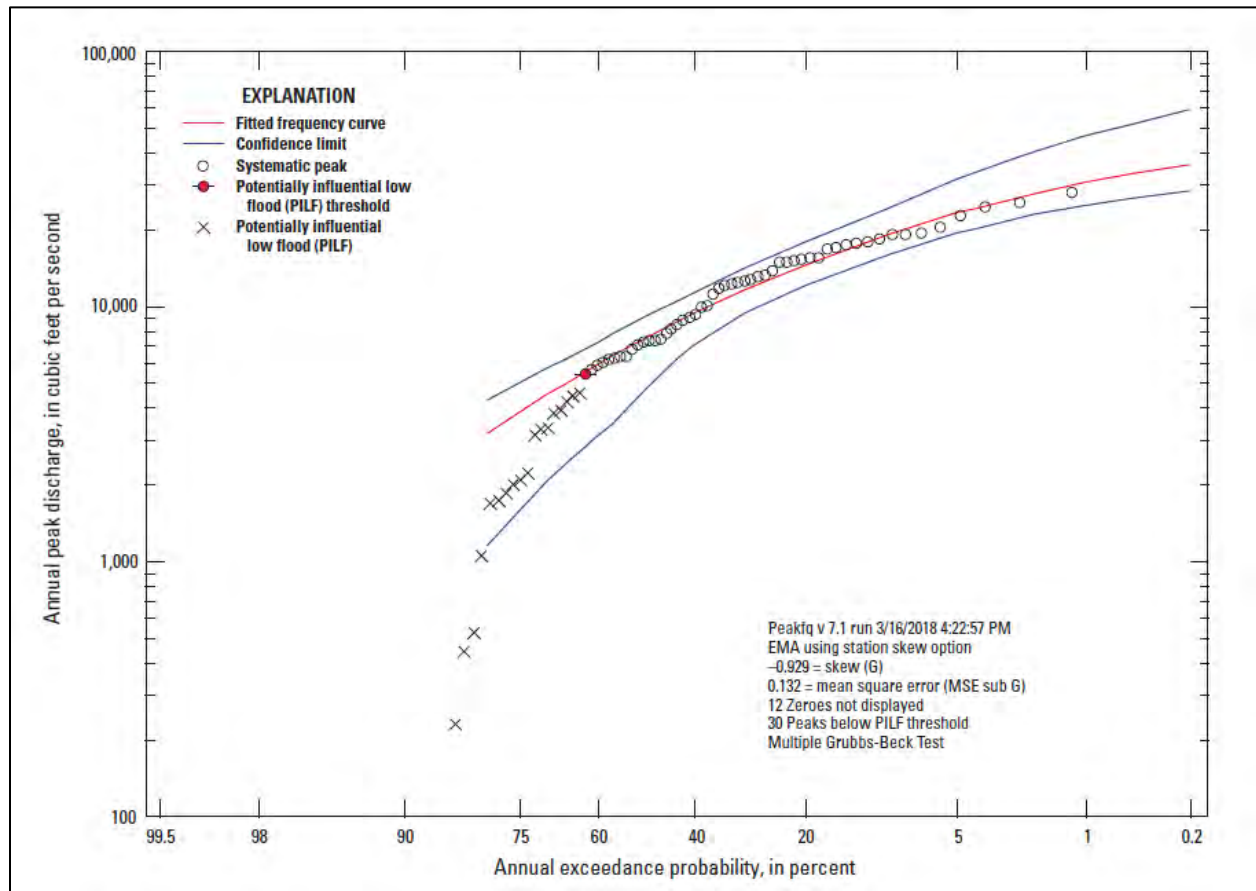


Figure 6-3. Annual Peak Flow Frequency from Annual Series of Flows.

(3) Regional. Frequency determinations for rare events are relatively unreliable because they are based on gauge records spanning at most only slightly over a hundred years. In addition, it is often necessary to estimate frequencies for locations where no record exists. For these reasons, regionalized frequency studies, in which frequency characteristics are related to drainage basin features and precipitation characteristics, are desirable. Regionalized frequency studies usually develop relationships for analytical frequency statistics. An alternative approach is to develop predictive equations for the flow for specific recurrence intervals. Chapter 9 of EM 1110-2-1415 presents regression analysis and its application to regional studies. The USGS publishes regional annual frequencies

analysis for ungauged watersheds. Refer to the USGS StreamStats website for more information (USGS 2015) (see Appendix B, Entry 34, p B-4 for website address).

b. **Flood Volume-Duration Frequencies.** A comprehensive flood volume-duration frequency series consists of a set of: 1, 3, 7, 15, 30, 60, 120, and 183-day average flows for each water year. These durations are the default durations in the software package HEC-SSP (HEC 2017a). Runoff volumes for each duration are expressed as average flows for consecutive days that peak flows and volumes for each year can be readily compared and coordinated. Paragraph 3-8 of EM 1110-2-1415 covers flood volumes.

c. **Low-Flow Frequencies.** Reservoir analysis often requires the evaluation of the frequencies of low flows for various durations. The same fundamental procedures can be used, except that minimum instead of maximum runoff values are selected from the basic data. For low flows, the effects of basin development are usually more significant than for high flows. A relatively moderate diversion may not be very significant during a flood; however, it may greatly modify or even eliminate low flows. Accordingly, one of the most important aspects of low flows concerns the evaluation of past and future effects of basin developments. Chapter 4 of EM 1110-2-1415 describes low-flow frequency analysis.

d. **Reservoir Level Frequency.** A reservoir frequency curve of annual maximum storage is typically produced through graphical methods using the procedures for flood flow frequency. Observed storage should be used to the extent available, but only if the reservoir has been operated in the past as it is expected to be operated in the future. If historical data are not available, or if it is not appropriate for future use, then reservoir routings should be used to develop data for expected reservoir operations. Stage-duration curves can be constructed from historical operation data or from simulations. These curves are usually constructed for the entire period of record, or for a selected wet or dry period. For some purposes, particularly recreational use, the seasonal variation of reservoir stages is of importance, and a set of frequency or duration curves for each month of the year may be required. Reservoir stage (or elevation) curves should indicate significant reservoir levels such as: minimum pool, top of conservation pool, top of flood control pool, spillway crest elevation, and top of dam.

6-5. Effect of Basin Developments on Frequency Relations.

a. **Flood Control Works.** Most hydrologic frequency estimates serve some purpose relating to the planning, design, or operation of water-resources management projects. The anticipated effects of a project on flooding can be assessed by comparing the peak discharge and volume frequency curves with and without the project. Also, projects that have existed in the past have affected the rates and volumes of floods, and recorded values must be adjusted to reflect uniform conditions for the frequency analysis to conform to the basic assumptions of randomness and common population. For a frequency curve to conform reasonably with a generalized mathematical or probability law, the flows must be essentially unregulated by man-made storage

or diversion structures. Consequently, wherever practicable, recorded runoff values should be adjusted to unregulated conditions before a frequency analysis is made. However, in cases where the water management results from a multitude of relatively small hydraulic structures that have not changed appreciably during the period of record, it is likely that the general mathematical laws will apply as in the case of natural flows, and that adjustment to natural conditions would be unnecessary. The effects of flood control works are presented in paragraph 3-9 of EM 1110-2-1415, and effects of urbanization in paragraph 3-10.

b. **Regulated Runoff Frequency Curves.** If it is practical, the most complete approach to determining frequency curves of regulated runoff consists of routing flows for the entire period of record through the proposed management works, arranging the annual peak regulated flows in order of magnitude, assigning a plotting position to the peak values, plotting the peak flow values at the assigned plotting position, and drawing the frequency curve based on the plotted data. A less involved method consists of routing the largest floods of record, or multiples of a large hypothetical flood, to estimate the regulated frequency curve. This approach requires the assumption that the frequency of the regulated peak flow is the same as the unregulated peak flow, which is probably true for the largest floods. Paragraph 3-9d of EM 1110-2-1415 describes these methods.

c. **Erratic Stage-Discharge Frequency Curves.** In general, cumulative frequency curves of river stages are determined from frequency curves of flow. In cases where the stage-discharge relation is erratic, a frequency curve of stages can be derived directly from stage data. Chapter 6 of EM 1110-2-1415 presents stage-frequency analysis.

d. **Reservoir or Channel Modifications.** Project construction or natural changes in streambed elevation may change the relationship between stage and flow at a location. By forming constrictions, levees may raise river stages half a meter for some distance upstream. Reservoir or channel modifications may cause changes in degradation or aggradation of streambeds, and thereby change rating curves. Thus, the effect of projects on river stages often involves the effects on channel hydraulics as well as the effects on stream flow. Consult EM 1110-2-1416, River Hydraulics, for information on modeling these potential changes.

6-6. Selection of Frequency Data.

a. **Primary Considerations.** The primary consideration in selecting an array of data for a frequency study is the objective of the frequency analysis. If the frequency curve that is developed is to be used for estimating damages that are related to instantaneous peak flows in a stream, peak flows should be selected from the record. If the damages are related to maximum mean daily flows or to maximum 3-day flows, these items should be selected. If the behavior of a reservoir under investigation is related to the 3-day or 10-day rain flood volume, or to the seasonal snowmelt volume, that should be reflected in the analysis. Normally, several durations are analyzed along with peak flows to develop a consistent relationship.

b. **Selecting a Related Variable.** Occasionally, it is necessary to select a related variable in lieu of the one desired. For example, where mean daily flow records are more complete than the records of peak flows, it may be desirable to derive a frequency curve of mean daily flows and then, from the computed curve, derive a peak flow curve by means of an empirical relation between mean daily flows and peak flows. All reasonably independent values should be selected, but the annual maximum events should ordinarily be segregated when the application of analytical procedures is contemplated.

c. **Data Selected.** Data selected for a frequency study must measure the same aspect of each event (such as peak flow, mean daily flow, or flood volume for a specified duration), and each event must be controlled by a uniform set of hydrologic and operational factors. For example, it would be improper to combine items from old records that are reported as peak flows but are in fact only daily readings, with newer records where the peak was actually measured. Similarly, care should be exercised when there has been significant change in upstream storage management during the period of record so as not to inadvertently combine unlike events into a single series. In such a case, the entire flow record should be adjusted to a consistent condition, preferably the unregulated flow condition.

d. **Separation of Events.** Hydrologic factors and relationships operating during a winter rain flood are usually quite different from those operating during a spring snowmelt flood or during a local summer cloudburst flood. Where two or more types of floods are distinct and do not occur predominantly in mutual combinations, they should not be combined into a single series for frequency analysis. They should be considered as events from different parent populations. It is usually more reliable in such cases to segregate the data by type and to combine only the final curves, if necessary. For example, in the mountainous region of eastern California, frequency studies are made separately for rain floods, which occur principally from November through March, and for snowmelt floods, which occur from April through July. Flows for each of these two seasons are segregated strictly by cause, those predominantly caused by snowmelt and those predominantly caused by rain. In desert regions, summer thunderstorms should be excluded from frequency studies of winter rain flood or spring snowmelt floods and should be considered separately. Similarly, in coastal regions it would be desirable to separate floods induced by hurricanes or typhoons from other general flood events.

e. **Data Adjustments.** When practicable, all runoff data should be adjusted to unregulated hydrologic conditions before making the frequency study because natural flows are better adapted to analytical methods and are more easily compared within a region. Frequency curves of present-regulated conditions (those prevailing under current practices of management and diversion) or of future-regulated conditions can be constructed from the frequency curve of natural flow by means of an empirical or logical relationship between natural and regulated flows. Where data recorded at two different locations are to be combined for construction of a single frequency curve, the data should be adjusted as necessary to a single location, usually the location of the longer record, accounting for differences of drainage area and precipitation and,

where appropriate, channel characteristics between the locations. Where the stream-gauge location is somewhat different from the project location, the frequency curve should be constructed for the stream-gauge location and subsequently adjusted to the project location.

f. **Runoff Record Interruptions.** Occasionally, a runoff record may be interrupted by a period of one or more years. If the interruption is caused by the destruction of the gauging station by a large flood, failure to fill in the record for that flood would have a biasing effect, which should be avoided. However, if the cause of the interruption is known to be independent of flow magnitude, the entire period of interruption should be eliminated from the frequency array, since no bias would result. In cases where no runoff records are available on the stream concerned, it is possible to estimate the frequency curve as a whole using regional generalizations. An alternative method is to estimate a complete series of individual floods from recorded precipitation by continuous hydrologic simulation and perform conventional frequency analysis on the simulated record.

6-7. **Climatic Change.** In the past, the USACE has assumed a stationary climatic baseline with the assumption that climate in the past will reflect climate in the future. Scientific evidence shows that in some locations and for some impacts relevant to USACE operations, climate change is shifting the climatological baseline about which the natural climate variability occurs, and may be changing the range of that variability as well. This is relevant to the USACE because the assumptions of stationary climatic baselines and a fixed range of natural variability as captured in the historic hydrologic record may no longer be appropriate for long-term projections of climatological parameters. Climate change analysis is now required in USACE projects. At the date of this EM update, the USACE does not have specific guidance on accounting for climate change in design but climate change should be at least qualitatively considered pending publication of guidance. Reference the **Engineering and Construction Bulletin: Guidance for Climate Change Adaptation Engineering Inputs to Inland Hydrology for Civil Works Studies, Designs, and Projects** for guidance as of this update. Refer to new ERs and EMs for future guidance involving quantitative analysis.

6-8. **Frequency Reliability Analyses.**

a. **Influences.** The reliability of frequency estimates is influenced by the amount of information available, the variability of the events, and the accuracy with which the data were measured.

(1) In general with regard to the amount of information available, errors of estimate are inversely proportional to the square root of the number of independent items contained in the frequency array. Therefore, errors of estimates based on 40 years of record would normally be half as large as errors of estimates based on 10 years of record, other conditions being the same.

(2) The variability of events in a record is usually the most important factor affecting the reliability of frequency estimates. For example, the ratio of the largest to the smallest annual flood of record on the Mississippi River at Red River Landing, LA, is about 2.7; whereas the ratio of the largest to the smallest annual flood of record on the Kings River at Piedra, CA, is about 100 or 35 times as great. Statistical studies show that as a consequence of this difference in variability, a flow corresponding to a given frequency that can be estimated within 10% on the Mississippi River, can be estimated only within 40% on the Kings River.

(3) The accuracy of data measurement normally has relatively little influence on the reliability of a frequency estimate, because such errors ordinarily are not systematic and tend to cancel. The influence of extreme events on reliability of frequency estimates is greater than that of measurement errors. For this reason, it is usually better to include an estimated magnitude for a major flood than to ignore it. For example, a flood event that was not recorded because of gauge failure should be estimated, rather than to omit it from the frequency array. However, it is advisable to always use the most reliable sources of data and to guard against systematic errors.

b. Errors in Estimating Flood Frequencies. It should be remembered that possible errors in estimating flood frequencies are very large, principally because of the chance of having a non-representative sample of the parent population of possible events. Sometimes the occurrence of one or two rare flood events can change the apparent exceedance frequency of a given magnitude from once in 1,000 years to once in 200 years. Nevertheless, the frequency curve technique is considerably better than any other tool available for certain purposes and represents a substantial improvement over using an array restricted to observed flows only. Reliability criteria useful for illustrating the accuracy of frequency determinations are described in Chapter 8 of EM 1110-2-1415.

6-9. Presentation of Frequency Analysis Results. Information provided with frequency curves should clearly indicate the scope of the studies and include a brief description of the procedure used, including appropriate references. When rough estimates are adequate or necessary, the frequency data should be properly qualified to avoid misleading conclusions that might seriously affect the project plan. A summary of the basic data consisting of a chronological tabulation of values used and indicating sources of data and adjustments made would be helpful. The frequency data can also advantageously be presented in graphical form, ordinarily on probability paper, along with the adopted frequency curves.

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CHAPTER 7

Flood-Runoff Analysis

7-1. Introduction. Flood-runoff analysis is usually required for any reservoir project. Even without flood risk management as a purpose, a reservoir must be designed to safely pass flood flows. Rarely are there sufficient flow records at a reservoir site to meet all analysis requirements for the evaluation of a reservoir project. This chapter describes the methods used to analyze the flood hydrographs and the application of hypothetical floods in reservoir projects. Most of the details on methods are presented in EM 1110-2-1417, Flood Runoff Analysis. The dam safety standards are dependent on the type and location of the dam. ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs, defines the requirements for design floods to evaluate dam and spillway adequacy. Requirements for flood development and application are also provided.

7-2. Flood Hydrograph Analysis.

a. Unit Hydrograph Method. The standard USACE procedure for computing flood hydrographs from catchments is the unit hydrograph method. The fundamental components are:

(1) Rainfall and/or snowmelt analysis to determine the time-distributed average precipitation input to each catchment area.

(2) Infiltration, or loss, analysis to determine the precipitation excess available for surface runoff.

(3) Unit hydrograph transforms to estimate the surface-flow hydrograph at the catchment outflow location.

(4) Base flow estimation to determine the subsurface contribution to the total runoff hydrograph.

(5) Hydrograph routing and combining to move catchment hydrographs through the basin and determine total runoff at desired locations.

(6) The kinematic-wave approach is often used to compute the surface-flow hydrograph for urban catchments instead of unit hydrograph transforms. Each of the standard flood-runoff and routing procedures is described in Part 2 of EM 1110-2-1417, Flood-Runoff Analysis. The Hydrologic Modeling System (HEC-HMS) (HEC 2017b) is a generalized software package that provides standard methods for performing the required computations for basin modeling.

b. Rainfall-Runoff Parameters. Whenever possible, unit hydrographs and loss rate characteristics should be derived from the reconstitution of observed storm and flood events on the study watershed, or nearby watersheds with similar characteristics. The HEC-HMS software

package has optimization routines to facilitate the determination of best-fit rainfall-runoff parameters for each event. When runoff records are not available at or near the location of interest, unit hydrograph and loss characteristics must be determined from regional studies of pertinent characteristics at existing gauge locations. Runoff and loss coefficients can be related to drainage basin characteristics by multiple correlation analysis and mapping procedures, as described in Chapter 16 of EM 1110-2-1417.

c. **Developing Basin Models.** Flood hydrographs may be developed for a number of purposes. Basin models provide hydrographs for historical events at required locations where gauged data are not available. Even in large basins, there will be limited gauged data and many locations where data are desired. With some gauged data, a basin model can be developed and calibrated for observed flood events. Chapter 13 of EM 1110-2-1417 provides information on model development and calibration.

d. **Estimating Runoff.** Basin models can estimate the runoff response under changing conditions. Even with historical flow records, many reservoir studies will require estimates of flood runoff under future, changed conditions. The future runoff with developments in the catchment and modifications in the channel system can be modeled with a basin runoff model.

e. **Application.** For reservoir studies, the most frequent application of flood hydrograph analysis is to develop hypothetical (or synthetic) floods. The three common applications are frequency-storms, SPFs and probable maximum floods (PMFs). Frequency-based design floods are used to develop flood-frequency information, like that required to compute expected annual flood damage. SPF and PMF are used as design standards to evaluate project performance under the rarer flood events.

7-3. Hypothetical Floods.

a. **General.** Hypothetical floods are usually used in the planning and design of reservoir projects as a primary basis of design for some project features and to substantiate the estimates of extreme flood-peak frequency. Where runoff data are not available for computing frequency curves of peak discharge, hypothetical floods generated through hydrologic modeling can be used to establish flood magnitudes for a specified frequency from rainstorm events of that frequency. This approach is not accurate at locations where variations in soil moisture conditions and rainfall distribution characteristics greatly influence flood magnitudes. In general, measured data should be used to the maximum extent possible, and when approximate methods are used, several approaches should be taken to compute flood magnitudes.

b. **Frequency-Based Design Floods.** In areas where infiltration losses are small, it may be feasible to compute hypothetical floods from rainfall amounts of a specified frequency and to assign that frequency to the flood event. NOAA Atlas 14 publishes generalized rainfall criteria for the United States and distributes it through the Hydrometeorological Design Studies Center

PFDS (NOAA 2015a) online. The site location can be entered through latitude and longitude coordinates or determined interactively from the map and point precipitations retrieved for either a partial duration or annual duration time series. Point precipitation depths must be adjusted for areas greater than 10 square miles based on precipitation duration and catchment area. For areas of the United States not currently covered by NOAA Atlas 14, Technical Paper No. 40 (TP-40, Weather Bureau 1961), NOAA Technical Memorandum NWS HYDRO-35 (NOAA 1977), Technical Paper No. 49 (TP-49), and other regional studies' frequency rainfall depths should be consulted. Section 13-4 of EM 1110-2-1417, Flood-Runoff Analysis, provides information on simulation with frequency-based design storms.

c. Standard Project Flood. The SPF is the flood that can be expected from the most severe combination of meteorological and hydrologic conditions that are considered reasonably characteristic of the region in which the study basin is located. Sometimes this event is an observed extreme event. The SPF, which provides a performance standard for potential major floods, is based on the Standard Project Storm (SPS). ER 1110-2-1464, Hydrologic Analysis of Watershed Runoff, allows the SPF to be approximated as $\frac{1}{2}$ of the PMF for sites west of the 105th meridian.

(1) The SPS is usually an envelope of all or almost all of the storms that have occurred in a given region. The size of this storm is derived by drawing isohyetal maps of the largest historical storms and developing a depth-area curve for the area of maximum precipitation for each storm. Depth-area curves for storm rainfall of specified durations are derived from this storm-total curve by a study of the average time distribution of precipitation at stations representing various area sizes at the storm center. When such depth-area curves are obtained for all large storms in the region, the maximum values for each area size and duration are used to form a single set of depth-area-duration curves representing SPS hyetographs for selected area sizes, using a typical time distribution observed in major storms. EM 1110-2-1411 is the source for generalized SPS estimates for small and large drainage basins, and projects for which SPF estimates are required.

(2) The SPF is ordinarily computed using the unit hydrograph approach with the SPS precipitation. The unit hydrograph and basin losses should be based on reasonable values for a flood of this magnitude. Part 2 of EM 1110-2-1417, Flood-Runoff Analysis, provides detailed information on the unit hydrograph procedure and the simulation of hypothetical floods is described in Chapter 13 of the same source.

d. Probable Maximum Flood (PMF). The PMF is the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the region. It is often used as the IDF for dam spillway design for Standard 1 or high hazard potential projects. The PMF is calculated from the PMP. The PMP values encompass the maximized intensity-duration values obtained from storms of a single type. Storm type and variations of precipitation are considered with respect to location, areal coverage of a watershed, and storm duration.

(1) Several Hydrometeorological Reports (HMRs) are available to quantify the PMP. Figure 7-1 shows the regions of the United States where these HMRs are applicable. HMR 51 and 52, for example, are used for the portion of the United States east of the 105th Meridian while HMR 55a applies to the region between the Continental Divide and the 103rd Meridian. HMR documentation can be accessed online from the NOAA's NWS Hydrometeorological Design Studies Center (NOAA 2012, Appendix B, Entry 22, p B-3).

(2) In the determination of both the SPF and the PMF, selection of rainfall loss rates and the starting storage of upstream reservoirs should be based on appropriate assumptions for antecedent precipitation and runoff for the season of the storm. Also, PMF studies should consider the capability of upstream reservoir projects to safely handle the PMF contribution from that portion of the watershed.

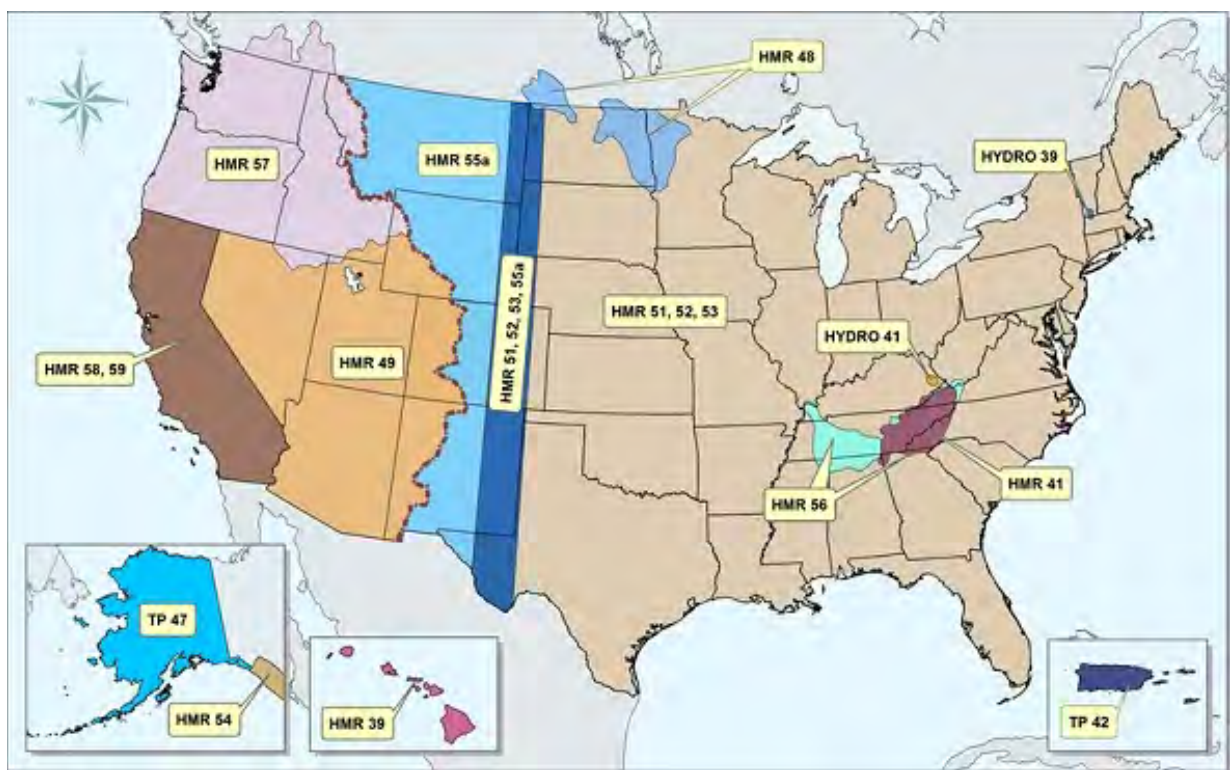


Figure 7-1. Regions Covered by Different NWS PMP Documents (NOAA 2012).

(3) In cases where there is a hydrologically deficient dam upstream of the dam being evaluated, the dam downstream should not be declared hydrologically deficit just because the upstream dam is deficient. Instead, the risk assigned to the upstream dam should be increased based on the consequences its failure has on the downstream dam. Mitigation of this risk is then focused on correcting problems at the upstream dam. However, risk mitigation alternatives can consider both changes at the upstream and downstream dams.

e. **Site-Specific PMP Estimates.** Site-specific PMP studies may be conducted when projects of considerable risk, such as a large reservoir upstream of a highly urbanized area, are designed. Site-specific studies are effort and time intensive, and their results are often subjective, based on the dew point data used and the adjustments considered. The USACE Extreme Storm Team database can be accessed to provide more information for site-specific PMP estimates. The Extreme Storm Database is available at the website listed in Appendix B (Appendix B, Entry 4, p B-1). Note that the USACE does not assign an annual frequency (e.g., 1% annual exceedance or 100-year storm) to the PMP so NOAA Atlas 14 cannot be used to determine the PMP. However, GIS shapefiles from the NOAA site can be used to scale PMP hyetographs to each subarea to provide a more realistic spatial pattern of the probable maximum precipitation.

(1) PMP estimations are based on storm representative and maximum persisting 12-hour, 1000-hPa (1000 millibar, mb) dew points (at sea level pressure). The storm representative dew point is derived by averaging dew point temperatures collected at several weather observation sites within the path of inflowing moisture to the observed extreme storm, while the maximum persisting dew point is obtained from historic data and may require frequency analysis. A source for maximum persisting dew points is the Climatic Atlas of the United States (NCDC 1968). The 1968 document is available through the NCEI website (Appendix B, Entry 18, p B-3). A persisting dew point temperature is the minimum value recorded during a specified time period. In other words, a persisting 12-hour dew point temperature will have no lesser values during those 12 hours.

(2) In many cases extreme storms must be transposed to the project watershed because an extreme storm has not been recorded over the watershed. Transposed storms should originate from areas with similar climatic and topographic characteristics as the study watershed. Usually extreme storms near the project watershed are the most useful but there is no limit on the distance of separation between the referenced extreme storm and the project watershed. Several observed extreme storms with varying durations and aerial extents should be used in the site-specific PMP estimation. Adjustments for storm elevations and intervening barriers may be necessary.

(3) The preceding discussion of site-specific PMP estimation is a very general summary. Refer to the World Meteorological Organization (WMO) Manual on Estimation of Probable Maximum Precipitation (PMP) (WMO 2009) for in-depth procedures and examples. This source can be found on the WMO website under the Mandatory Publications link (Appendix B, Entry 28, p B-4). Hydrometeorological Report 55a also contains some explanation. This source is available on the National Oceanic and Atmospheric Hydrometeorological Design Studies Center website (Appendix B, Entry 22, p B-3) under PMP Documents.

f. **Storm Duration.** For application in the design of local flood risk management projects, only peak flows and runoff volumes for short durations are usually important. Accordingly, the maximum pertinent duration of storm rainfall is only on the order of the time of travel for flows from the headwaters to the location concerned. After a reasonable maximum duration of interest is established, rainfall amounts for this duration and for all important shorter durations must be

established. For SPS determinations, this consists of the amounts of observed rainfall in the most severe storms within the region that correspond to area sizes equal to the drainage area above the project. In the case of hypothetical storms and floods of a specified frequency, these rainfall amounts correspond to amounts observed to occur with the specified frequency at stations spread over an area the size of the project drainage area. Larger rates and smaller amounts of precipitation would occur for shorter durations, as compared with the longer durations of interest. Once a depth-duration curve is established that represents the desired hypothetical storm rainfall, a time pattern must be selected that is reasonably representative of observed storm sequences. The HEC-HMS software package has the capability of accepting any depth-duration relation and selecting a reasonable time sequence. It is also capable of accepting specified time sequences for hypothetical storms.

g. Snowmelt Contribution. Satisfactory criteria and procedures have not yet been developed for the computation of standard project and probable maximum snowmelt floods. The problem is complicated in that deep snowpack tends to inhibit rapid rates of runoff, and consequently, probable maximum snowmelt flood potential does not necessarily correspond to maximum snowpack depth or water equivalent. Snowpack and snowmelt differ at various elevations, thus adding to the complexity of the problem.

(1) Where critical durations for project design are short, high temperatures occurring with moderate snow pack depths after some melting has occurred will probably produce the most critical runoff. Where critical durations are long, as is the more usual case in the control of snow melt floods, prolonged periods of high temperature or warm rainfall occurring with heavy snowpack amounts will produce critical conditions.

(2) The general procedure for the computation of hypothetical snowmelt floods is to specify an initial snow pack for the season that would be critical. In the case of SPF's a maximum observed snowpack should be assumed. The temperature sequence for SPF computation would be that which produces the most critical runoff conditions and should be selected from an observed historical sequence. In the case of PMF computation, the most critical snowpack possible should be used and it should be considerably larger or more critical than the standard project snowpack. The temperature pattern should be selected from historical temperature sequences augmented to represent probable maximum temperature for the season. Where simultaneous contribution from rainfall is possible, a maximum rainfall for the season should be added during the time of maximum snowmelt. This would require some moderation of temperatures to ensure that they are consistent with precipitation conditions. EM 1110-2-1406, Runoff from Snowmelt, covers snowmelt for design floods, standard project and maximum probable snowmelt flood derivation.

(3) Snowmelt computations can be made with an energy budget computation, accounting for radiation, evaporation, conductivity, and other factors, or by a simple relation with air temperature, which reflects most of these other influences. The latter procedure is usually more satisfactory in practical situations. Snowmelt, loss rate, and unit hydrograph computations can be made by using a

software package like the Hydrologic Modeling System, HEC-HMS. The HEC-HMS package contains energy-balance snowmelt modeling. Chapter 5 of this EM contains more information on snow packs and that data available for computations. EM 1110-2-1417, Flood-Runoff Analysis, has detailed descriptions of each computational component.

7-4. Land Acquisition Floods.

a. Development of Land Acquisition Flood. A land acquisition flood is used to determine the amount of land to acquire to minimize the impacts of the surcharge operation of a reservoir. To establish a reasonable surcharge allowance above the top of flood-pool elevation, a land acquisition flood, which includes the effects of any upstream reservoirs, should be selected and routed through the project to determine the impact on the establishment of the guide acquisition line. See Chapter 8 for information on water surface profiles for land acquisition floods.

b. Nonurban Areas. In nonurban areas, the land acquisition flood should be selected from an evaluation of a range of floods with various frequencies of occurrence. The impact of induced surcharge operations on existing and future developments, hazards to life, land use, and relocations must be evaluated. The land acquisition flood will be chosen based on an evaluation of the risk and uncertainty associated with each of these frequency events. Basic considerations to be addressed during the land acquisition flood selection process should include the credibility of the analysis, identification and significance of risk, costs and benefits, and legal (to include potential takings—see Chapter 18-4 for further guidance), social, and political ramifications.

c. Urban Areas. In urban areas or other areas with highly concentrated areas of development, the SPF will be used for the land acquisition flood.

d. Project Design Sedimentation Volume. Project capacity data should be adjusted for projected sediment volumes when routing the land acquisition flood. Project design sediment should be based on appropriate rates of sedimentation for the project area for the life of the project.

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CHAPTER 8

Water Surface Profiles

8-1. Introduction.

a. General. Water surface profiles are required for most reservoir projects, both upstream and downstream from the project. Profile computations upstream from the project define the “backwater” effect due to high reservoir pool levels. The determination of real estate requirements are based on these backwater profiles. Water surface profiles are required downstream to determine channel capacity, flow depths and velocities, and other hydraulic information for evaluation of pre- and postproject conditions.

b. Choosing a Method. The choice of an appropriate method for computing profiles depends upon the following characteristics: the river reach, the type of flow hydrograph, and the study objectives. The gradually varied, steady-flow profile computation (e.g., HEC-RAS), is used for many studies. However, the selection of the appropriate method is part of the engineering analysis. EM 1110-2-1416, River Hydraulics, and HEC-RAS reference manuals provide information on formulating a hydraulic study and a discussion of the analytical methods in general use. The following sections provide general guidance on the methods and the potential application in reservoir related studies. Refer to Hydrologic Engineering website (Appendix B, Entry 14, p B-2) for manuals and the USACE Official Publications website (Appendix B, Entry 29, p B-4) for the EMs.

8-2. Steady-Flow Analysis.

a. Method Assumptions. A primary consideration in one-dimensional, gradually varied, steady-flow analysis is that flow is assumed to be constant, in time, for the profile computation. Additionally, all the one-dimensional methods require the modeler to define the flow path when defining the cross-sectional data perpendicular to the flow. The basic assumptions of the method are as follow:

- (1) Steady-flow - depth and velocity at a given location do not vary with time.
- (2) Gradually varied flow - depth and velocity change gradually along the length of the water course.
- (3) One-dimensional flow - variation of flow characteristics—other than in the direction of the main axis of flow—may be neglected, and a single elevation represents the water surface of a cross section perpendicular to the flow.
- (4) Channel slope less than 0.1 meter/meter because the hydrostatic pressure distribution is computed from the depth of water measure vertically.

(5) Averaged friction slope - the friction loss between cross sections can be estimated by the product of the representative slope and reach length.

(6) Rigid boundary - the flow cross section does not change shape during the flood.

b. Gradually Varied Steady-Flow. The assumption of gradually varied steady-flow for general rainfall and snowmelt floods is generally acceptable. Discharge changes slowly with time and the use of the peak discharge for the steady-flow computations can provide a reasonable estimate for the flood profile. Backwater profiles, upstream from a reservoir, are routinely modeled using steady-flow profile calculations. However, inflow hydrographs from short duration, high intensity storms, e.g., thunderstorms, may not be adequately modeled assuming steady-flow.

c. Downstream Profile. Obviously, the downstream profile for a constant reservoir release meets the steady-flow condition. Again, the consideration is how rapidly flow changes with time. Hydropower releases for a peaking operation may not be reasonably modeled using steady-flow because releases can change from near zero to turbine capacity, and back, in a short time (e.g., minutes) relative to the travel time of the resulting disturbances. Dam-break flood routing is another example of rapidly changing flow that is better modeled with an unsteady flow method.

d. Flat Stream Profiles. Another consideration is calculating profiles for very flat streams. When the stream slope is less than 0.0004 meter/meter (2 feet/mile), there can be a significant loop in the downstream stage-discharge relationship. Also, the backwater effects from downstream tributaries, or storage, or flow dynamics may strongly attenuate flow. For slopes greater than 0.0009 meter/meter (5 feet/mile), steady-flow analysis is usually adequate. Be aware when integrating HEC-HMS and HEC-RAS simulations so routing in the backwater areas and storage affects are not double-counted through modeling in both software packages.

e. Further Information. Chapter 6 of EM 1110-2-1416, River Hydraulics, provides a detailed review of the assumptions of the steady-flow method, data requirements, and model calibration and application. Appendix D of the same source provides information on the definition of river geometry and energy loss coefficients, which is applicable to all the one-dimensional methods.

8-3. Unsteady Flow Analysis.

a. Unsteady Flow Methods. One-dimensional unsteady flow methods require the same assumptions listed in 8-2(a), herein, except flow, depth, and velocity can vary with time. Therefore, the primary reason for using unsteady flow methods is to consider the time varying nature of the problem. Examples of previously mentioned rapidly changing flow are thunderstorm floods, hydroelectric peaking operations, and dam-break floods. The second application of unsteady flow analysis consideration, mentioned above, is streams with very flat slopes. Unsteady flow analysis uses the full flood hydrograph and not just the peak flow of the flood.

b. Predicting Downstream Stages. Another application of unsteady flow is in the prediction of downstream stages in river-reservoir systems with tributaries, or lock-and-dam operations where the downstream operations affect the upstream stage. Flow may not be changing rapidly with time, but the downstream changes cause a time varying downstream boundary condition that can affect the upstream stage. Steady-flow assumes a unique stage-discharge boundary condition that is stable in time.

c. Further Information. Chapter 5 of EM 1110-2-1416, River Hydraulics, provides a detailed review of model application including selection of method, data requirements, boundary conditions, calibration, and application.

8-4. Multidimensional Analysis.

a. Multidimensional analysis includes both two- and three-dimensional modeling. In river applications, two-dimensional modeling is usually depth-averaged. That is, variables like velocity do not vary with depth, so an average value is computed. For deep reservoirs, the variation of parameters with depth is often important (see Chapter 12 of this EM and EM 1110-2-1201, Reservoir Water Quality Analysis). Two-dimensional models, for deep reservoirs, are usually laterally-averaged. Three-dimensional models are available; however, their applications have mostly been in estuaries where both the lateral and vertical variation are important.

b. Two-dimensional, depth-averaged analysis is usually performed in limited portions of a study area at the design stage of a project. The typical river-reservoir application requires both the direction and magnitude of velocities. Potential model applications include areas upstream and downstream from reservoir outlets. Additionally, flow around islands, and other obstructions, may require two-dimensional modeling for more detailed design data.

c. Chapter 4 of EM 1110-2-1416, River Hydraulics, provides a review of model assumptions and typical applications.

8-5. Movable-Boundary Profile Analysis.

a. Reservoirs. Reservoirs disrupt the flow of sediment and ice when they store or slow down water. At the upper limit of the reservoir, the velocity of inflowing water decreases and the ability to transport sediment decreases and deposition occurs. Chapter 9 herein presents reservoir sediment analysis. Reservoir releases may be sediment deficient, which can lead to channel degradation downstream from the project because the sediment is removed from the channel. In addition to sediment deposition, delta areas of reservoirs are also susceptible to ice jams due to the decrease in the velocity of flow. Refer to EM 1110-2-1612, Ice Engineering, for more information on ice control for flood mitigation.

b. River and Reservoir Sedimentation. USACE projects in the river and reservoir environment are affected by sediment processes that include surface erosion, sediment transport, scour,

and deposition. EM 1110-2- 4000, Sediment Investigations of Rivers and Reservoirs, is the primary USACE reference on reservoir sedimentation. Primary topics within the document include an introduction that discusses typical sediment analysis problems and scope followed by chapters on sediment properties and measurement techniques, sediment yield, river sediment transport, reservoir sediment processes, modeling, and evaluation of risk.

8-6. Water Surface Profile Computations for Land Acquisition Floods.

a. Backwater Model Development. A basic backwater model should be developed for the project area from the proposed flat pool area through the headwater area where impacts of the proposed reservoir are expected to be significant. The model should reflect appropriate cross-sectional data and include parameters based on historical flood discharges and high-water marks. EM 1110-2-1416, River Hydraulics, presents the model requirements and calibration procedures.

b. Preproject Profiles. A series of preproject water surface profiles should be developed using preproject cross section geometry, calibrated Manning's "n" values, and appropriate starting water surface elevations for the initial cross section. Flow rates used in the water surface profile computations should be selected from the peak and recession side of the land acquisition flood hydrograph.

c. Postproject Profiles. A series of water surface profiles will be developed using the postproject cross sections that are adjusted to reflect project design sedimentation over the life of the project. Manning's roughness coefficients are based on adjusted preproject roughness coefficients to account for factors such as vegetation and land use changes, which decrease hydraulic conveyance. Agricultural lands existing in the headwater areas prior to land purchases will likely revert to forested areas some years after the reservoir is filled. Preproject flow rates and coincident reservoir pool elevations from land acquisition flood routing should be used to compute postproject profiles.

d. Project Design Sedimentation Distribution. Postproject cross section geometry must be adjusted to reflect the impacts of sedimentation over the life of the project. Sedimentation problems associated with reservoir projects and methods of analysis to address sediment volumes and distributions are given in Chapter 5 of EM 1110-2-4000, Sedimentation Investigations and Rivers and Reservoirs.

e. Development of an Envelope Curve. An envelope curve connects the high points of intersection of preproject and postproject water surface profiles. The development of an envelope curve is based on preproject and postproject water surface profiles. A selected discharge from the land acquisition flood is used to compute a preproject and a postproject profile. A point of intersection is established where the profiles are within one foot of each other. The point of intersection is placed at the elevation of the higher of the two profiles. A series of points of intersection are derived from water surface profile computations using a range of selected discharges

from the land acquisition flood. A curve is drawn through the series of points of intersection to establish the envelope curve.

f. Evaluations to Determine Guide Taking Lines (GTL). The land acquisition flat pool of a reservoir project is established by the maximum pool elevation designated for storing water for allocated project purposes to include induced surcharge storage and is not impacted by the back-water effects of main stream or tributary inflows. In flat pool areas, the elevations of the GTL are based on the flat pool elevation and a freeboard allowance to account for adverse effects of saturation, bank erosion, and wave action.

g. Headwater Areas. In headwater areas, the GTL may be based on the envelope curve elevations and appropriate allowances to prevent damages associated with saturation, bank erosion, and wave action.

h. Flood Control Projects. The selection of an appropriate land acquisition flood for flood risk management projects located in rural areas should be based on an elevation of a range of frequency flood events. The land acquisition flood selection for flood-risk management projects in rural locations must include the effects of upstream reservoirs and reflect postproject conditions that minimize adverse impacts within the project area resulting from induced flood elevations and duration of flooding. In highly developed areas along the perimeter of flood control projects, the SPF should be used for land acquisition. An envelope curve can be developed from the land acquisition flood routings and water surface profile computations for preproject and postproject conditions. The land acquisition GTL may be established from the envelope curve and appropriate allowances for reservoir disturbances.

i. Nonflood Risk Management Projects. Nonflood risk management projects may be any combination of purposes such as water supply, hydropower, recreation, navigation and irrigation. The land acquisition flood selection process for nonflood risk management projects located in rural areas is based on an evaluation of a range of frequency floods and is used to determine postproject flood elevations and duration of flooding in the project area. As with flood risk management projects, management of flows by upstream reservoirs must be incorporated in the development process. The land acquisition flood used to evaluate real estate acquisitions in rural areas should reflect postproject conditions that minimize adverse impacts. The land acquisition flood for developed areas should be the SPF. The maximum pool elevation designed for storing water for allocated project purposes is used in the development of the land acquisition flood routing. An envelope curve based on preproject and postproject water surface profiles using project design sedimentation and distribution should be developed. The envelope curve and appropriate allowances for reservoir disturbances may be used to establish the land acquisition GTL.

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CHAPTER 9

Reservoir Sediment Analysis

9-1. Introduction.

a. Parameters of a Natural River. Nature maintains a very delicate balance between the water flowing in a natural river, the sediment load moving with the water, and the stream's boundary. Any activity that changes any one of the following parameters:

- (1) Water yield from the watershed.
- (2) Sediment yield from the watershed.
- (3) Water discharge duration curve.
- (4) Depth, velocity, slope or width of the flow.
- (5) Size of sediment particles.

or that tends to fix the location of a river channel on its floodplain and thus constrains the natural tendency will upset the natural trend and initiate the formation of a new one. The objective of most sediment studies is to evaluate the impact on the flow system resulting from changing any of these parameters.

b. Changes Caused by Reservoirs. Reservoirs interrupt the flow of water and, therefore, sediment. In terms of the above parameters, the reservoir causes a change in the upstream hydraulics of flow depth, velocity, etc. by forcing the energy gradient to approach zero. This results in a loss of transport capacity with the resulting sediment deposition in the reservoir. The reservoir also alters the downstream water discharge duration relation and reduces the sediment supply that may lead to the degradation of the downstream channel.

c. Analysis. Sedimentation investigations usually involve the evaluation of the existing condition as well as the modified condition. The primary areas of reservoir sediment analysis are the estimation of the volume and location of sediment deposits in the reservoir and the evaluation of reservoir releases' impact on the downstream channel system. Sediment deposits start upstream of the backwater area of the reservoir, which increase the elevation of the bed profile and the resulting water surface profile. However, reservoirs may also cause sediment deposits upstream from the project, which affect the upstream water surface profiles.

d. Further Information. The primary USACE reference for sediment analysis is EM 1110-2-4000, Sedimentation Investigations of Rivers and Reservoirs. Major topics include developing a study work plan, sediment yield, river sedimentation, reservoir sedimentation, and model studies.

9-2. Sediment Yield Studies.

a. General. Sediment yield studies determine the amount of sediment that leaves a basin for an event or over a period of time. Sediment yield, therefore, involves erosion processes as well as sediment transport and delivery to the study area. The yield provides one of the necessary inputs to determine sedimentation impacts on a reservoir.

b. Required Analysis. Each reservoir project requires a sediment yield analysis to determine the storage depletion resulting from the deposition of sediment during the life of the project. For most storage projects, as opposed to sediment detention structures, the majority of the delivered sediment is suspended. However, the data required for the headwater reaches of the reservoir should include total sediment yield by particle size because that is where the sands and gravels will deposit.

c. Further Information. Corps of Engineer methods for predicting sediment yields are presented in EM 1110-2-4000, Sedimentation Investigations of Rivers and Reservoirs. There are multiple sediment yield analysis methods. A thorough research of methods applicable to the study area is recommended early in the study process. Evaluation with one or more methods may be warranted depending upon study complexity and identified risks.

9-3. Reservoir Sedimentation Problems.

a. Sediment Deposition. As previously mentioned, the primary reservoir sediment problem is the determination of the sediment deposition rate and location within the reservoir. The determination of the sediment accumulation over the life of the project is the basis for the sediment reserve. Typical storage diagrams of reservoirs, showing sediment (or dead) storage at the bottom of the pool can be misleading. Sediment deposition often occurs in the upstream zone of the reservoir, which can adversely impact flood control storage. While the reservoir storage capacity may ultimately fill with sediment, the distribution of the deposits can be a significant concern during the life of the project. The reservoir sedimentation study should forecast sediment accumulation and deposition location over the life of the project. Sediment typically forms deltas with an unequal distribution of both sediment volume and deposited material size within the reservoir. A number of problems associated with delta formations are discussed below.

(1) Deposits forming the delta may raise the water surface elevation during flood flows, thus requiring special consideration for land acquisition on the inflowing main stem and also larger tributaries. Delta location is affected by many factors including pool level, inflowing river flow rate, sediment load, and sediment material size. Within the inflowing main stem and significant tributaries, floods of equal frequency often have higher water surface elevations after a project begins to develop a delta deposit than was experienced before the project was constructed. These backwater impacts may continue far upstream of the actual deposition area. Land acquisition studies must consider such a possibility.

(2) Aggradation problems are often more severe on tributaries than on the main stem. Analysis is complicated by the amount of hydrologic data available on the tributaries, which is usually less than on the main stem itself. Land use along the tributary often includes recreation sites, where aggradation problems are particularly undesirable.

(3) Reservoir deltas often attract phreatophytes due to the high moisture level. This may cause water use problems due to their high transpiration rate.

(4) Reservoir delta deposits are often aesthetically undesirable.

(5) Reservoir sediment deposits may increase the water surface elevation sufficiently to impact the groundwater table, particularly in shallow impoundments.

(6) In many existing reservoirs, the delta and backwater-swamp areas support wildlife. Because the characteristics of the area are closely controlled by the operation policy of the reservoir, any reallocation of storage would need to consider the impact on the present delta and swamp areas.

b. Upstream Projects. It is important to identify and locate all existing reservoirs in a basin where a sediment study is to be made. The projects upstream from the point of analysis potentially modify both the sediment yield and the water discharge duration curve. The date of impoundment is important so that observed inflowing sediment loads may be coordinated with whatever conditions existed in the basin during the periods selected for calibration and verification. In addition, sediment yield should consider that the trap efficiency of upstream reservoirs may decline as these projects fill with sediment. Useful information on the density of sediment deposits and the gradation of sediment deposits along with sediment yield are often available from other reservoirs in the basin. Information on the rate of sediment deposition that has occurred at other reservoir sites in the region is often one of the most valuable sources of information when estimating sediment deposition rate for a new reservoir.

9-4. Downstream Sediment Problems.

a. Channel Degradation. Channel degradation usually occurs downstream from the dam. Initially, after reservoir construction, the hydraulics of flow (velocity, slope, depth, and width) remain unchanged from preproject conditions. However, the reservoir acts as a sink and traps sediment, especially the bed material load. This reduction in sediment delivery to the downstream channel causes the energy in the flow to be out of balance with the boundary material for the downstream channel. As a result of the available energy to transport material, the water attempts to re-establish the former balance with sediment load from material in the stream bed, and this results in a degradation trend. In many projects, the amount of excess sediment transport can be somewhat offset by the modified flows due to reservoir operations. Initially, degradation may persist for only a short distance downstream from the dam because the equilibrium sediment load is soon re-established by removing material from the stream bed. Inflowing tributaries with significant sediment load can also affect the degradation rate and extent.

b. Downstream Migratory Degradation. As time passes, degradation tends to migrate downstream. However, several factors are working together to establish a new equilibrium condition in this movable-boundary flow system. The potential energy gradient is decreasing because the degradation migrates in an upstream-to-downstream direction. As a result, the bed material is becoming coarser and, consequently, more resistant to being moved. This tendency in the main channel has the opposite effect on tributaries. Their potential energy gradient within the degradation zone is increasing, which results in an increase in transport capacity. This often results in an upstream migrating head cut on tributaries. Finally, a new balance will tend to be established between the flowing water-sediment mixture and the boundary.

c. Extent of Degradation. The extent of degradation is complicated by the fact that the reservoir also changes the discharge duration curve. The altered flow regime will impact a considerable distance downstream from the project because the existing river channel reflects the channel forming flows from the main stem and tributaries. That channel forming flow will be changed by the operation of the reservoir. Also, the reduced flow will probably promote vegetation growth at a lower elevation in the channel. The result is a condition conducive to sediment deposition in the vegetation. Detailed simulation studies should be performed to determine future channel capacities and to identify problem areas of excessive aggradation or degradation. All major tributaries should be included.

9-5. Sediment Water Quality.

a. Sediments and Pollutants. When a river carrying sediments enters a reservoir, the flow velocity decreases and the suspended and bed load sediments start settling down. Reservoirs generally act as depositories for the sediments because of their high sediment trap efficiency. Due to a high capacity to absorb contaminants, sediments can act as sinks for contaminants in reservoirs. In agricultural and industrial areas, sediments can contain absorbed nutrients, PCB's, chlorinated hydrocarbon pesticides, oil and grease, heavy metals, pathogenic bacteria, or mutagenic substances. Burial of these contaminants by sedimentation can be an effective process in sequestering them from overlying surface waters. Toxic inorganic and organic contaminants associated with the sediments can be ingested by benthic organisms and bioconcentrated in aquatic organisms present in reservoirs.

b. Monitoring Chemical Contaminants. Incoming sediments and associated pollutants can significantly affect the water quality of the reservoir pool and downstream releases. To assess the situation it is essential that sediment reservoir interactions be characterized by their depositional behavior, particle size distribution, and pollutant concentrations. This will provide information to successfully plan a monitoring strategy to quantify contaminant movement within reservoirs. Sediment diagenesis models are currently being refined to better assess the influence of contaminated sediments in reservoirs, but have not been refined enough to be used in USACE field offices. General guidance on the design and conduct of programs for monitoring chemical contaminants in reservoir waters, sediments, and biota are provided in the following documents:

(1) World Education Studies (WES) Instruction Report E-86-1, General Guidelines for Monitoring Contaminants in Reservoirs (Waide 1986).

(2) EM 1110-2-1201, Reservoir Water Quality Analysis.

c. **Sedimentation Patterns.** Sedimentation patterns are often associated with water quality characteristics as sediment transport and deposition lead to longitudinal gradients in water quality. This is especially true in long, dendritic reservoirs (where tributary inflows are located a considerable distance from the outflow) where unidirectional flow from headwater to dam develop sedimentation patterns and water quality gradients in space and time. High concentrations of inorganic particulates can reduce water transparency and light availability near inflows, and thus reduce algal photosynthetic activity and associated dissolved oxygen production. The adsorption of dissolved nutrients, such as phosphorus, with suspended solids can reduce the bioavailability of nutrients within a reservoir.

9-6. Sediment Investigations.

a. **General.** The level of detail required for the analysis of any sediment problem depends on the objective of the study. Chapters 1 and 2 of EM 1110-2-4000, Sedimentation Investigations of Rivers and Reservoirs, describes typical sediment related issues, sediment analysis scope, staged sedimentation studies, problem identification and the development of a study work plan.

b. **Sediment Deposits.** Considering a dam site as an important natural resource, it is essential to provide enough volume in the reservoir to contain anticipated deposits during the project life. If the objective of a sediment study is only to estimate the rate of sediment deposition for use in screening studies, then estimates of sediment yield combined with reservoir trap efficiency can provide a satisfactory solution. Sustainable design should also consider the long-term effects of sediment deposition on project function and evaluate practical methods to achieve a sediment balance. Sediment flushing and bypass are examples of sustainable design methods that should be considered.

c. **Land Acquisition.** If the sediment study must address land acquisition for the reservoir, then knowing only the volume of deposits is not sufficient. The location of deposits must also be known, and the study must take into account sediment movement. This generally requires simulation of flow in a mobile boundary channel. Sorting of grain sizes must be considered because the coarser material will deposit first, and armoring must be considered because scour is involved. Movable-bed modeling is useful to predict erosion or scour trends downstream from the dam, general aggradation or degradation trends in river channels, and the ability of a stream to transport the bed material load. The software package HEC-RAS (HEC 2018a) is designed to provide long-term trends associated with changes in the frequency and duration of the water discharge and/or stage or from modifying the channel geometry.

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d. Details of Investigations. The details of reservoir sedimentation investigations, sediment properties, and modeling are covered in EM 1110-2-4000. The reservoir sediment investigation should evaluate both the existing and project condition. The analysis should consider sustainable design as well as water quality and environmental issues.

PART 3:

RESERVOIR STORAGE REQUIREMENTS

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CHAPTER 10

Flood Control Storage

10-1. General Considerations.

a. Reservoir flood storage should be considered when flood damage at a number of locations on a river can be significantly reduced by the construction of one or more reservoirs. It should also be considered if a reservoir site immediately upstream from one damage center provides more economical flood risk management than local flood risk management works. Whenever such reservoirs can serve needs other than flood risk management, the integrated design and operation of the project for multipurpose use should be considered.

b. It is important in the planning, design, and evaluation of the flood control features of a reservoir that the degree and extent of flood risk management be greater than that provided by a local flood risk management project. This means that the storage space and release schedule for flood risk management must be provided at all times when the flooding potential exists to be comparable with channel and levee improvement alternatives. In some regions this may be for the entire year, but more commonly there are dry seasons when the flood potential is greatly reduced and storage reservation for flood risk management can be reduced correspondingly.

c. It is typically not feasible to provide flood control space only after a flood is forecast. Exceptions to this are where spring snowmelt floods can be forecasted reliably or where safe release rates are sufficient to empty flood space in a very short time. This means that the flood control space must be maintained for large flood events during seasons of flooding and not be used as multipurpose storage at those times. Flood pools must also be drawn down as quickly as possible after capturing large inflow to provide protection from possible future events. Space must be provided at all times during the flood season unless it can be demonstrated that the necessary space can be evacuated on a realistic forecast basis. Also, space may be reduced if less storage is needed due to low snowpack, or there is some other reliable basis for long range flood forecasting.

d. Runoff volumes for pertinent durations are critical in the design of reservoirs for flood risk management. The critical durations will be a function of the degree of flood risk management selected and of the release rate or maximum rate of flow at the key downstream control point.

(1) The critical duration is increased as the proposed degree of risk reduction is increased and as the proposed rates of controlled flows at key damage centers are reduced. If this critical duration corresponds to the duration of a single rainstorm period or a single snowmelt event, the computation of hypothetical floods from rainfall and snowmelt can constitute the principle hydrologic design element.

(2) If the critical duration is much longer than a single rainstorm period or a single snowmelt event, hypothetical floods and sequences of hypothetical floods computed from rainfall or snowmelt become less dependable as guides to design. It then is necessary to base the reservoir design primarily on the frequency of observed runoff volumes for long durations. Even when this is done, it will be advisable to construct a typical hydrograph that corresponds to runoff volumes for the critical duration and that reasonably characterizes hydrographs at the location, to examine the operation of the proposed project under realistic conditions.

e. When hypothetical floods are selected, they must be routed through the proposed reservoir under the operation rules that would be specified for that particular design. In effect, a simulation study of the proposed project and operation scheme would be conducted for each flood. It is also wise to simulate the operation for major floods of historical record to ensure that some peculiar feature of a particular flood does not upset the plan of operation. With present software, it is relatively inexpensive to perform a complete period of record simulation once the flood control storage is set.

10-2. Regulated Release Rates.

a. Flood Risk Management Purposes. For flood risk management purposes reservoirs must store only the water that cannot be released without causing damage downstream. If more water can be released during a flood, less water needs to be stored. Thus, less storage space needs to be planned for flood risk management. Because reservoir space is costly and usually in high demand for other purposes, good flood control practice consists of releasing water whenever necessary at the highest practical rates so that a minimum amount of space need be reserved for flood risk management. As these rates increase, it becomes costly also to improve downstream channels and to provide adequate reservoir outlets, so there is an economic balance between release rates and storage capacity for flood risk management. In general, it is economical to use the full non-damage capacity of downstream channels, and it may pay to provide some additional channel or levee improvements downstream. However, full channel capacity may not be available, so analyses should consider the impact of reduced capacity.

b. Channel Capacities and Nondamaging Discharges. Channel capacities should be evaluated by examining water surface profile data from actual flood events whenever possible. It will typically be found that, under natural channel conditions, floods that occur more frequently than once in 2 years are not seriously damaging while larger floods are damaging. This is not always the case as the frequency of flow that is damaging depends on the channel, the basin, and the history of actual flows. Discharges that exceed channel capacity are not always damaging discharges and these nondamaging discharges may at times be released from a dam. Likewise, the channel capacity depends on the geomorphology of the site and may be a flow larger or smaller than the 50% annual change exceedance (2-year) frequency flow.

c. **Minor versus Major Damaging Releases.** In some cases, it is most economical to sustain minor damage by releasing flows above nondamaging stages to accommodate major floods and thereby decrease the risk to the more important potential damage areas from flooding. In such situations, a stepped-release schedule designed to protect all areas against frequent minor floods, with provision to increase releases after a specified reservoir stage is reached, might be considered. However, such a plan has serious drawbacks in practice because flood risk management of the minor damage areas results in greater improvements in those areas; and it soon becomes highly objectionable, if not almost impossible, to make the larger releases when they are required for flood risk management of major damage areas. In any case, it is necessary to make sure that the minor damage areas are not flooded more frequently or severely with the project (e.g., dam or levee) than they would have been without the project in place.

d. **Maintenance and Zoning.** It is important on all streams in developed areas to provide for proper maintenance of channel capacity and zoning of the floodplain where appropriate. This is vital where upstream reservoirs are operated for flood risk management because proper reservoir management depends as much on the ability to release water without damage as it does on the ability to store water. Minor inadequacies in channel capacity can lead to the loss of reservoir management options and result in major flooding. The maintenance of these channel corridors is particularly important because the reduction in flooding following dam construction often increases efforts of land developers to develop the floodplain.

e. **Forecasted Runoff.** When a reservoir is located some distance upstream from a damage center, runoff between the reservoir and the damage area must be considered. Runoff must be forecasted, a possible forecast error added, and the resulting quantities subtracted from project channel capacity to determine permissible release rates considering attenuation when routing the release from the dam to the damage center. The contribution of flow from the intermediate drainage area that enters the channel between the dam and the damage area must also be considered.

f. **Delaying Flood Releases.** Experience in the flood control operation of reservoirs has demonstrated that the actual operation does not make 100% use of downstream channel capacities. Due to many contributing factors, average outflows during floods are less than maximum permissible values. It is usually wise to approach maximum release rates with caution as changes in channel capacity since the last flood may have occurred and impact downstream capacity. It may be necessary to delay flood releases to permit removal of equipment, cattle, etc., from areas that would be flooded. Releases might be curtailed temporarily to permit emergency repairs to canals, bridges, and other structures downstream. If levees fail, releases might be reduced to hasten the drainage of flooded areas. Release can also be reduced to facilitate rescue operations. These and various other conditions result in reduced operation efficiency of the system during floods. To account for this, less non-damage flow capacity than actually exists (often about 80%) is assumed for design studies. It is important, however, that every effort be made in actual operation to reach full non-damage releases to attain maximum flood control benefits.

g. Gradually Increasing and Decreasing Releases. During flood operations, reservoir releases must be increased and decreased gradually to prevent damage and undue hardship downstream. Gradually increasing releases will usually permit an orderly evacuation of people, livestock, and equipment from the river areas downstream. If releases are curtailed too rapidly, there is some danger that the saturated riverbanks will slough and result in the loss of valuable land or damage to levees.

10-3. Flood-Volume Frequencies.

a. Critical Durations. Flood-volume frequency studies derive frequency curves of annual maximum volumes for several specified durations that might be critical in project design. Critical durations range from a few hours in the case of regulating “cloudburst” floods to a few months where large storage and very low release rates prevail. The annual maximum volumes for a specific duration are usually expressed as average rates of flow for that duration. It is essential that these flows represent a uniform condition of development for the entire period of observation, preferably unregulated conditions. Procedures for computing the individual frequency curves are discussed briefly in Chapter 6 herein and are described in detail in EM 1110-2-1415, Hydrologic Frequency Analysis.

b. Flood Control Space Requirement. Determination of the flood control space needed to provide a selected degree of flood risk management is based on detailed hydrograph analysis, but a general evaluation can be made as illustrated in Figure 10-1. The curve of runoff versus duration is obtained from frequency studies of runoff volumes or from SPF studies at the location. The tangent line represents a uniform flow equal to the project release capacity (reduced by an appropriate contingency factor). The intercept represents the space required for control of the flood.

(1) Figure 10-1 demonstrates that a reservoir capable of storing 155,000 units of water and releasing 30,000 units per day can control 100-year runoff for any duration, and that the critical duration (period of increasing storage) is about 5 days.

(2) The volume-duration curve is made for each damage area and should include more than 100 percent of the local uncontrolled runoff downstream from the reservoir and above the control point. This allows for errors in forecasts that are reflected in reduced project releases. If the local runoff appreciably exceeds nondamaging flow capacity at the damage centers, the volume over and above the flow capacity is damaging water that cannot be stored in the project reservoir.

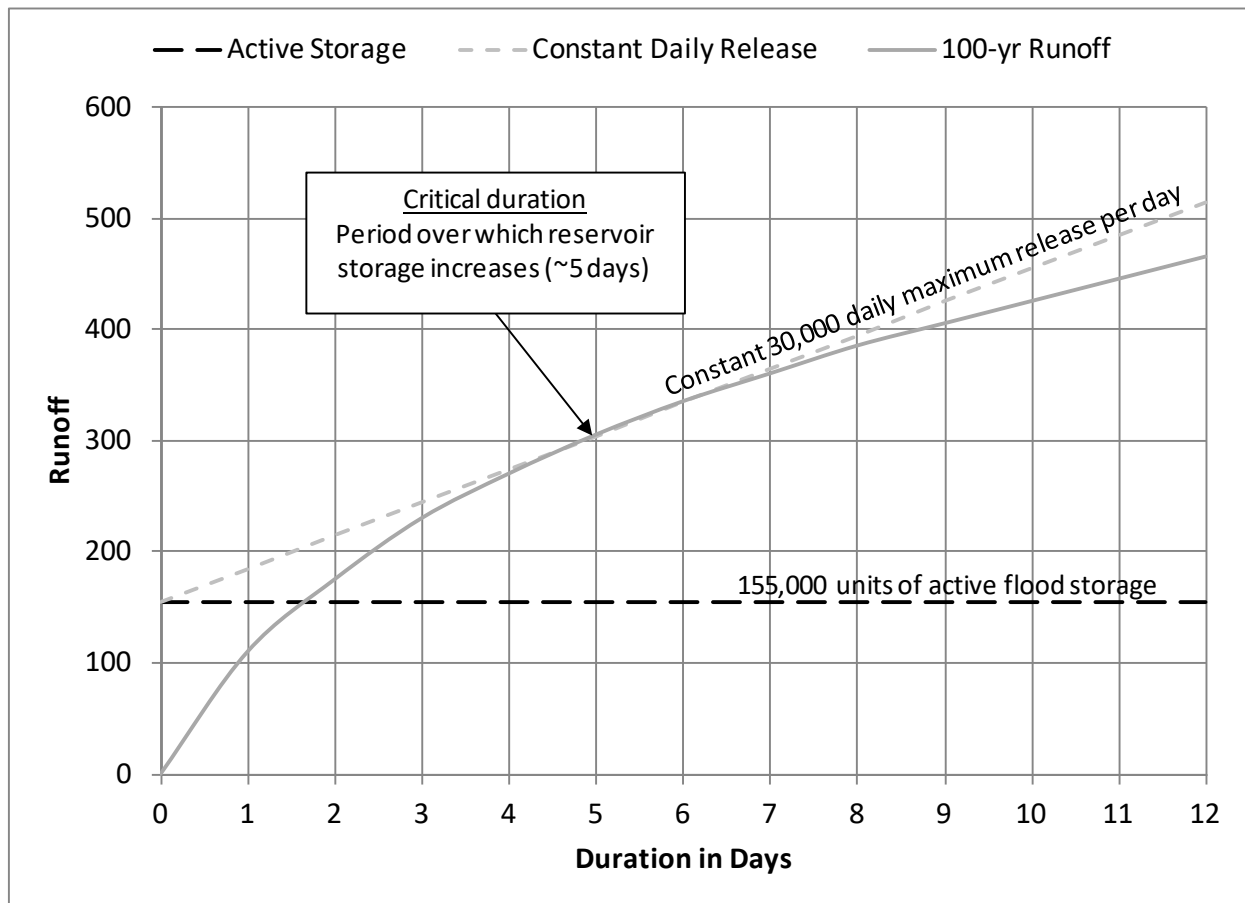


Figure 10-1. Flood Control Space Requirement.

10-4. Modeling Hypothetical Floods.

a. **Two Classes.** Two classes of hypothetical floods are important in the design of reservoirs for flood risk management. One is a balanced flood that corresponds to a specified frequency of occurrence; the other is a flood that represents a maximum potential for the location, such as the SPF or PMF. ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs, sets forth hydrologic engineering requirements for selecting and accommodating IDF's for dams and reservoirs.

b. **Specified Frequencies.** A hypothetical flood corresponding to a specified frequency should contain runoff volumes for all pertinent durations corresponding to that specified frequency. The derivation of frequency curves is discussed in the preceding section. A balanced flood hydrograph is constructed by selecting a typical hydrograph pattern and adjusting the ordinates so that the maximum volumes for each selected duration correspond to the volumes for that duration at the specified frequency.

c. **Longer Duration Floods.** Where flood durations longer than the typical single-flood duration are important in the design, a sequence of flood hydrographs spaced reasonably in time should be used as a pattern flood. To represent average natural sequences of flood events, the largest portions of the pattern flood should ordinarily occur at or somewhat later than the midpoint of the entire pattern, because rainfall sequences are fairly random but ground conditions become increasingly wet and conducive to larger runoff as any flood sequence continues.

d. **Maximum Flood Potential.** The PMF and the SPF are the two types of hypothetical floods that are important in the design of reservoirs because they represent maximum flood potential. The PMF, which is the largest flood that is reasonably possible at the location, is ordinarily the design flood for the spillway of a structure where loss of life or major property damage would occur in the event of project failure. The SPF, which represents the largest flood reasonably characteristic for the region of the reservoir site, is a flood of considerably lesser magnitude. It represents a high degree of design for projects protecting major urban and industrial areas. These floods can result from heavy rainfall or from snowmelt in combination with some rainfall.

e. **Computing Hydrographs.** SPF and PMF hydrographs are computed from the storm hydrographs by unit hydrograph procedures or other transform methods such as kinematic-wave. In the case of the SPF, ground conditions that are reasonably conducive to heavy runoff are used. In the case of the PMF, the most severe ground conditions that are reasonably consistent with storm magnitudes are used. A general description of these analyses is provided in Chapter 7 of this manual. Detailed methods for performing these computations are described in EM 1110-2-1417, Flood-Runoff Analysis. The software package HEC-HMS, the Hydrologic Modeling System, contains routines for computing floods from rainfall and snowmelt and also contains SPF criteria for the eastern United States.

10-5. Operation Constraints and Criteria.

a. **General.** As stated earlier, it is important that required flood releases be at maximum rates that are determined by downstream nondamaging discharge capacities, the mitigation of impacts to downstream communities, and streamflow and meteorological conditions upstream. This means that the reservoir outlets should be designed to permit releases at maximum rates at all reservoir levels within the flood control space. In some cases where controlled releases are very high, such an outlet design is not economical, and releases at lower stages might be restricted because of limited outlet capacity. This constraint, of course, should be taken into account during design studies.

b. **Downstream Damage Centers.** As discussed previously, local runoff below the reservoir and above the damage center must be considered when determining reservoir releases. This will ordinarily require some forecasting of the local runoff and, consequently, some estimate of the forecast uncertainty. The permissible release at any time is determined by adding a safe error

allowance to the forecasted local inflow and subtracting this sum from the nondamaging flow capacity.

c. **Rate of Change of Release.** The rate of change of release must be restricted to the maximum changes that will not cause critical conditions downstream. These rates of change of release should be less than the rates of change of flow that occurred before the reservoir was built. After the main flood has passed, water stored in the flood control space of the reservoir must be released at maximum rates-of-release until the desired amount of water is released from the pool. The rate-of-release should be decreased gradually toward the end of the release period. This reduction in rate-of-release must be started while a considerable amount of flood waters remain in the reservoir so that water retained for other purposes is not inadvertently released. Schedules for this operation are discussed in Part 3.

10-6. Storage Capacity Determinations.

a. **Determining required storage capacity.** The storage capacity required to regulate a specific flood (represented by a flood hydrograph at the dam) to a specified control discharge immediately downstream of the dam is determined by routing the hydrograph through a hypothetical reservoir with unlimited storage capacity and noting the maximum storage. However, there are many special practical considerations that complicate this process. To be realistic, release rates should not be changed suddenly meaning the routing should conform to criteria that specify the maximum rate of change of release. Another complication is that reservoir outlet capacities may not be adequate to supply full regulated releases with the low head associated with low reservoir pool elevations. If this is the case, a preliminary reservoir design is required to define the relation of storage capacity to outlet capacity. In the more common cases, where damage centers exist at some distance downstream of the reservoir, the storage requirement for a specified flood is determined by successive approximations in which the hypothetical reservoir is operated to regulate flows at each damage center and allow for local inflow and some forecasting error.

b. **Detailed Operational Study.** Although there are approximate methods for estimating storage capacity, it is essential that the final project design be tested by a detailed operational study. The analyses are based on actual outlet capacities and realistic assumptions for limiting rates-of-release change, forecast errors, and operational contingencies. Analysis must include various combinations of reservoir inflow and local flow that can produce a specific downstream flood event. It is also important to route both the largest floods of record and synthetic hypothetical floods through the project to ensure the project design is adequate and that the project provides the degree of flood risk management for which it was designed.

c. **Seasonal Distribution of Storage Requirements.** Where some of the flood control space will be made available for other uses during the dry season, a seasonal distribution of flood control storage requirement should be developed. The most direct approach to this involves the gen-

eration of runoff frequency curves for each month of the year. The average frequency of the design flood during the rainy-season months can be used to select flood magnitudes for other months. These then serve as a basis for determining the amount of space that must be made available during the other months.

d. **Software Packages.** Sequential routing in planning, design, and operation of flood control reservoirs can be accomplished with the Hydrologic Engineering Center Reservoir System Simulation (HEC-ResSim, HEC 2013) software package.

10-7. **Spillways.** Spillways are provided to release floodwater that normally cannot be passed by other outlet works. The spillway is sized to ensure the passage of major floods without overtopping the dam. A general discussion of spillways is provided in Section 4-2 of EM 1110-2-3600, Management of Water Control Systems. EM 1110-2-1603, Hydraulic Design of Spillways, describes the technical aspects of design for the hydraulic features of spillways and ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs, sets forth requirements for selecting and accommodating IDFs.

a. **Inflow Design Flood.** Guidance for developing the IDF for a reservoir is provided by ER 1110-8-2. The IDF is selected based on the dam safety standard assigned to the dam and is used to balance the spillway design capacity and the top-of-dam elevation. Other methods of estimating extreme flood magnitudes, such as flood-frequency analysis, are not reliable due to limited observations. Usually, the spillways for major dams, whose failure might constitute a major disaster, are designed to pass the PMF without a major failure. However, the spillways for many small dams are designed for smaller floods based on a risk-informed analysis used to establish the base-safety condition. This is discussed farther in ER 1110-8-2.

b. **Hydrologic Design.** The hydrologic design of a spillway is determined by estimating a design and then testing it by routing the spillway IDF. Experience has demonstrated that an unusual sequence of floods can result in filling all or a major portion of the flood control storage in a reservoir immediately before the beginning of the IDF. ER 1110-8-2 accounts for this with the consideration of two antecedent pool elevations, one pool set to the top of the flood control storage and the second considering an antecedent inflow $\frac{1}{2}$ of the full IDF. Refer to ER 1110-8-2 for information on required antecedent pool elevations and IDF routing. In the case of ungated spillways, it is possible that the outlets of the dam will be closed gradually as the spillway goes into operation, to delay damaging releases as long as possible and possibly to prevent them. However, if spillway flows continue to increase, it may be necessary to reopen the outlets. In doing so, care should be exercised to prevent releases from exceeding maximum inflow quantities. The exact manner in which outlets will be operated should be specified so that the spillway design will be adequate under conditions that will actually prevail after project construction. Consideration should be given to the possibility that some outlets or turbines might be out of service during flood periods. See ER 1110-8-2 for specific guidance.

c. Large Spillway Gates. The operation of large spillway gates can be extremely hazardous, since opening them inadvertently might cause major flooding downstream. Their operation should be controlled by rigid water management guidance. In particular, the opening of the gates during floods should be scheduled on the basis of inflows and reservoir storage so that the lake level will continue to rise as the gates are opened. This will ensure that inflow exceeds outflow as outflows are increased. The adequacy of a spillway to pass the IDF is tested for gated spillways in the same manner as for ungated spillways described above. Methods for developing spillway gate operation guidance are described in Chapter 14.

d. Preventing Overtopping. To ensure that the project is adequately sized to prevent the structure from overtopping from the design event, freeboard is added to the dam above the maximum pool water surface elevation. Freeboard determination is discussed in ER 1110-8-2, Inflow Design for Dams and Reservoirs. Freeboard includes an allowance for wind setup and wave action, which are discussed in Chapter 15.

e. Spillway Types. While the spillway is primarily intended to protect the structure from overtopping from the design event, the fact that it can cause some water to be stored above ordinary full pool level (surcharge storage) is of some consequence in reducing downstream flooding. Narrow, ungated spillways require higher dams and can, therefore, be highly effective in partially regulating floods that exceed project design magnitude, whereas wide spillways and gated spillways are less effective for regulating floods exceeding design magnitude. Where rare floods can cause substantial damage downstream, the selection of spillway type and characteristics can appreciably influence the benefits that are obtained for flood control. Accordingly, it is not necessarily the least costly spillway that yields the most economical plan of development. In evaluating flood control benefits, computing frequency curves for regulated conditions should be based on spillway characteristics and operation criteria as well as on other project features.

10-8. Flood Risk Management Formulation.

a. Objectives. The objectives of flood risk management plan formulation are to identify the individual reservoir components, determine the size and nature of each, determine the order in which the system components should be implemented, and develop and display the information required to justify the decisions and thus secure system implementation. Section 4-10 of this EM describes several formulation strategies.

b. Criteria. Criteria for system formulation are needed to distinguish the best system. The definition of “best” is crucial. It should be recognized that simply aggregating the most attractive individual components into a system, while assuring physical compatibility, could result in the inefficient use of resources because of system effects, data uncertainty, and the possibility that all components may not be implemented. It is proposed that the best system be considered to be as follows:

- (1) The system that is the most cost effective (greatest net benefits).

(2) The system that includes the best flood risk management features while preserving flexibility for modification of components at future dates.

(3) The system that could be implemented at a number of stages, if staging is necessary, such that each stage could stand on its own merits (be of social value) if no more components were to be added.

c. General Guidance. General guidance for formulation criteria are contained in ER 1105-2-100, Planning Guidance Notebook. The criterion of economic efficiency from the national viewpoint has been interpreted to require that each component in a system should be incrementally justified, that is, each component addition to a system should add to the value (net benefits) of the total system. The environmental quality criteria can be viewed as favoring alternatives that can be structured to minimize adverse environmental impacts and provide opportunities for mitigation measures. Additional criteria that are not as formally stated as U.S. national policy are important in decisions among alternatives. A formulated flood risk reduction system must draw sufficient support from responsible authorities to be implemented. In addition, flood risk reduction systems should be formulated so that the risk of death or injury from floods following project implementation is greatly reduced over the preproject condition, including for events that exceed the project's design.

d. Environmental and Other Assessments. Environmental quality analysis and social/political/institutional analyses related to implementation are generally not quantifiable in a flood risk reduction study. As a consequence, these criteria must guide the formulation studies but, as yet, probably cannot directly contribute in a structured formulation strategy. Of these criteria, only the national economic efficiency and minimum performance standard have generally accepted methods available for their rigorous inclusion in formulation studies. In the discussions that follow, focus is of necessity upon the economic criteria with acceptable performance as a constraint, with the assumption that the remaining criteria will be incorporated when the formulation strategy has narrowed the range of alternatives to a limited number for which the environmental and other assessments can be performed.

e. Degrees of Uncertainty. Information used in system formulation will have varying degrees of uncertainty. The hydrology will be better defined near gauging stations than it is in remote areas, and the longer the period of streamflow or reservoir regulation record, the less the uncertainty in analytical results. In addition, the accuracy of economic data, both costs and value, existing or projected, is generally lower than the more physically based data. Also, since conditions change over time, the data must be continuously updated at each decision point. The practical accommodation of information uncertainty is by limited sensitivity analysis and continuing reappraisal as each component of a system is studied for implementation.

f. Sensitivity Analysis. The objective of sensitivity analysis is to identify either critical elements of data or particularly sensitive reservoir system components so that further studies can

be directed toward firming up the uncertain elements or making adjustments in system formulation to reduce the uncertainty. Particular attention must be paid to the selection or development of system hydrology because combinations of historic and synthetic floods are typically used to evaluate reservoir flood reduction performance (i.e., to develop regulated conditions frequency relations at damage index stations). Problems arise when a complex system of multiple reservoirs is evaluated above common damage centers. The problem increases with the size and complexity of the basin because the storm magnitudes and locations can favor one reservoir location over another. A large number of storm centerings could yield similar flows at a particular control point. Because of this, the contribution of a specific system component to reduced flooding at a downstream location is uncertain and dependent upon storm centering. This makes the selection or development of representative centerings crucial if all upstream components are to be evaluated on a comparable basis.

g. **Desired Evaluation.** The desired evaluation for regulated conditions is the expected or average condition so that economic calculations are valid. The representative hydrograph procedure is where several proportions (ratios of one or more historic or synthetic events used to represent system hydrology) are compatible with the simulation technique used, but care must be taken to reasonably accommodate the storm centering uncertainty. Testing the sensitivity of the expected annual damage (EAD) to the system hydrology (event centering) is appropriate and necessary. Even if all historical floods of record are used, there still may be some bias in computing EADs if most historical floods were, by chance, centered over a certain part of the basin and not over others. For instance, one reservoir site may have experienced several severe historical floods, while another site immediately adjacent to the area may, due to chance, not have had any severe floods.

10-9. General Study Procedure. After various alternative locations are selected for a reservoir site to protect one or more damage centers, the following steps are suggested for conducting the required hydrologic engineering studies:

a. Create a detailed map of the region showing the locations of the damage areas, of proposed reservoir sites, and of all pertinent precipitation, snowpack, and stream-gauging stations. Prepare a larger scale map of the drainage basin tributary to the most downstream damage location. Locate damage centers, project sites, pertinent hydrologic measurement stations, and drainage boundaries above each damage center, project site, and stream-gauging station. Measure all pertinent tributary areas.

b. Establish stage-discharge relations for each damage reach, relating the stages for each reach to a selected index location in that reach; procedures for doing this are described in the Flood Damage Reduction Analysis (HEC-FDA) software package (HEC 2018b). Where local flood risk management works are considered part of an overall plan of improvement, establish the stage-discharge relation for each plan of local flood risk management.

c. Obtain area- and storage-elevation curves for each reservoir site. Select alternative reservoir capacities as appropriate for each site, outlet and spillway rating curves for each reservoir, and develop a plan of flood control operation for each reservoir. Determine maximum regulated flows for each damage center.

d. Estimate the maximum critical duration of runoff for the plans of improvement. Consider the relation of regulated flows at damage centers to unregulated flood hydrographs of design magnitude at those damage centers. Prepare frequency curves of unregulated peak flows and volumes for various representative durations for each damage center index location and for each reservoir site as described for peak flows in Chapter 6. If seasonal variation of flood control space is considered, frequency curves should be developed for each season.

e. Flood control simulation can be continuous simulation of a period of record or a representative storm flood analysis. If flooding can occur during any time of the year, the complete sequential analysis might be favored. However, if there is a separable flood season, e.g., in the western states, then the representative storm approach may be sufficient. For the storm approach, develop data for historical floods with storm centerings throughout the basin and use several proportions of those floods to obtain flows at the damage centers representing the full range of the flow-frequency-damage relationship for base conditions and for regulated conditions. Also, develop synthetic events that have consistency in volumes of runoff and peak flows and are reasonably representative regarding upstream contributions to downstream flows.

f. Perform sequential analysis with the developed hydrology. The period of record simulation provides simulated regulated flow, which can be analyzed directly to develop flow-frequency relations. The representative flood approach requires an assumption that the regulated-flow frequency is the same as the natural-flow frequency. Frequency curves of regulated conditions at each damage center can then be derived from frequency curves of unregulated flows simply by assuming that a given ratio of the base flood will have the same recurrence frequency whether it is modified by regulatory structures or not. This assumption is valid as long as larger unregulated floods always correspond to the larger regulated flows.

g. Derive a flow-frequency and stage-discharge curve for the index station at each damage center for unregulated conditions for each plan of improvement as described in Chapters 6 and 8. These curves can be used to determine the average annual damage for both the unregulated conditions and for each system alternative. Therefore, they provide the basis for project selection.

h. Develop a PMF for each reservoir site, using procedures described in Chapter 7. These are used as a possible basis for spillway design. Route the PMF through each reservoir, assuming reasonably adverse conditions for initial storage and available outlet capacity. See ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs, for guidance.

CHAPTER 11

Conservation Storage

11-1. General Considerations.

a. **Purposes.** Water stored in the conservation pool can serve many purposes, including water supply, navigation, fish and wildlife, and hydroelectric power. The water requirements for accomplishing these purposes are discussed in this chapter along with water quality considerations. Methods for estimating the conservation storage, or yield, are presented in Chapter 12.

b. **Operational Policy.** The general operational policy for conservation storage in a reservoir is to conserve available supplies and to release only when supplemental flow is needed to meet downstream requirements. Water stored in the conservation pool also provides benefits within the pool, such as lake recreation and fish and wildlife habitat.

c. **Changing Hydrology.** The hydrology of the inundated area and its immediate surroundings is changed in a number of ways when a reservoir is filled. The effects of inflows at the perimeter of the reservoir are translated rapidly to the reservoir outlet, thus, effectively speeding the flow of water through the reservoir. Also, large amounts of energy are stored and must be dissipated or used at the outlet. The reservoir loses water by evaporation, and this usually exceeds preproject evapotranspiration losses from the lake area. Siltation usually seals the reservoir bottom, but rising and falling water levels may alter the pattern of groundwater storage due to movement into and out of the surrounding reservoir banks. At high stages, water may seep from the reservoir through permeable soils into neighboring catchment areas and so be lost to the area of origin. Finally, sedimentation takes place in the reservoir and scour occurs downstream.

d. **Storage Allocation.** The joint use of storage for more than one purpose can create problems in storage allocation. While retained in reservoir storage, water may provide benefits to recreation, fish, wildlife, hydropower, and aesthetics. When this water is properly discharged from the reservoir, similar overlapping benefits can be achieved downstream. Other benefits that can be derived from the reservoir are those covered in this chapter, including M&I water supply, agricultural water supply, and navigation.

e. **Storage Capacity for Sediment.** Additional sediment storage capacity is designed into many USACE reservoirs. This helps protect the storage-yield capacity of the reservoir throughout its economic life by providing sediment storage volume that would otherwise reduce the usable reservoir storage volume. Historically, many USACE project designs assumed a simplified view of sediment processes and allocated all sediment deposition within the inactive reservoir pool. However, sediment deposition affects all levels of reservoir storage because deposition occurs in all pools and not just the inactive storage pool. This accumulation within different zones of the reservoir can impact reservoir operations depending upon the location of the accumulation. The volume of sediment storage allocated within the reservoir is typically a function of the basin

sediment yield, sediment delivery ration, reservoir trap efficiency, and the economic life of the project. Trap efficiency of the reservoir is evaluated and the distribution of this estimated volume of sediment is determined, using methods described in EM 1110-2-4000, Sedimentation Investigations of Rivers and Reservoirs. Sediment surveys within the reservoir during actual operation will establish the reliability of these estimates. Storage allocation levels may then be revised if the sediment surveys show a significant difference between what was projected and what was measured. More complete descriptions of the techniques used to determine reservoir sedimentation are presented in EM 1110-2-4000.

f. **Minimum Pool.** A minimum pool at the bottom of active conservation storage is usually established to identify the lower limit of normal reservoir drawdown. The inactive storage below the minimum pool level is used for sediment deposition. Inactive storage is dead storage and cannot be relied upon for emergency water supply. Further uses of inactive storage can be addressed in a drought contingency plan.

g. **Reservoir Outlets.** Reservoir outlets must be located low enough to release water with the reservoir stage near minimum pool. These outlets can discharge directly into an aqueduct or into the river. In the latter case, a diversion dam may be required downstream at the main canal intake.

h. **Computing Storage Capacity.** The most important physical characteristic of a reservoir is storage capacity because the primary function of a reservoir is to provide storage. Storage capacities of natural sites for proposed reservoirs must be determined from topographic surveys or from the processing of DEMs in GIS.

(1) Reservoir storage capacity was once computed by planimetry the area enclosed within each elevation contour throughout the full range of elevations within the reservoir site and using the average-end-area method or the conic method to determine a volume. Advances in GIS now allow the calculation of volume directly from DEMs using the 3D Analyst tool Surface Volume. These calculations can be checked by determining areas enclosed within each contour and using the average-end-area method or the conic method to calculate the volume. The Surface Volume tool only works for the full depth of the reservoir site when the DEM being processed does not currently have a reservoir pool.

(2) DEM resolutions generally vary from 2 arc-second (about 60 meters) to 1/9-arc-second (about 3 meters) or smaller if LiDAR data are available. The higher the resolution of the data, the better the volume estimate. The USGS National Map Viewer and Download Platform (USGS 2013b) provides NEDs derived from 10- to 30-meters DEMs for most of the United States. In the absence of adequate DEM and topographic data, cross sections of the reservoir area are sometimes surveyed, and the capacity is computed from these vertical cross sections by using the formula for the volume of a prism.

11-2. Water Supply.

a. Introduction. Water supply for any purpose is usually obtained from groundwater or from surface waters. Groundwater yields and the methods currently in use are covered in Physical and Chemical Hydrogeology (Domenico and Schwartz 1990). This discussion is limited to surface water supplies for low-flow water management or for diversion to demand areas.

(1) In some cases, water supply from surface waters involves only the withdrawal of water as needed from a nearby stream. However, this source can be unreliable because streamflow can be highly variable, and the desired amount might not always be available. An essential requirement of most water supply projects is that the supply be available on a dependable basis. Reservoirs play a major role in fulfilling this requirement. Whatever the ultimate use of water, the main function of a reservoir is to stabilize the flow of water, either by regulating a varying supply in a natural stream or by satisfying a varying demand by the ultimate consumer. Usually, some overall loss of water occurs in this process.

(2) A number of factors should be considered when determining the location of a proposed reservoir to satisfy water needs. The dam should be located so that adequate capacity can be obtained, social and environmental effects of the project will be acceptable, sediment deposition in the reservoir and scour below the dam will be tolerable, the quality of water in the reservoir will be commensurate with the ultimate use, and the cost of storing and transporting the water to the desired location is acceptable. It is virtually impossible to locate a reservoir site with completely ideal characteristics and many of the factors listed previously will be competitive. The factors can be used, however, as general guidelines for evaluating prospective reservoir sites.

(3) The basic hydrologic problem in the planning and design of reservoirs for water supply is determining how much water a specified reservoir capacity will yield. Yield is the amount of water that can be supplied from the reservoir to a specified location and in a specified time pattern. Firm yield is the largest consistent flow rate (demand) that can be provided throughout a period of historic streamflow. Use of the firm yield brings the stored water volume exactly to zero once during the historic period of record during what is designated the critical period for that firm yield and storage capacity. The critical period is that period in a sequential record that requires the largest volume from storage to provide a specified yield. Critical periods are the driest periods of record where the inflow does not satisfy the demand and reservoir storage is required. Firm yield and safe yield may not be equivalent. Safe yield typically includes a factor of safety over the firm yield to account for possible events outside the period of streamflow record. Chapter 12 describes procedures for yield determination.

b. M&I Water Use. When designing or modifying existing conservation storage space for M&I water use, USACE engineers and planners must work closely with the state and local agencies that will make use of the storage. Local water systems may be divided into source, transmission, treatment and distribution components. For some systems the reservoir may represent the only source of water, while in others the reservoir may be supplemented by other sources. The characteristics, capacity and reliability of other sources are important factors in determining

the reliability of the overall water supply. While the USACE role in water supply is typically limited to the quantity of source water available as reservoir yield, environmental conditions in the reservoir may have impacts downstream through transmission, treatment and distribution components of the system, ultimately affecting the end users. More detailed information on water supply system planning is available through textbooks and industry publications such as the American Water Works Association's (AWWA) Manual of Practice M-50, Water Resources Planning (AWWA 2007).

(1) Water needs are often developed by state and local agencies using forecasts of future population and economic conditions. Water needs are very specific to individual locations depending on climate, population, the local economy and other factors. Technical guidance on forecasting future water needs is available in textbooks and industry publications such as Forecasting Urban Water Demand (AWWA 2008). Measures to reduce future needs and conserve water during shortages are essential in planning for M&I water supply.

(2) In any system, M&I water use also varies on annual, seasonal, daily and hourly scales. Hourly variations in water use are typically met by storage in the local distribution system, such as elevated water storage tanks. At the other end of the time scale, average annual M&I water use is typically used as the basis for computing how much reservoir storage is needed to dependably supply water through critical drought periods. However, variations in water use in the daily to seasonal range may impact reservoir operations and should be evaluated. Historical data and forecasts of future trends can be used to estimate seasonal, monthly and daily peaking factors to apply to the base annual average water withdrawals when simulating reservoir operations at comparable time steps. M&I water needs are typically highest in summer months and in many regions a seasonal guide curve is useful in meeting these needs.

(3) Environmental conditions in reservoirs used as the source for M&I water supply can have a major impact on the costs to supply water to end users as well as public health. Within a local water system, different sectors of water use may have different requirements for the quality of treated water. At a minimum, water systems must meet Federal and state requirements for public health. The physical, chemical and biological condition of the source water determines the cost of treatment to meet the required standards for end users. Extreme events such as harmful algal blooms may render the reservoir unusable for a period of time. The presence of invasive species such as zebra mussels may require expensive operational and maintenance measures throughout the water supply system. Different water control plans may increase or decrease the potential for specific physical, chemical and biological conditions and resulting water supply impacts. Changes in operations may exacerbate or mitigate conditions that impact water supply systems. The physical location and configuration of water supply intakes may also be a factor in determining impacts and potential responses.

11-3. Navigation and Low-Flow Augmentation.

a. Objective. The objective of reservoir design for navigation is to supplement flows at one or more points downstream from the reservoir. These flows aid in maintaining the necessary depth of water and alleviate silting problems in the navigable channel. Low-flow augmentation serves a number of purposes including recreation, fish and wildlife, ice control, pollution abatement, and run-of-river power projects. Under certain conditions, low-flow augmentation provides water for the other purposes discussed in this chapter. For instance, if the intake for a M&I water supply is at some point downstream of the reservoir, the objective may be to supplement low flows at that point.

b. Criteria for Navigability. There are no absolute criteria for navigability and, in the final analysis, economic criteria control. The physical factors that affect the cost of waterborne transport are depth of channel, width and alignment of channel, locking time, current velocity, and terminal facilities. Commercial inland water transport is, for the most part, accomplished by barge tows consisting of 1 to 10 barges pushed by a shallow-draft tug. The cost of a trip between any two terminals is the sum of the fuel costs and wages, fixed charges, and other operating expenses depending on the time of transit. Reservoirs aid in reducing these costs by providing the proper depth of water in the navigation channel, or by providing a slack-water pool in lock-and-dam projects. Storage reservoirs can rarely be justified economically for navigation purposes alone and are usually planned as multipurpose projects. Improving navigation by using reservoirs is possible when flood flows can be stored for release during low-flow seasons.

c. Supplying Deficiencies without Waste. The ideal reservoir operation for navigation or low-flow augmentation would provide releases timed so as to supply the deficiencies in natural flow without waste. This is possible only if the reservoir is at the head of a relatively short control reach. As the distance from the reservoir to the reach is increased, releases must be increased to allow for uncertainties in estimating intermediate runoff and for evaporation and seepage en-route to the reach to be served. Moreover, the releases must be made sufficiently far in advance of the need to allow for travel time to the reach, and in sufficient quantity so that after reduction by channel storage, the delivered flows are adequate. The water requirement for these releases is considerably greater than the difference between actual and required flows.

d. Climate. Climate can also affect reservoir operation for low-flow water management. Depending on the purpose to be served, the releases may be required only at certain times of the year or may vary from month to month. For pollution abatement, the important factors are the quality of the water to be supplemented, the quality of the water in the reservoir, and the quality standard to be attained. Also, the level of the intakes from which releases will be made can be a very sufficient factor in pollution abatement, since the quality can vary from one level to another in the reservoir. Long-term variations can occur due to increased contamination downstream of a reservoir. This should be considered in determining the required storage in the reservoir.

11-4. Fish and Wildlife.

a. Authority. Consideration of fish and wildlife and subsequent environmental purposes have increased over time under congressional legislation since around 1960. These considerations should be taken into account when planning and designing reservoir projects. Additionally, because many USACE reservoirs were built prior to that time, their management plans may not have taken into account more recent environmental objectives.

b. Water-Level Fluctuations. The seasonal fluctuations that occur at many flood control reservoirs and the daily fluctuations that occur with hydropower operation can result in the elimination of shoreline vegetation and an increase in shoreline erosion, water quality degradation, and loss of habitat for fish and wildlife. Adverse impacts of water-level fluctuations also include loss of shoreline shelter and physical disruption of spawning and nests.

c. Water-Level Management. Water-level management in fluctuating warm-water and cool-water reservoirs generally involves raising water levels during the spring to enhance spawning. Pool levels are lowered during the summer to permit re-growth of vegetation in the fluctuation zone. Fluctuations may be timed to benefit one or more target species; therefore, several variations in operation may be desirable. In the central United States, managers frequently recommend small increases in pool levels during the autumn for waterfowl management.

d. Fishery Management. Guidelines to meet downstream fishery management potentials are developed based on project water quality characteristics and water control capabilities. An understanding of the reservoir water quality regimes is critical for developing the water control criteria to meet the objectives. For example, temperature is often one of the major constraints of fishery management in the downstream reach, and water control managers must understand the temperature regime in the pool and downstream temperature requirements, as well as the capability of the project to achieve the balance required between the inflows and the releases. Releasing cold water downstream where fishery management objectives require warm water will be detrimental to the downstream fishery. Conversely, releasing warm water creates difficulty in maintaining a cold water fishery downstream.

e. Water Temperature Management. Water control activities can also impact water temperatures within the pool by changing the volume of water available for a particular layer. In some instances, cold water reserves may be necessary to maintain a downstream temperature objective in the late summer months; therefore, the availability of cold water must be maintained to meet this objective. For some projects, particularly in the southern United States, water control objectives include the maintenance of warm-water fisheries in the tail waters. In other instances, fishery management objectives may include the maintenance of a two-story fishery in a reservoir, with a warm-water fishery in the surface water, and a cold water fishery in the bottom waters. Such an objective challenges water control managers to regulate the project to maintain the desired temperature stratification while maintaining sufficient dissolved oxygen in the bottom

waters for the cold water fishery. Management to meet this objective requires an understanding of operational effects on seasonal patterns of thermal stratification, and the ability to anticipate thermal characteristics.

f. Minimum Releases. Minimum instantaneous flows can be beneficial for maintaining gravel beds downstream for species that require this habitat. However, dramatic changes in release volumes, such as those that result from flood control management, as well as hydropower, can be detrimental to downstream fisheries. Peaking hydropower operations can result in releases from near zero to very high magnitudes during operations at full capacity. Maintaining minimum releases and incorporating re-regulation structures are two of the options available to mitigate this problem.

g. Fishing versus Peak Power. In some instances, tail water fishing is at a maximum during summer weekends and holidays, and this is a time when power generation may be at a minimum and release near zero. Maintaining minimum releases during weekend daylight hours may improve recreational fishing, but may reduce the capability to meet peak power loads during the week because of lower water level (head) in the reservoir. In these instances, water control managers will be challenged to regulate the project with consideration of these two objectives.

h. Anadromous Fish. Water management for anadromous fish (e.g., salmon) is particularly important during certain periods of the year. Generally, upstream migration of adult anadromous fish begins in the spring of each year and continues through early fall, and downstream migration of juvenile fish occurs predominantly during the spring and summer months. The reduced water velocities through reservoirs, in comparison with preproject conditions, may greatly lengthen the travel time for juvenile fish to travel downstream through the impounded reach. In addition, storage for hydropower reduces the quantity of spill, and as a result, juvenile fish must pass through the turbines. The delay in travel time subjects the juvenile fish to greater exposure to birds and predator fish, and passage through the powerhouse turbines increases mortality. To improve juvenile survival, storage has been made available at some projects to augment river flows, and flows are diverted away from the turbine intakes and through tailraces where the fish are collected for transportation or released back into the river. Barges or tank trucks can be used to transport juveniles from the collector dams to release sites below the projects. Other USACE projects have been modified so the ice and trash spillways can be operated to provide juvenile fish passage.

i. Wildlife Habitat. Project management can influence wildlife habitat and management principally through water-level fluctuations. The beneficial aspects of periodic draw downs on wildlife habitat are well documented in wildlife literature. Draw downs as a wildlife management technique can, as examples, allow the natural and artificial re-vegetation of shallows for waterfowl, permit the installation and maintenance of artificial nesting structures, allow the control of vegetation species composition, and ensure mast tree survival in greentree reservoirs.

Wildlife benefits of managing water levels in natural areas include inhibiting the growth of undesirable and perennial plants, creating access and foraging opportunities for waterfowl in areas such as greentree reservoirs, and ensuring certain water levels in stands of vegetation to encourage waterfowl nesting and reproduction.

11-5. Hydroelectric Power.

a. General. The feasibility of hydroelectric development is dependent upon the need for electric power, the availability of a transmission system to take the power from the point of generation to the points of demand, and the availability of water from streamflow and storage to produce power for energy demands in the power market area. Also, the project's power operations must be coordinated with the operations for other project purposes to ensure that all purposes are properly served. Each of these factors must be investigated to ensure that the project is both feasible and desirable and to minimize the possibility that unforeseen conflicts will develop between power and other water uses during the project life.

(1) The ability of a project to supply power is measured in terms of two parameters: capacity and energy. Capacity, commonly measured in kilowatts (kW), is the maximum amount of power that a generating plant can deliver. Energy, measured in kilowatt-hours (kWh), is the amount of actual work done. Both parameters are important, and the operation of a hydroelectric project is sensitive to changes in the demand for either capacity or energy.

(2) Experience has indicated that it is very unlikely that power demands will remain unchanged during the project life. Furthermore, the relative importance of various other water uses can change during the project life, and there are often legal, institutional, social, or environmental factors that might affect the future use of water at a particular project. Consequently, the feasibility studies for a proposed project must not be limited to conditions that are only representative of the current time or the relatively near future. Instead, the studies must include considerations of future conditions that might create irreconcilable conflicts unless appropriate remedial measures are provided for during project formulation.

(3) This section presents general concepts for the hydrologic analyses associated with the planning, design, and operation of hydroelectric projects and systems. More detailed information is provided in EM 1110-2-1701, Hydropower. Other investigations that influence or affect the hydrologic studies will be discussed to the extent that their outcome must be understood by the hydrologic engineer.

b. Types of Hydroelectric Load. Power developments, for purposes of this discussion, are classified with respect to the type of load served or the type of site development proposed. The two categories related to the type of load served are baseload and peaking plants.

(1) Base Load. Baseload plants are projects that generate hydroelectric power to meet the baseload demand. The baseload demand is the demand that exists 100% of the time. The base load

can readily be seen in Figure 11-1 as the horizontal dashed line on a typical annual load duration curve. This curve displays the percent of time during a given year that a given capacity demand is equaled or exceeded. The area under this curve represents the total energy required to meet the load during the year. Usually, the baseload demand is met by thermal generating facilities. However, in cases where there is a relatively abundant supply of water that is available with a high degree of reliability and where fuel is relatively scarce, hydroelectric projects may be developed to meet the baseload demands. These projects would then operate at or near full capacity 24-hour per day for long periods of time. This type of development is not feasible where there is a large seasonal variation in streamflow unless the base flow is relatively high or unless there is a provision for a large volume of power storage in the project.

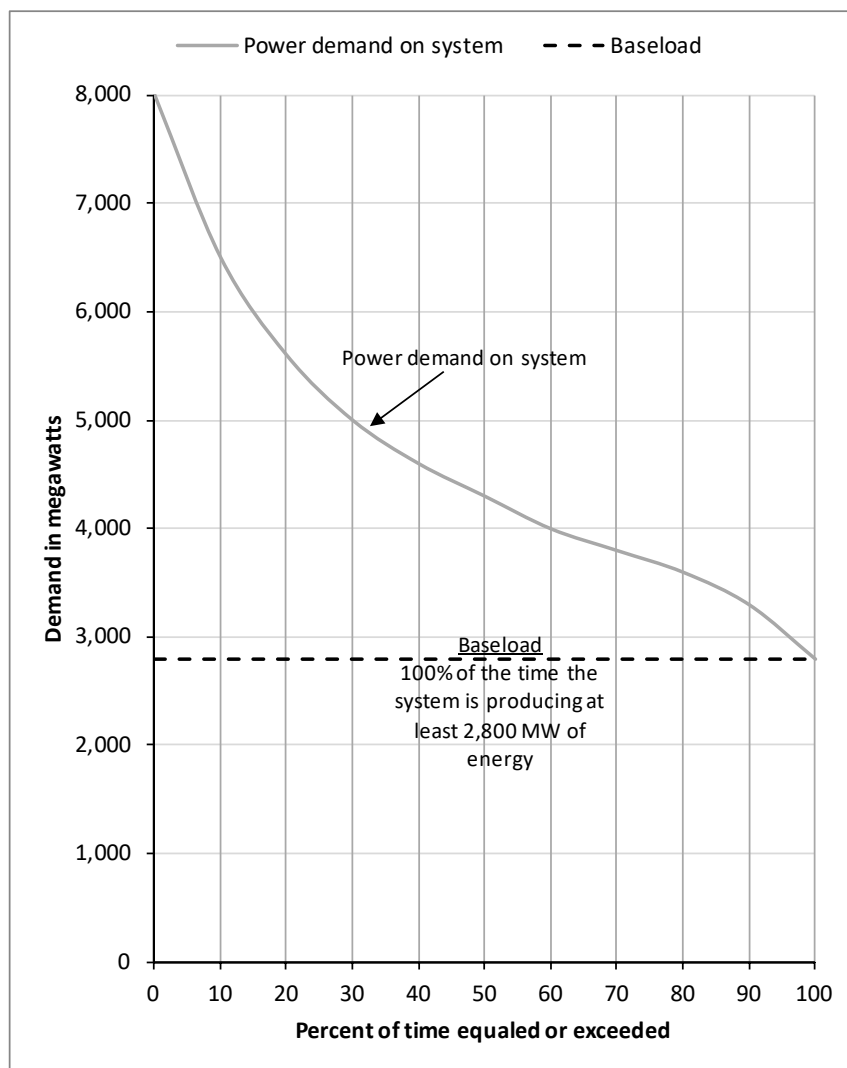


Figure 11-1. Typical Annual Load Duration Curve.

(2) Peaking Load. Peaking plants are projects that generate hydroelectric power to supplement baseload generation during periods of peak power demands. The peak power demands are the loads that exist primarily during the daylight hours. The time of occurrence and magnitude of peak power demands are shown on a load curve in Figure 11-2. This curve shows the time variation in power demands for a typical week. Depending upon the quantity of water available and the demand, a peaking plant may generate power from as much as 18 hours a day to as little as no generation at all, but it is usually 8 hours a day or less. Peaking plants must supply sufficient capacity to satisfy the peak capacity demands of a system and sufficient energy to make the capacity usable on the load. This means that energy or water should be sufficient to supply peaking support for as long and as often as the capacity is needed. In general, a peaking hydroelectric plant is desirable in a system that has thermal generation facilities to meet the baseload demands. The hydroelectric generating facilities are particularly adaptable to the peaking operation because their loading can be changed rapidly. Also, the factors that make seasonal variations in streamflow a major problem in baseload operation are usually quite easily overcome in a peaking plant if some storage can be provided.

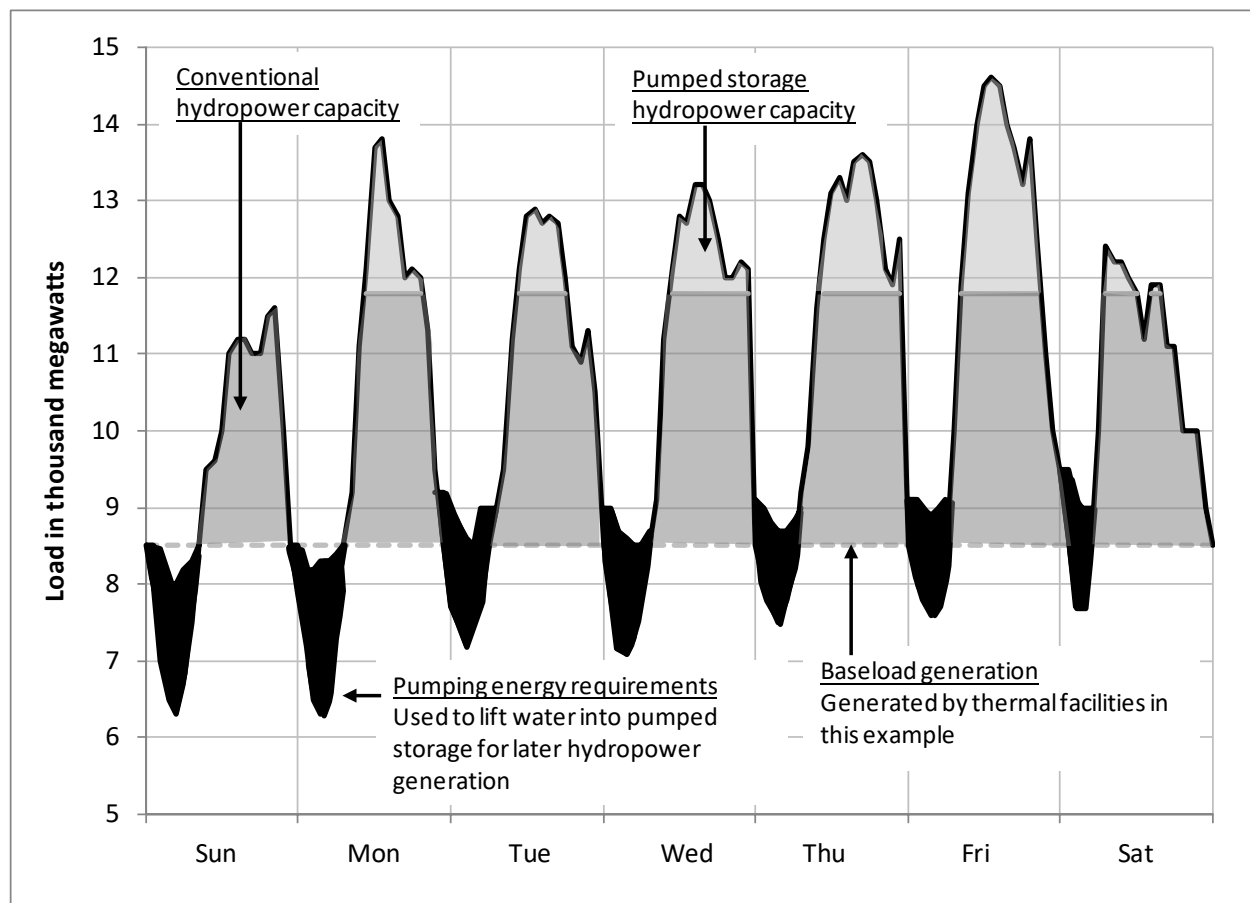


Figure 11-2. Weekly Load Curve for a Large Electric System.

c. Project Types. Hydroelectric projects have three major classifications: storage, run-of-river, and pumped storage. There are also combinations of projects that might be considered as separate classifications, but for purposes of discussing hydrologic analysis it is necessary to define only these three types.

(1) Storage plants are projects that usually have heads in the medium to high range (> 25 meters) and have provisions for storing relatively large volumes of water during periods of high streamflow to provide water for power generation during periods of deficient streamflow. Considerable storage capacity may be required because the period of deficient flow is quite frequently more than a year long and, in some instances, may be several years long. Because use of the stored water entails drawdown of the power storage, it is desirable that other water uses associated with the development of a storage plant permit frequent and severe draw downs during dry periods. Peaking operation, which is typically associated with storage projects, requires large and sometimes rapid fluctuations in releases of water through the generating units. It is often necessary to provide facilities to re-regulate the power releases if fluctuations of water levels below the project are not tolerable. Because storage projects are conducive to multipurpose use and because the power output from a storage plant is a function of the guaranteed output during a multi-year dry period, it is usually necessary to make detailed routing studies to determine the storage requirements, installed capacity, firm energy, and an operating plan.

(2) Run-of-river plants have little or no power storage and, therefore, must generate power from streamflow as it occurs. These projects generally have productive heads in the low to medium range (5-30 meters) and are usually associated with navigational developments or other multipurpose developments with limitations on reservoir drawdowns. Run-or-river projects usually have very little operational flexibility due to the absence or near-absence of storage and it is necessary that all water uses be compatible. The existence of one or more storage projects in the upstream portion of a river basin may make a run-of-river project in the lower portion of the basin feasible where it would not otherwise be feasible. In this situation, the storage projects provide a regulated outflow that is predictable and usable, while the natural streamflow might be neither predictable nor usable.

(a) Run-of-river projects may have provisions for a small amount of storage, often called pondage. This pondage detains the streamflow during off-peak periods in daily or weekly cycles for use in generating power during peak demand periods. If the cycle of peaking operation is a single day, the pondage requirements are based on the flow volume needed to sustain generation at or near installed capacity for 12 hours. If more storage capacity is available and large fluctuations in the reservoir surface are permissible, a weekly cycle of peaking operation may be considered. Because industrial and commercial consumption of power is significantly lower on weekends than on week days, an “off-peak” period is created from Friday evening until Monday morning. If generation from the hydroelectric peaking plants is not required during this period, water can be stored in the pondage for use during the 5-day peak-load period.

(b) Fluctuations in the tail waters of run-of-river projects are important, particularly in peaking operations, because of the dams have relatively low heads. Also, flood flows may curtail power generation due to high tail water. While flow-duration analysis can be used to estimate average annual energy production, sequential analysis may be required for more detailed analysis of extreme conditions. Chapter 12 of this EM discusses sequential analysis.

(3) Pumped-storage plants are projects that depend on pumped water as a partial or total source of water for generating electric energy. This type of project derives its usefulness from the fact that the demand for power is generally low at night and on weekends; therefore, pumping energy at a very low cost will be available from idle thermal generating facilities or run-of-river projects. If there is a need for peaking capacity and if the value of peaking power generation sufficiently exceeds the cost of pumping energy (at least a ratio of 1.5 to 1.0 because roughly 3 kWh of pumping energy are necessary to deliver enough water to provide 2 kWh of energy generation), pumped storage might be feasible. There are three types of pumped-storage development: diversion, off-channel, and in channel, which are detailed in Chapter 7 of EM 1110-2-1701, Hydropower.

(a) In general, pumped-storage projects consist of a high-level forebay where pumped water is stored until it is needed for generation and a low-level afterbay where the power releases are regulated, if necessary, and from which the water is pumped. The pumping and generating are done by generating units composed of reversible pump turbines and generator motors located along a tunnel or penstock connecting the forebay and afterbay. The water is pumped from the afterbay to the forebay when the normal power demand is low and least expensive and released from the forebay to the afterbay to generate power when the demand is high and most costly. The feasibility of pumped-storage developments is dependent upon the need for relatively large amounts of peaking capacity, the availability of pumping energy at a guaranteed favorable cost, and a load with an off-peak period long enough to permit the required amount of pumping.

(b) A unique feature of pumped-storage systems is that very little water is required for their operation. Once the headwater and tail water pools have been filled, only enough water is needed to take care of evaporation and seepage. For heads up to 300 meters, reversible pump turbines have been devised to operate at relatively high efficiency as either a pump or turbine. The same electrical unit serves as a generator and motor by reversing poles. Such a machine may reduce the cost of a pumped-storage project by eliminating the extra pumping equipment and pump house. The reversible pump turbine is a compromise in design between a Francis turbine and a centrifugal pump. Its function is reversed by changing the direction of rotation.

d. Need for Hydroelectric Power. The need for power is established by a power market study or survey. The feasibility of a particular hydroelectric project or system is determined by considering the needs as established by the survey, availability of transmission facilities, and the economics of the proposed project or projects. Although forecasts of potential power requirements within a region to be served by a project are not hydrologic determinations, they are essential to the development of plans for power facilities and to the determination of project feasibility

and justification. The power market survey is a means of evaluating the present and potential market for electrical power in a region.

(1) The survey must provide a realistic estimate of the power requirements to be met by the project and must show the anticipated rate of load growth from initial operation of the project to the end of its economic life. The survey also provides information regarding the characteristics of the anticipated demands for power. These characteristics, which must be considered in hydrologic evaluations of hydroelectric potential, include the seasonal variation of energy requirements (preferably on a monthly basis), the seasonal variation of capacity requirements (also preferably on a monthly basis), and the range of usable plant factors for hydroelectric projects under both adverse and average or normal flow conditions.

(2) The results of a power market survey might be furnished to the hydrologic engineer in the form of load duration or load curves (Figures 11-1 and 11-2) showing the projected load growth, the portion of the load that can be supplied by existing generating facilities, and the portion that must be supplied by future additions to the generating system. From these curves, the characteristics of planned hydroelectric generating facilities can be determined. Because these data are developed from the needs alone without consideration of the potential for supplying these needs, the next step is to study the potential for hydroelectric development, given the constraints established in the study of needs.

e. Estimation of Hydroelectric Power Potential. Traditionally, hydroelectric power potential has been determined on the basis of the critical hydroperiod as indicated by the historical record. The critical hydroperiod is defined as the period when the limitations of hydroelectric power supply due to hydrologic conditions are most critical with respect to power demands. Thus, the critical period is a function of the power demand, the streamflow, and the available storage. In preliminary project planning, the estimates of power potential are often based on a number of simplifying assumptions because of the lack of specific information for use in more detailed analyses. Although these estimates and the assumptions upon which they are based are satisfactory for preliminary investigations, they are not suitable for every level of engineering work. Many factors affecting the design and operation of a project are ignored in these computations. Therefore, detailed sequential analyses of at least the critical hydroperiod should be initiated as early as possible, usually when detailed hydrologic data and some approximate physical data concerning the proposed project become available. Because of the availability of software packages for accomplishing these sequential routings, they can be done rapidly and at a relatively low cost.

(1) The manner in which the streamflow at a given site is used to generate power depends upon the storage available at the site, the hydraulic and electrical capacities of the plant, streamflow requirements downstream from the plant, and characteristics of the load to be served. In theory, the hydroelectric power potential at a particular site, based on repetition of historical runoff, can be estimated by identifying the critical hydroperiod and obtaining estimates of the average head and average streamflow during this critical period. The data can then be used in one of the equations below to

calculate the power available from the project. Equation 11-1 uses English units and Equation 11-2 uses Metric units.

$$P_{kW} = \frac{1}{11.81} QHe \quad (11-1)$$

$$P_{kW} = 9.81 QHe \quad (11-2)$$

To convert a project's power output to energy, Equations 11-1 and 11-2 must be integrated over time. Equations 11-3 and 11-4 show these integrations. Equation 11-3 uses English units and Equation 11-4 uses Metric units.

$$E_{kWh} = \frac{1}{11.81} \int_0^t Q_t H_t e dt \quad (11-3)$$

$$E_{kJ} = 9.81 \int_0^t Q_t H_t e dt \quad (11-4)$$

where:

- P_{kW} = power available from the project, kilowatts (kW).
- E_{kWh} = energy generated during a time period, kilowatt-hour (kWh).
- E_{kJ} = energy generated during a time period, kilojoules (kJ).
- Q = average streamflow during the time period, cfs or m³/sec.
- H = average head during the time period, feet or meter (Head = headwater elevation - tailwater elevation - hydraulic losses).
- t = number of hours in the time period.
- e = overall efficiency expressed as the product of the generator efficiency and the turbine efficiency (usually 80 to 85% expressed as a decimal).

In practice, the summation of energy production over the critical period is performed with a sufficiently small time step to provide reasonable estimates of head and, therefore, energy. Two basic approaches are available: flow-duration and sequential analysis.

(2) For run-of-river projects, where the headwater elevation does not vary significantly, the flow-duration approach can be used to estimate average annual energy production. The duration curve can be truncated at the minimum-flow rate for power production. The curve can also be trun-

cated for high flows if the tailwater elevation is too high for generation. The remaining curve is converted to capacity-duration and integrated to obtain average annual energy. EM 1110-2-1701, Hydropower, describes the method to perform energy computations based on flow-duration data in Section 5-7.

(3) Sequential streamflow analysis will be applied to most reservoir studies. The procedure allows detailed computations of the major parameters affecting hydropower (e.g., headwater and tailwater elevation, efficiency, and flow release). By performing the analysis in sufficiently small time steps, an accurate simulation of the reservoir operation, power capacity and energy production can be obtained. Chapter 5 of EM 1110-2-1701, Sections 5-8 through 5-10, provides a discussion of sequential routing studies. Appendix C in EM 1110-2-1701 provides information concerning software packages that are available for use in these studies.

f. Hydropower Effect on Other Project Purposes. Thorough investigations of all aspects of the power operation must be conducted to ensure that the power operations are consistent with operations for other authorized or approved water uses, such as flood risk management, and for other conservation storage uses. The operation rules that are necessary to effect the coordination are usually developed and tested using engineering judgment and detailed sequential routing studies. However, it is necessary to define the interactions between power and other project purposes before initiating operation studies.

(1) Power generation is generally compatible with other purposes that require storage of water in a reservoir, and releases of water from a reservoir for downstream needs. However, power generation may compete with purposes that require withdrawal of the water directly from the reservoir or that restrict fluctuations in the reservoir level. Furthermore, when extensive flooding is anticipated downstream from a reservoir project, it may be necessary to curtail power releases to accomplish flood control objectives. It is often possible to pass part or all of the flood control releases through the generating units, thereby reducing the number of additional outlets needed and significantly increasing the energy production over what would be possible if the flood control releases were made through conduits or over the spillway. Also, many of the smaller floods can be completely regulated within the power drawdown storage, an operation that is beneficial to power because it provides water for power generation that might otherwise have been spilled. This joint use can reduce the exclusive flood control storage requirements and also reduce the frequency of use of flood control facilities.

(2) Water for municipal, industrial, or agricultural use can be passed through the generating units with no harmful effects if the point of withdrawal for the other use is below the point where the power discharge enters the river. Re-regulation may be required for hydropower peaking operations to “smooth out” the power releases. Re-regulation structures capture the peaking flows and attenuate them through additional storage before releasing them farther downstream.

(3) Low-flow augmentation for navigation, recreation, fish and wildlife, or other purposes can be accomplished by releases through power-generating units. In the case of baseload projects, the power release is ideally suited for this type of use. With peaking projects, however, a re-regulation structure may be necessary to provide the relatively uniform releases that might be required for navigation or for in-stream recreation. Release of water for quality enhancement can sometimes be accomplished through the generating units. Although the intakes for the turbines are usually located at a relatively low elevation in the reservoir where dissolved oxygen content might be low, the oxygenation that occurs in the tailrace and in the stream below the project may produce water with an acceptable dissolved oxygen content. The water released from the lower levels of the reservoir is normally at a relatively low temperature and, thus, ideal for support or enhancement of a cold water fishery downstream. If warm waters are needed for in-stream recreation, for fishery requirements, or for any other purpose, a special multilevel intake may be required to obtain water of the desired temperature.

(4) Large hydropower projects may provide enhanced value for recreation, because a much larger reservoir is frequently required. However, the large drawdowns associated with large storage projects may create special problems with respect to the location of permanent recreational facilities and may create mudflats that are undesirable from the standpoint of aesthetics and public health requirements. The drawdown may also expose boaters, swimmers, and other users to hazardous underwater obstacles unless provisions are made to remove these obstacles to a point well below the maximum anticipated drawdown. Consideration to these factors should be given when planning and siting recreation activities.

11-6. Water Quality Considerations.

a. Water Quality. Water quality deals with the kinds and amounts of matter dissolved and suspended in natural water, the physical characteristics of the water, and the ecological relationships between aquatic organisms and their environment. It is a term used to describe the chemical, physical, and biological characteristics of water in respect to its suitability for a particular purpose. The same water may be of good quality for one purpose or use, and bad for another, depending on its characteristics and the requirements for the particular use. Refer to ER 1110-2-1201 for more information.

b. Reservoirs In Streams. The presence of a reservoir in a stream affects the quality of the outflow as compared to the inflow by virtue of the storage and mixing that takes place in the reservoir. The effect of such an impoundment may be easily evaluated for conservative parameters if the waters of the reservoir are sufficiently mixed that an assumption of complete mixing within an analysis time period does not lead to appreciable error. However, this assumption is limited to relatively small, shallow reservoirs.

c. Reservoir Outflow and Inflow. The simplest technique requires the assumption that the reservoir outflow during a given time period is of constant quality and equal to the quality of the reservoir storage at the end of the computation time period. It is then assumed that the inflow for

the time period occurs independently of the outflow, and reservoir quality is determined by a quality mass balance at the end of the time period. This approach is equivalent to the mass balance of water in reservoir routing.

d. **Reservoir Water Quality.** Simple mass balance procedures may be applicable in some situations; however, usually more comprehensive methods should be considered. Chapter 4, “Water Quality Assessment Techniques,” in EM 1110-2-1201, Reservoir Water Quality Analysis, describes various techniques available for assessing reservoir water quality conditions. There is a hierarchy of available techniques that reflects increasing requirements of time, cost, and technical expertise. The increasing efforts should provide accompanying increases in the degree of understanding and resolution of the problem and causes. This hierarchy includes screening diagnostic and predictive techniques, which are described.

e. **Reservoirs as Detention Basins.** Reservoir mixing is a continual process where low inflows of poor quality are stored and mixed with higher inflows of better quality. Generally, this is accomplished in large reservoirs where annual or even multiple-year flows are retained, but the concept extends to small reservoirs in which weekly or even daily quality changes occur due to variability of loading associated with the inflow.

(1) The use of a reservoir as a mixing device should be considered whenever the inorganic water quality is unacceptable during some periods but where the average quality falls within the acceptance level. Lake Texoma on the Red River is an example of a reservoir that modifies the quality pattern. Although monthly inflow quality has equaled 1,950 mg/l chloride concentration, the outflow has not exceeded 520 mg/l.

(2) Many materials that enter a reservoir are removed by settling. This applies not only to incoming settleable solids, but also to colloidal and dissolved materials that become of settleable size by chemical precipitation or by synthesis into biological organisms. Reservoirs are often used to prevent such settleable material from entering navigable rivers where settleable materials would interfere with desired uses. However, reservoirs that receive substantial sediment will have a short useful life if they are not sustainably designed with sediment flushing, sediment bypass or some other method of sediment management. Planning should include the evaluation of the sustainability of a reservoirs, their ultimate fate, and possible replacement. Reservoir sedimentation is covered in EM 1110-2-4000, Sedimentation Investigations of Rivers and Reservoirs. Planning should include the evaluation of the sustainability of a reservoirs, their ultimate fate, and possible replacement.

f. **Reservoirs as Stratified Systems.** Reservoirs become stratified if density variations caused by temperature or dissolved solids are sufficiently pronounced to prevent complete mixing. This stratification may be helpful or harmful depending on the outlet works, inflow water quality, and the operating procedure of the reservoir.

(1) Temperature stratification can be beneficial for cold water fisheries if the water that enters the reservoir during the cooler months can also be stored and released during the warmer months. The

cooler water released during the warm months can also be valuable as a cooling water source, can provide for higher oxygen transfer (re-aeration) or slower organic waste oxidation (deoxygenation), and can make the water more aesthetically acceptable for water supply and recreational purposes.

(2) Dissolved oxygen stratification usually occurs in density stratified lakes, particularly during the warmer months. The phenomenon occurs because oxygen that has been introduced into the epilimnion by surface re-aeration does not transfer through the metalimnion into the hypolimnion at a rate high enough to satisfy the oxygen demand by dissolved and suspended materials and by the benthic organisms. Thus, the cool bottom waters that are sometimes desirable may be undesirable from a dissolved oxygen standpoint unless energy dissipation structures are constructed to transfer substantial oxygen into the reservoir pool or the reservoir discharge. Mechanical reservoir mixing to equalize temperature and transfer oxygen to lower reservoir levels is one possible tool for managing reservoir water quality.

g. Reservoirs as Flow Management Devices. Reservoirs may improve water quality by merely permitting the management of flow. This management may include maintenance of minimum flows, blending selective releases from one or more reservoirs to maintain a given stream quality, and the exclusion of a flow from a system by diversion.

(1) Minimum flow is often maintained in a stream for navigation, recreation, fish and wildlife, and water rights purposes. Such flows may also aid in maintaining acceptable water quality.

(2) There is general agreement that water may be stored and selectively released to help reduce natural water quality problems where source control is not possible, and also that water should not be stored and released solely to improve water quality where similar improvement may be achieved by treatment at the source. The use of a water resource to dilute treatable waste materials is regarded as the misuse of a valuable resource in most cases.

(3) Selective release of water from one or more reservoirs may help improve quality at one or more downstream locations. Such releases may be one of the governing factors in establishing reservoir management rules. The water to be released may either be good quality water that will improve the river quality or poor quality water that is to be discharged when it will do a minimum of harm (e.g., during high flow). The HEC-RAS software package has a Nutrient Sub Module (NSM) library that accounts for water quality in the river system. NSM is the same water quality library used in the Gridded Surface Subsurface Hydrologic Analysis (GSSHA) software package. CE-QUAL-W2, another water quality model is designed to perform quality analysis based on a reservoir simulation for quantity demands and subsequently determine the additional releases needed to meet water quality objectives (Portland State University 2012). The ERDC publication library contains additional documentation on the model (ERDC 2014). Refer to Appendix B for website information. Section III, Chapter 4 of EM 1110-2-1201, Reservoir Water Quality Analysis, describes various predictive techniques, including numerical and physical models.

CHAPTER 12

Conservation Storage Yield

12-1. Introduction. The conservation pool of a Federal reservoir can support many uses and objectives and can provide some yield for the demand of each of those uses. The procedures presented are generally used to determine the relationship between reservoir storage capacity and reservoir yield (supply) for a single reservoir. The procedures may be used to determine storage requirements for water supply, water quality control, hydroelectric power, navigation, irrigation, and other conservation purposes. Although the discussions are limited to single reservoir systems, many of the principles are generally applicable to multi-reservoir systems. Chapter 4, Reservoir Systems, presents concepts regarding the analysis of multi-reservoir systems.

12-2. Storage-Yield Theory.

a. **Firm Yield.** Firm yield is the largest consistent flow rate (demand) that can be provided throughout a historic period of streamflow. Use of the firm yield brings the stored water volume exactly to zero once during the period of historic record during what is designated the critical period for that firm yield and storage capacity. It is important to note that firm yield is determined from an observed streamflow record, which means that it is not guaranteed that the reservoir will be able to deliver that yield throughout its life cycle. Reservoir storage capacity and its firm yield are related through the storage-yield curve. The objective of an analysis of firm yield is to determine the maximum cumulative shortfall between the water demand and inflow supply, which defines the water storage capacity needed to meet the demand throughout the period of record. The ability to store water increases the firm yield by allowing water to be saved and used to meet demand when the streamflow is less than the demand. The storage-firm yield relationship is initially warranted for a reservoir site during the planning stage and is then reevaluated when the reservoir operation policies are developed or changed.

b. **Safe Yield.** Safe yield may not be equivalent to firm yield. As stated above, firm yield is based on the analysis of critical periods of water shortage in an observed streamflow record that means that there is not a 100% guarantee that this flow rate will satisfy all future demands. Safe yield typically includes a factor of safety over the firm yield to help account for uncertainty in the future streamflow at the site.

c. **Critical Periods.** Critical periods are the driest periods of record where the inflow does not satisfy the demand and reservoir storage is required. Therefore, it is the period in the sequential record that requires the largest volume from storage to provide a specified yield. A period of record can have more than one critical period and all these periods must be analyzed to determine the minimum storage necessary to meet the demand.

d. **Storage-Yield Relationships.** The determination of storage-yield relationships for a reservoir project is one of the basic hydrologic analyses for reservoirs. The basic objective is to either determine the reservoir yield given a storage allocation or find the storage required to obtain a desired yield. At the beginning of the analysis, determine the project purpose and objectives, the project constraints (e.g., reservoir storage available, outflow capacity, and downstream channel characteristics), compile basic data (e.g., demands, inflow, and losses), select an appropriate time interval, and select an appropriate method of analysis.

e. **Methods.** The storage-yield relationship can be determined through several methods. These range in complexity and have their limitations. The method selected will depend on the objectives of the analysis and the basic hydrologic and physical data available as detailed models require more data that may not be available. The simple methods require less data, but the reliability of the results decreases rapidly as the length of hydrologic record decreases. Hydrologic data and data simulation are discussed in Chapter 5.

f. **References.** References for storage-yield determinations include the HEC Methods for Storage/Yield Analysis (January 2014 Draft, HEC 2014 Draft) and Volume 8, Reservoir Yield, of the International Hydrological Decade (IHD) document Hydrologic Engineering Methods for Water Resources Development (IHD1975).

12-3. Simple Methods.

a. **General.** Firm yield can be determined through several simplified computational methods: the Rippl Mass Diagram, Sequent Peak method, linear programming optimization, and simplified iterative simulation. These methods are adequate for preliminary analysis, but they do not adjust the inflow to remove the effects of evaporation, precipitation, leakage, past withdrawals from the reservoir, and past returns. They also do not account for realistic multi-objective reservoir operations, or the interaction with other reservoirs and downstream supplies. Simple methods still have a role in screening studies or as tools to obtain estimates of input data for sequential routings. These simple methods focus on computing the maximum cumulative deficit in supplying a specified demand and define the amount of required storage to supply the demand through the critical periods.

b. **Rippl Mass Diagram.** The Rippl Mass Diagram approach is restricted to evaluating systems with a constant demand. The method compares the accumulated inflow over time to the accumulated demand. The diagram is constructed by accumulating inflows to a reservoir site throughout the period of record and plotting these accumulated inflows versus the sequential time period. As an example, Figure 12-1 shows the sequential mass curve for the Fountain Creek, Colorado gauge.

(1) The desired yield rate, in this example 11,000 acre-feet per year (AF/yr), is represented by the slope of the demand curve (dotted lines in Figure 12-1). Demand curves are constructed tangent to

the mass curve at each low point (line ABC) and at the preceding high point that gives the highest tangent (line DEF). The vertical distance between these two lines (line BE) represents the storage required to provide the desired yield during the time period between the two tangent points (points D and B). The maximum vertical difference in the period is the required minimum storage to meet the desired yield, during the given flow sequence. Of the two time periods shown in Figure 12-1, the period from December 1973 through December 1983 provides the maximum of the two storages and is the most critical.

(2) The critical period is the duration of time from point D, when conservation storage drawdown begins, to point F, when the reservoir conservation storage fills. The critical drawdown is from point D to point B, while during the time from B to F the reservoir would be refilling. The critical period is the time over which water would be drawn from reservoir storage to satisfy the demand. It includes both the critical drawdown period and the period of time over which the reservoir refills.

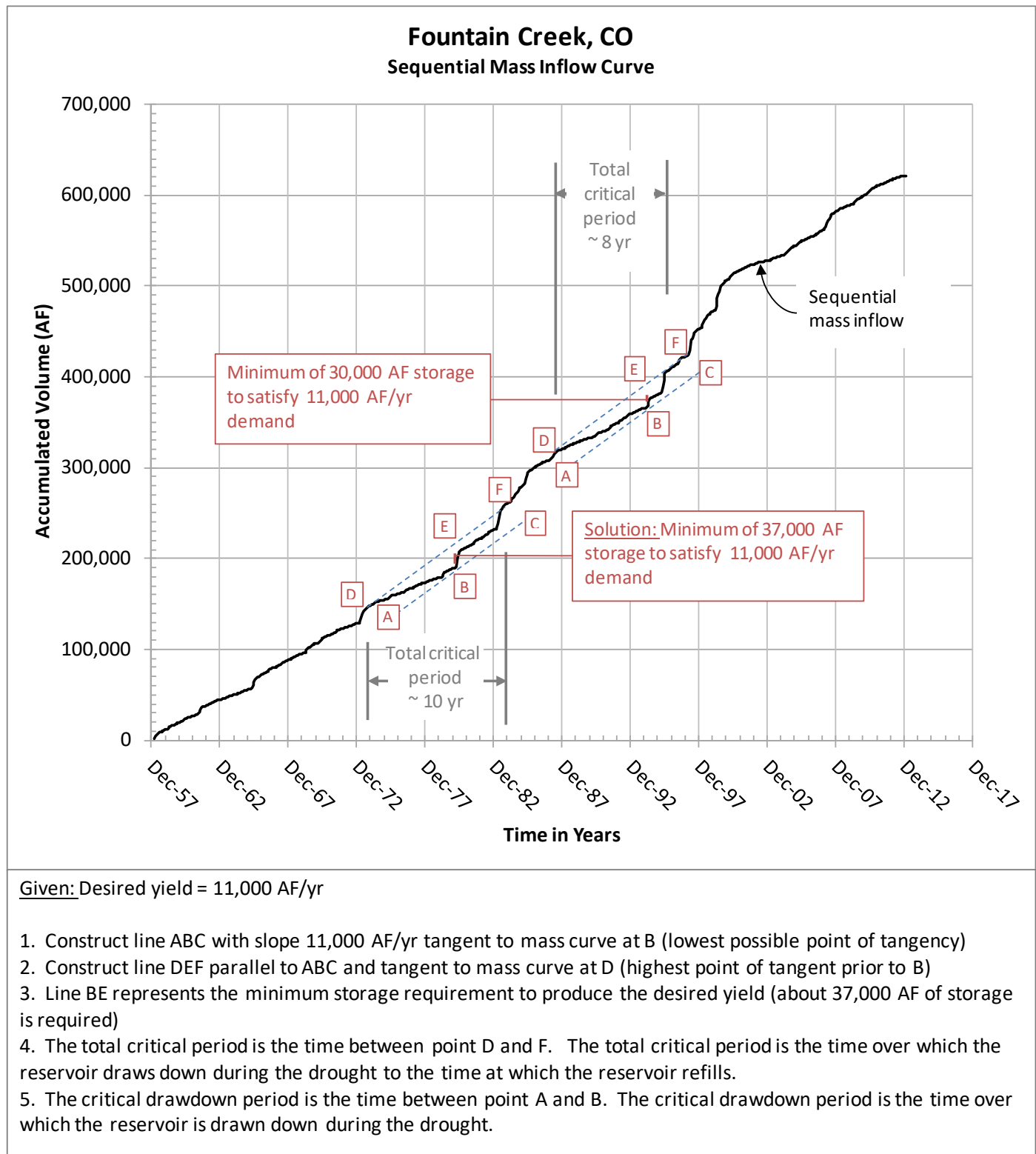


Figure 12-1. Storage Determination Using Rippl Diagram for Fountain Creek, CO.

(3) A limitation of a sequential mass curve analysis is its results are based only on the record of data collected. The sequential mass curve method does not indicate the relative frequency of a shortage and future streamflow are assumed to be within the ranges of the flows on the historic record. However, by using nonsequential methods, a curve of yield versus shortage frequency can be determined. The International Hydrologic Decade document (IHD 1975) is an additional source on sequential mass curve analysis.

c. Sequent Peak Method. Unlike the Rippl Mass Diagram, the Sequent Peak method allows a variable seasonal water demand. The net inflow (inflow minus demand) is calculated for every time period and plotted for the period of record. The plotted cumulative net inflow contains peaks and troughs, and the largest difference between a peak and a subsequent trough defines a critical period and the stored water volume required to supply the demand for that period. A non-graphical variation of the Sequent Peak method computes the deficit between inflow and demand (demand minus inflow) for every time period and accumulates the deficit over the period of record. Equation 12-1 shows the cumulative deficit calculation used in the non-graphical method. The final adopted storage-yield relationship is the dashed yield curve, which is the lower of the two curves. Often a storage-yield curve is continued to span the storage required to develop a firm yield equal to the average annual flow. Refer to the HEC Methods for Storage/Yield Analysis (HEC 2014 Draft) for details and examples.

$$\text{Cumulative_Deficit}(t) = \max \left(0, \sum_{i=1}^{i=t} \text{Demand}(i) - \text{Inflow}(i) \right) \quad (12-1)$$

d. Linear Programming Optimization. Linear programming optimization can be used to represent simple reservoir mass balance across all time periods. Linear constraints capture continuity (change in storage between periods equals inflow minus outflow volume), reservoir capacity as an upper bound on reservoir storage, and demand as a lower bound on reservoir withdrawal. The objective function either maximizes a consistent withdrawal given a specified storage capacity, or minimizes the storage capacity given a defined withdrawal rate. A model using period of record inflows would provide the same result as the simple methods that determine maximum deficit for a given demand.

e. Simple Iterative Simulation Approach. The iterative simulation approach can account for both constant and seasonally varying demands. Equation 12-2 shows the mass balanced used to calculating the reservoir storage with each time step. Note that a more sophisticated reservoir simulation tool using the iterative simulation approach would produce different results than those determined using the simplified method. The difference in results would be due to the tool's ability to simulate reservoir losses, gains, returns, and the interaction with other reservoirs and downstream flows. Refer to the HEC document Methods for Storage/Yield Analysis for details and examples.

$$\text{Mass Balance: } \text{Storage}(t) = \max(\text{Storage}(t-1) + \text{Inflow}(t) - \text{Release}(t), \text{storage capacity}) \quad (12-2)$$

12-4. Detailed Simulation.

a. Detailed computer model simulation to obtain storage-yield relationships accounts for the effects of evaporation, precipitation, leakage, past withdrawals from the reservoir, and past returns to the reservoir pool. They may also include stochastically generated streamflow. Detailed sequential routing is particularly adaptable to the use of variable demand schedules and every effort should be made to incorporate the analysis of reservoir yield to incorporate all known demand data into the criteria for routing. Project purposes that often require analysis of seasonal variations in demand include low-flow water management, diversion and return flows, water quality control, and hydro-power generation.

(2) Low-Flow Water Management. The operation of a reservoir for low-flow water management at a downstream control point is difficult to evaluate without a detailed sequential routing, because the operation is highly dependent upon the flows that occur between the reservoir and the control point, called intervening local flow. As these flows can vary significantly, a yield based on long-period average intervening flows can be subject to considerable error. A detailed sequential routing, in which allowance is made for variations of intervening flows within the routing interval, produces a more reliable estimate of storage requirements for a specified yield and reduces the chance of overestimating a firm yield. Ordinarily, the yield and the corresponding operation of a reservoir for low-flow management are determined by detailed sequential routing of the critical period and several other periods of low flow. The entire period of recorded streamflow may not be required, unless summary-type information is needed for functions such as power.

(3) Diversion and Return Flows. The analysis of yield for diversions is complicated by the fact that diversion requirements may vary from year to year as well as from season to season. Furthermore, the diversion requirements may be stated as a function of the natural flow and water rights rather than as a fixed amount. In addition, diversion amounts may often be reduced or eliminated when storage in the reservoir reaches a certain critical low value. When any one of these three items is important to a given reservoir analysis, detailed sequential analysis for the entire period of flow record should be made to accurately determine the yield and the water management criteria. Coordination of the water management criteria for other purposes with the diversion requirements may also be achieved with the detailed sequential analysis results.

(4) Water Quality Control. Practically every variable under consideration in a water quality study will vary seasonally. Water quality variables include variation in all of the following: quality of the inflow, quality of the reservoir waters due to inflow quality and evaporation, quality of natural streamflow entering the stream between the reservoir and the control station, effluent from treatment plants and storm drainage outflow between the reservoir and the control station, and quality requirements at the control station. Accurate evaluation of project performance in terms of water quality must consider all of these variations. Additionally, there are several quality parameters that may require

study, and each parameter introduces additional variations that should be evaluated. For example, if temperature is an important parameter, the level of the reservoir from which water is released should be considered in addition to the above variables. Likewise, if oxygen content is important, the effects of release through power units versus release through conduits must be evaluated.

(5) Hydroelectric power generation. Power production is a function of both head and flow, which requires a detailed sequential study when the conservation storage is relatively large and the head can be expected to fluctuate significantly. Refer to the hydroelectric power generation section of this chapter for more information.

a. Data Requirements. Data requirements for a firm yield analysis include a period of record inflow, evaporation records, precipitation records, and water withdrawal and return data for the period if they exist. Accurate data on precipitation and evaporation are extremely important because the storage requirements will be determined by a drought period in which evaporation is likely high and precipitation is low. A record of measured evaporation from the region is preferred to average monthly values.

b. Computation Time Interval. While monthly computation intervals are typically adequate for analysis with simple methods, smaller computational intervals may be more appropriate. The selection of a computation interval is dependent upon four major factors: the demand schedule that will be used in determining the yield, the accuracy required by the study objectives, the data available for use, and the phase relationship between periods of high and low demands and high and low flow. As a general rule, shorter computational intervals will provide more accurate results. This is due to many factors, such as better definition of relationships between inflow and releases, and better estimates of average reservoir levels for evaporation and power-head calculations. The computational time step should not be shorter than the shortest period for which flow and demand data are available. Attempts to “manufacture” flow or demand data are usually time consuming and may create errors or give a false impression of accuracy.

c. Stochastic Streamflow. Stochastic (random) streamflow can be generated to extend the observed period of record with synthetic values. By increasing the length of the streamflow record additional critical periods will be generated to test reservoir operation and compute firm yield. It should be kept in mind, however, that the statistical characteristics of the original streamflow record are maintained in the synthetically-generated flows. This means that it is unlikely that the full range of possibilities in flow will be captured by the reproduced record as it cannot be assumed the period of record captures all possible magnitudes of flows that the site can physically experience. The use of stochastic streamflow is discussed further in the HEC document Methods for Storage-Yield Analysis (HEC 2014 Draft).

d. Software Packages. Modern computing power has allowed for the increase in the modeling of system detail and optimization. Software packages like the Hydrologic Engineering Center Reser-

voir System Simulation (HEC-ResSim) (HEC 2013) provide efficient models for the analysis of reservoir systems. Methods within these software packages are preferred over the simplified methods discussed previously.

12-5. Physical Constraints and Storage Limitation. Physical constraints of storage-yield analysis include the conservation storage available, minimum pool, outlet capacities, and channel capacities. The addition of hydropower as a purpose will require the inclusion of constraints to power generation, e.g., maximum and minimum head, penstock capacity, and power capacity. If flood risk management is to be included as a project purpose, the maximum conservation storage feasible at a given site will be affected by the flood risk management analysis.

(1) The boundaries between the storage zones and operational boundaries within the zones may be fixed throughout the year, or they may vary from season to season. The varying boundaries usually offer a more flexible operational plan that may result in higher yields for all purposes, although an additional element of chance is often introduced when the boundaries are allowed to vary.

(2) Water management criteria can be tested by detailed sequential routing for the period of recorded streamflow. Several alternative patterns and magnitudes of seasonal variations should be evaluated to determine the response of the storage-yield relationship and the flood control efficiency to the seasonal variation of the boundary. A properly designed seasonally varying storage boundary should not reduce the effectiveness of flood control storage to increase the conservation yield.

(3) Flood control operation is generally simplified in conservation studies because the routing interval for such studies is frequently too long to adequately define the flood control operation. Nevertheless, flood control constraints should be observed insofar as possible. For example, channel capacities below the reservoir are considered for release purposes, and storage above the top of flood control pool is not used.

12-6. Operation Criteria. In multipurpose projects, every effort should be made to develop operation criteria that maximize the complementary uses for the various conservation purposes. However, in operating all its projects, regardless of their authorized purposes, the USACE always seeks to minimize risk to public safety.

a. Hydroelectric Power Generation.

(1) If hydroelectric power is included as a project purpose, detailed sequential routings are necessary to develop water management criteria, to coordinate power production with other project purposes, and to determine the project's power potential. As a rule, simplified methods are usable for power projects only for preliminary or screening studies, reservoirs with very little power storage, or when energy is a by-product to other operations. Flow-duration analysis, described in Chapter 11, is typically applied in these situations. Power production is a function of both head and flow, which requires a detailed sequential study when the conservation storage is relatively large and the head can be expected to fluctuate significantly.

(2) Determination of firm power or firm energy is usually based on sequential routings over the critical period. The critical period must consider the combination of power demand and critical hydrologic conditions. Various operational plans are used in an attempt to maximize power output while meeting necessary commitments for other project purposes. When the optimum output is achieved, a water management guide curve can be developed. The curve is based on the power output itself and on the plan of operation followed to obtain the maximum output. Critical period analysis and curve development are described in Section 12-11. Additional sequential routings for the entire period of flow record are then made using the guide curve developed in the critical period studies. These routings are used to coordinate power production with flood control operation and to determine the average annual potential energy available from the project.

(3) In areas where hydroelectric power is used primarily for peaking purposes, it is important that storage requirements be defined as accurately as possible because the available head during a period of peak demand is required to determine the peaking capability of the project. An error in storage requirements, on the other hand, can adversely affect the head with a resultant loss of peaking capability.

(4) Tailwater elevations are also of considerable importance in power studies because of the effect of head on power output. Several factors that may adversely affect the tailwater elevation at a reservoir are construction of a re-regulation reservoir below the project under consideration, high pool elevations at a project immediately downstream from the project under consideration, and backwater effects from another stream if the project is near the confluence of two streams. If any of these conditions exist, the resultant tailwater conditions should be carefully evaluated. For projects in which peaking operation is anticipated, an assumed "block-loading" tailwater should be used to determine reservoir releases for the sequential reservoir routing. The "block-loading" tailwater elevation is defined as the tailwater elevation resulting from sustained generation at or near the plant's rated capacity, which represents the condition under which the project is expected to operate most of the time. Although in reality the peaking operation tailwater would fluctuate considerably, the use of the block-loading tail water elevation ensures a conservative estimate of storage requirements and available head.

(5) Reversible pump turbines have enhanced the feasibility of the pumped-storage type of hydroelectric development. Pumped storage includes reversible pump turbines in the powerhouse along with conventional generating units, and an afterbay is constructed below the main dam to retain water for pumping during nongenerating periods. Sequential routing studies are required for analyses of this type because of the need to define storage requirements in the afterbay, pumping requirements and characteristics, and the extent to which plan should be developed. Many of the existing and proposed pumped-storage projects in the United States, however, are single-purpose projects that do not have conventional units and often use off-channel forebays.

12-7. Drought of Specified Return Period.

a. An effort can be made to define a critical dry period that has a particular return period. This return period can then be used to compute a firm yield or storage requirement having a failure probability equal to the inverse of the determined return period. The method shown here is the nonsequential mass curve analysis described in Volume 8, Reservoir Yield, of the International Hydrological Decade document Hydrologic Engineering Methods for Water Resources Development (IHD1975). This procedure is limited to water supply demands that are uniform in time. It involves the development of probability relations for varying durations of streamflow. Because the nonsequential analysis is restricted to uniform demands, it does not produce results as accurate as those obtained by sequential methods.

b. Use the historical flows, supplemented by simulated flows where needed, to determine frequency tables of independent low-flow events for several durations (e.g., 3, 6, 9, 12, 18, 24, 36 and 48 months). A series of low-flow events for a particular duration is selected by computing and arranging in order of magnitude, the independent minimum-flow rates for the duration for the period of record.

c. After the frequency tables of independent low-flow events are computed for various durations, low-flow frequency curves are obtained by plotting the ranked events by their plotting positions on log-probability scales. The patterns in the distribution of low flows vary from site to site, so it is recommended that several types of distributions be used to fit the data. Chapter 4 of EM 1110-2-1415, Hydrologic Frequency Analysis, describes the procedure and presents an example table and frequency plot. Figure 12-2 shows example low-flow frequency curves from the International Hydrologic Decade (IHD 1975).

d. Use care when interpreting the low-flow curves because the abscissa is “Nonexceedance frequency per 100-years,” or the number of events within 100-years that have a flow equal to or less than the indicated flow. Thus, when low-flow durations in excess of one year are evaluated, the terminology cannot be used interchangeably with probability. For instance, during a 100-year period, the maximum number of independent events of 120 months (10-years) duration is 10. Therefore, the 120-month duration curve cannot cross the value of 10 on the “Nonexceedance frequency per 100-year” scale.

e. Plot the points from the low-flow frequency curves (Figure 12-2) on logarithmic scales to determine the minimum runoff-duration curves for various frequencies, as shown in Figure 12-3. The flow rates are converted to volumes (million cubic meters in this example) and the durations are converted from months to years. The logarithmic scales simply permit more accurate interpolation between the durations represented by the frequency curves.

f. The nonsequential mass curve (Figure 12-4) is developed by selecting the desired volume-duration curve from Figure 12-3 and plotting this curve on an arithmetic grid. The desired yield is then used to determine the storage requirement for the reservoir. The storage requirement is determined by drawing a straight line, with slope equivalent to the required gross yield,

and by plotting this line tangent to the mass curve. The absolute value of the negative vertical intercept represents the storage requirement. The application of this procedure is severely limited in the case of seasonal variations in runoff and yield requirements because the nonsequential mass curve does not reflect the seasonal variation in streamflow, and the tangent line does not reflect seasonal variations in demand. However, the method does provide an estimate of yield reliability.

g. Evaporation Losses. Another disadvantage of these simplified types of storage-yield analysis is the inability to evaluate evaporation losses accurately. This may not be critical in humid areas where net evaporation (lake evaporation minus preproject evapotranspiration) is relatively small, but it can cause large errors in studies for arid regions. Also, these procedures do not permit consideration of seasonal variations in requirements, system nonlinearities, conflicting and complementing service requirements, and several other factors.

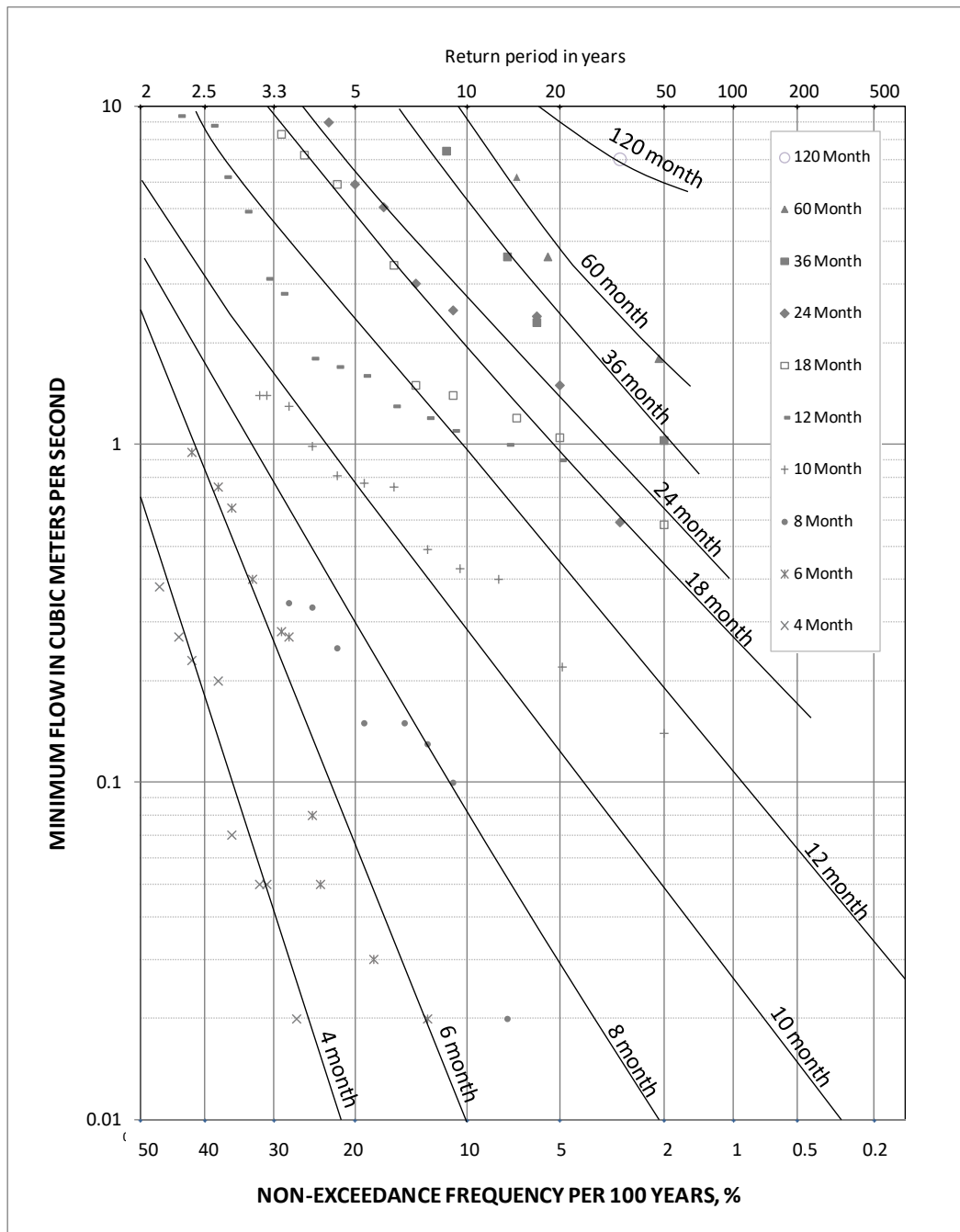


Figure 12-2. Nonexceedance Frequency Curves.

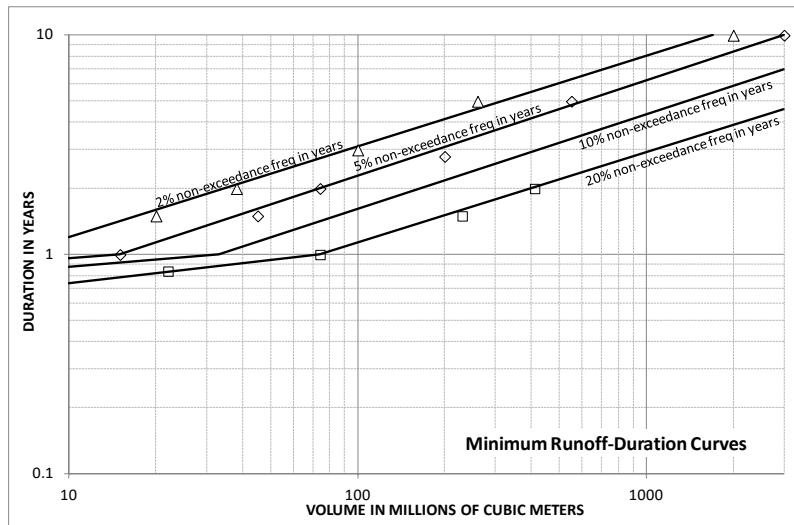


Figure 12-3. Minimum Runoff-Duration Curves.

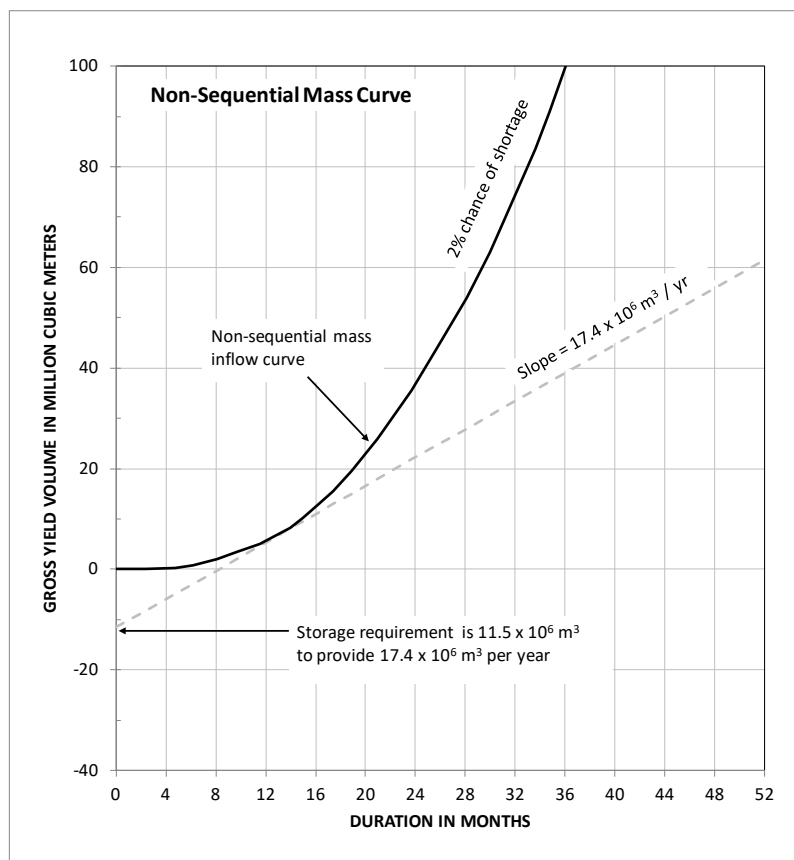


Figure 12-4. Nonsequential Mass Curve.

12-8. Effects of Water Deficiencies.

a. Water Shortages. Absolute guarantees of water yield are usually not practical, and the designer should therefore provide estimates of shortages that could reasonably develop in supplying the demands with available storage. If nonsequential procedures have been used, information on future shortages is limited to the probability or frequency of occurrence, and the duration or severity of shortages will not be known. In using the Rippl Method, the computations are based on just meeting the demand; therefore, no shortages are allowed during the period of analysis. The result gives no information on the shortages that might be expected in the future. Only in the detailed sequential analysis procedure is adequate information on expected future shortages obtainable.

b. Amount and Duration of Water Shortage. The amount and duration of shortage that can be tolerated in serving various project purposes can greatly influence the amount of storage required to produce a firm yield. These tolerances vary a great deal for different project purposes and should be analyzed carefully in reservoir design. Also, changes in reservoir operation should be considered to meet needs during drought (HEC 1990a).

c. Intolerable Shortages. Shortages are generally considered to be intolerable for purposes such as drinking water. However, some reduction in the quantity of M&I water required can be tolerated without serious economic effects by reducing some of the less important uses of water such as lawn watering, car washing, etc. Most designs of reservoir storage for M&I water supply are based on supplying the firm yield during the most critical drought of record. Typically, drought contingency plans are developed to meet essential demands during drought conditions that may be more severe than the historic critical period.

d. Irrigation Shortages. For irrigation water supply, shortages are usually acceptable under some conditions. Often the desired quantity can be reduced considerably during the less critical parts of the growing season without great crop loss. Also, if there is a reliable forecast of a drought, the irrigator may be able to switch to a crop having less water requirements or use groundwater to make up the deficit. Shortages of 10% usually have negligible economic effect, whereas shortages as large as 50% are usually disastrous.

e. Water Supply for Navigation and Low-Flow Augmentation. In designing a reservoir to supply water for navigation or low-flow augmentation, the amount and duration of shortages are usually much more important than the frequency of the shortages. Small shortages might only require rescheduling of fully-loaded vessels, whereas, large shortages might stop traffic altogether. The same thing is true for such purposes as fish and wildlife where one large shortage during the spawning season, for example, could have serious economic effects for years to come.

f. Effects of Shortages. Each project purpose should be analyzed carefully to determine what the effects of shortages will be. In many cases, this will be the criterion that determined the ultimate amount of reservoir storage needed for water supply and low-flow water management.

12-9. Shortage Index.

a. Definition.

(1) A general approach to shortage definition is to use a shortage index. The shortage index is equal to the sum of the squares of the annual shortages over a 100-year period when each annual shortage is expressed as a ratio to the annual requirements, as shown below:

$$SI = \frac{100 \sum_{i=1}^{i=N} \left[\frac{S_A}{D_A} \right]^2}{N} \quad (12-3)$$

where:

- SI = shortage index.
- N = number of years in routing study.
- S_A = annual shortage (annual demand volume minus annual volume supplied).
- D_A = annual demand volume.

(2) This shortage index reflects the observation that economic and social effects of shortages are roughly proportional to the square of the degree of shortage. For example, a shortage of 40% is assumed to be four times as severe as a shortage of 20%. Similarly, as illustrated in Table 12-1, shortages of 50% during 4 out of 100 years are assumed four times as severe as shortages of 10% during 25 out of 100 years.

Table 12-1. Illustration of Shortage Index.

Shortage Index	No. of Annual Shortages per 100 Years	Annual Shortage (S_A/D_A) in %
1.00	100	10
1.00	25	20
1.00	4	50
0.25	25	10
0.25	1	50

(3) The shortage index has considerable merit over shortage frequency alone as a measure of severity because shortage frequency considers neither magnitude nor duration. The shortage index can be multiplied by a constant to obtain a rough estimate of associated damages.

b. Additional Criteria Needed. There is a definite need for additional criteria delineating shortage acceptability for various services under different conditions. These criteria should be based on social and economic costs of shortages in each individual project study, or certain standards could be established for the various services and conditions. Such criteria should account for degree of shortage as well as expected frequency of shortage.

12-10. General Study Procedures.

a. Water Supply. After alternative plans for one or more water supply reservoirs have been established, the following steps can be followed in performing hydrologic studies required for each plan:

(1) Obtain all available daily and monthly streamflow records that can be used to estimate historical flows at each reservoir and diversion or control point. Compute monthly flows and adjust as necessary for future conditions at each pertinent location. A review of hydrologic data is presented in Chapter 5 of this EM.

(2) Obtain area-elevation or storage-elevation data on each reservoir site to be studied and compute storage capacity curves. Determine maximum practical reservoir stage from physiographic and cultural limitations.

(3) Estimate monthly evapotranspiration losses from each site and monthly lake evaporation that is likely to occur if the reservoir is built.

(4) Determine seasonal patterns of demands and total annual requirements for all project purposes, if applicable, as a function of future time. Synthesize stochastic variations in demands, if significant.

(5) Establish a tentative plan of operation, considering flood risk management and reservoir sedimentation as well as conservation requirements, and perform an operation study based on runoff during the critical period of record. The Reservoir System Simulation (HEC-ResSim) software package can be used for this purpose (HEC 2013).

(6) Revise the plan of operation, including sizes of various facilities, as necessary to improve accomplishments and perform a new operation study. Repeat this process until a near-optimum plan of development is obtained.

(7) Depending on the degree of refinement justified in the particular study, test this plan of development using the entire period of estimated historical inflows and as many sequences of synthetic streamflow and demands as might be appropriate. Methods for developing synthetic flow sequences are presented in Chapter 12, Hydrologic Frequency, of EM 1110-2-1415, Hydrologic Frequency Analysis.

(8) Modify the plan of development to balance yields and shortages for the maximum overall accomplishment of all project objectives.

b. Hydroelectric Power. The study procedure for planning, designing, and operating hydroelectric developments can be summarized as follows:

(1) From an assessment of the need for power generation facilities, obtain information concerning the feasibility and utility of various types of hydroelectric projects. This assessment could be made as part of the overall study for a given project or system, or it could be available from a national, regional, or local power authority.

(2) From a review of the physical characteristics of a proposed site and a review of other project purposes, if any, develop an estimate of the approximate amount of space that will be available for either sole- or joint-use power storage. This determination and the needs developed under step (1) will determine whether the project will be a storage, run-of-river, or pumped-storage power project and whether it will be operated to supply demands for peaking or for baseload generation.

(3) Using information concerning seasonal variation in power demands obtained from the assessment of needs, and knowing the type of project and the approximate storage usable for power production, determine the historical critical hydroperiod by review of the historical hydrologic data.

(4) An estimate of potential hydroelectric energy for the assumed critical hydroperiod is made using Equation 11-3 or 11-4. If the energy calculated from this equation is for a period other than the basic marketing contract period (usually a calendar year), the potential energy during the critical hydroperiod should be converted to a firm or minimum quantity for the contract period (minimum annual or annual firm in the case of a calendar year).

(5) Because the ability of a project to produce hydroelectric energy and peaking capacity is a complex function of the head, the streamflow, the storage, and operation for all other purposes, the energy estimate obtained in step (4) is only an approximation. Although this approximation is useful for planning purposes it should be verified by simulating the operation of the project for all authorized purposes by means of a sequential routing study. Chapter 11 of this EM provides methods for performing and analyzing sequential routing studies.

(6) From the results of detailed sequential routing studies, the data necessary for designing power-generating units and power-related facilities of the project should be developed. The design head and design output of the generating units, approximate powerhouse dimensions, approximate sizes of water passages, and other physical dimensions of the project depend on the power installation.

(7) Operation rules for the project must be developed before construction is completed. These rules are developed and verified through sequential routing studies that incorporate all of the factors known to affect the project's operation. For many multipurpose projects, these operation rules are

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relatively complex and require the use of computerized simulation models to facilitate the computations involved in the sequential routing studies.

(8) If the project is to be incorporated into an existing system or if the project is part of a planned system, system operation rules must be developed to define the role of the project in supplying energy and water to satisfy the system demands. These operation rules can be tested using the Hydrologic Engineering Center Reservoir System Simulation (HEC-ResSim) software package.

CHAPTER 13

Reservoir Sedimentation

13-1. Introduction. “The ultimate destiny of all reservoirs is to be filled with sediment” (Linsley et al. 1992). The question is how long will it take? Also, as the sediment accumulates with time, will it adversely affect water control goals?

a. Transport Capacity. A reservoir changes the hydraulics of flow by forcing the energy gradient to approach zero. This results in a loss of transport capacity with the resulting deposition. The smaller the particles, the farther they will move into the reservoir before depositing. Some may even pass the dam. Deep reservoirs are not fully mixed and are conducive to the formation of density currents.

b. Sediment Deposits. The obvious consequence of sediment deposits is a depletion in reservoir storage capacity. Figure 13-1 illustrates components of sediment deposition in a deep reservoir. The volume of sediment material in the delta and the main reservoir depends on the inflowing water and sediment, reservoir geometry, project operation and life among other things. The reservoir will eventually fill with sediment as the delta continues to develop with time.

13-2. Reservoir Deposition.

a. Total Available Sediment. The first step is to estimate the total sediment that will be available for deposition during the design life of the project. Required data include design life of the reservoir, reservoir capacity, water and sediment yield from the watershed, the composition of the sediment material, and the unit weight of sediment deposits. With this information, the trap efficiency can be determined.

b. Trap Efficiency. Trap efficiency is the percent of inflowing sediment that remains in the reservoir. Some proportion of the inflowing sediment leaves the reservoir through the outlet works. The proportion remaining in the reservoir is typically estimated based on the trap efficiency. Trap efficiency is described in Section 3-7(a) of EM 1110-2-4000, Sediment Investigations of Rivers and Reservoirs, and the calculations are described in an appendix therein. The efficiency is primarily dependent on the detention time, with the deposition increasing as the time in storage increases.

c. Existing Reservoirs. Existing reservoirs are routinely surveyed to determine sediment deposition, and resulting loss of storage. Section 5-30 and Appendix K of EM 1110-2-4000 describe the USACE program. This historic deposition data can be useful for checking computed estimates. Sediment Deposition in U.S. Reservoirs (Summary of Data Reported 1981-85) provides data on reservoir locations, drainage areas, survey dates, reservoir storage capacities, ratios of reservoir capacities to average annual inflows, specific weights (dry) of sediment deposits,

and average annual sediment accumulation rates (USGS 1992). Reservoirs are grouped by drainage basins.

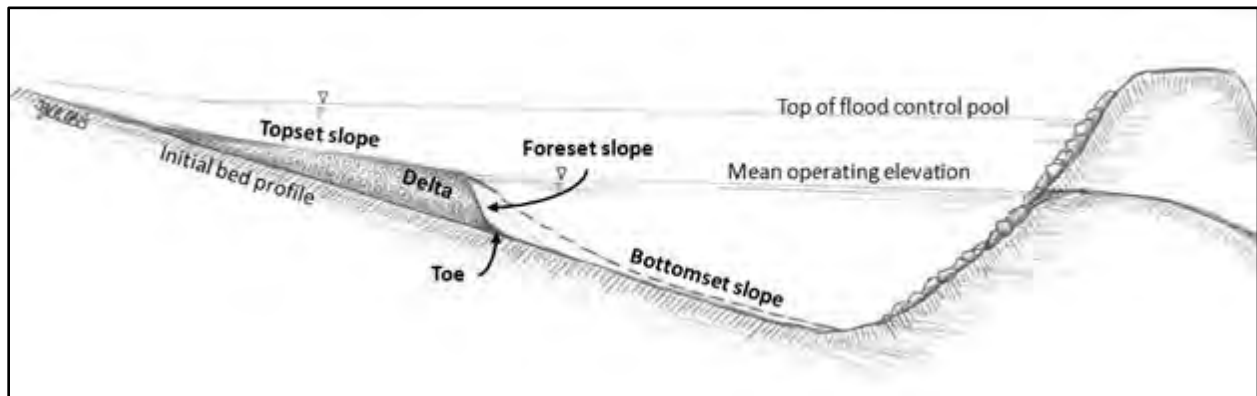


Figure 13-1. Conceptual Deposition in Deep Reservoirs.

13-3. Distribution of Sediment Deposits in the Reservoir. The planning or design of a reservoir requires an analysis to determine how sediment deposits will be distributed in the reservoir. This is a difficult aspect of reservoir sedimentation because of the complex interaction between hydraulics of flow, reservoir operating policy, inflowing sediment load, and changes in the reservoir bed elevation. The traditional approach to analyzing the distribution of deposits has relied on empirical methods, all of which require a great deal of simplification from the actual physical problem.

a. **Main Channel Deposition.** Conceptually, deposition starts in the main channel. As flow enters a reservoir, the main channel fills at the upstream end until the elevation is at or above the former overbank elevations on either side. Flow then shifts laterally to one side or the other, but present theory does not predict the exact location. During periods of high-water elevation, deposition will move upstream. As the reservoir is drawn down, a channel is cut into the delta deposits and subsequent deposition moves material farther into the reservoir. The lateral location of the channel may shift from year to year, but the hydraulic characteristics will be similar to those of the natural channel existing prior to impounding the reservoir. Vegetation will cover the exposed delta deposits and thus attract additional deposition until the delta takes on characteristics of a floodplain.

b. **Sediment Diameters.** The diameter of sediment particles commonly transported by streams ranges over five log cycles. Generally, the coarse material will settle first in the outer reaches of the reservoir followed by progressively finer fractions farther down toward the reservoir dam. Based on this depositional pattern, the reservoir is divided into three distinct regions: top-set, fore-set, and bottom-set beds. The top-set bed is located in the upper part of the reservoir and is largely composed of coarse material or bed load. While it may have a small effect on the reservoir storage capacity, it could increase upstream stages. The fore-set region represents

the live storage capacity of the reservoir and comprises the wash load. The bottom-set region is located immediately upstream of the dam and is primarily composed of suspended sediments brought from upstream by density currents. The region is called the reservoir dead storage and generally does not affect the storage capacity. Some of the finest material may not settle out and will pass through the dam. To calculate the volume of material that will deposit as a function of distance, grain size must be included as well as the magnitude of the water discharge and the operating policy of the reservoir.

c. Reservoir Shape. Reservoir shape is an important factor in calculating the deposition profile. For example, flow entering a wide reservoir spreads out, thus reducing transport capacity, but the path of expanding flow does not necessarily follow the reservoir boundaries. It becomes a 2-dimensional problem to calculate the flow distribution across the reservoir to approximate transport capacity and, therefore, the resulting deposition pattern. On the other hand, flow entering a narrow reservoir has a more uniform distribution across the section resulting in hydraulic conditions that are better approximated by 1-dimensional hydraulic theory.

d. Flood Waves. Flood waves attenuate upon entering a reservoir. Therefore, their sediment transport capacity decreases from two considerations: (1) a decrease in velocity due to the increase in flow area and (2) a decrease in velocity due to a decrease in water discharge resulting from reservoir storage. As reservoir storage is depleted by the sediment deposits in the delta, the impact of attenuation on transport capacity diminishes. The resulting configuration, therefore, is assumed to depend upon the first consideration, whereas, the time for delta development is influenced somewhat by the second consideration.

e. Flood-Pool Index Method. If flood risk management is a project purpose, the next level of detail in reservoir sedimentation studies is to divide the total volume of predicted deposits into that volume settling into the flood control pool and that volume settling in the remainder of the reservoir. The flood pool index method requires the depth of flood control pool, depth of reservoir, and the percent of time the reservoir water level is at or above the bottom of the flood control pool. Based on the index, the percent of sediment trapped in the flood control pool is estimated by a general empirical relationship. Appendix H of EM 1110-2-4000, Sedimentation Investigations of Rivers and Reservoirs, describes the index method and provides several other methods for estimating the distribution of sediment deposits in reservoirs. Chapter 5, Section IV, EM 1110-2-4000, provides an overview of levels of sedimentation studies and methods of analysis.

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CHAPTER 14

Spillways and Outlet Works

14-1. Function of Spillways and Outlet Works. Spillways and outlet works are necessary to provide capability to release an adequate rate of water from the reservoir to satisfy dam safety and water control management of the project. Sections 4-2 and 4-3 in EM 1110-2-3600, Management of Water Control Systems, provide general descriptions of types and operation requirements for spillways and outlet works, respectively.

a. Spillway Adequacy. While the outflow capability must be provided throughout the operational range of the reservoir, the focus of hydrologic studies is usually on the high flows and spillway adequacy. Dam failures have been caused by overtopping flows from improperly designed spillways or by insufficient spillway capacity. Ample capacity is of great importance for earthfill and rockfill dams, which are likely to be destroyed if overtopped; whereas concrete dams may be able to withstand moderate overtopping.

b. Spillway Classification. Spillways are ordinarily classified according to their most prominent feature, either as it pertains to their shape, location, or discharge channel. Spillways are often referred to as controlled or uncontrolled, depending on whether they are gated or ungated. EM 1110-2-1603, Hydraulic Design of Spillways, describes a variety of spillway types and provides hydraulic principles, design criteria, and results from laboratory and prototype tests.

c. Outlet Works. Outlet works serve to regulate or release water impounded by a dam. It may release incoming flows at a reduced rate, as in the case of a detention dam; divert inflows into canals or pipelines, as in the case of a diversion dam; or release stored water at such rates as may be dictated by downstream needs, evacuation considerations, or a combination of multiple-purpose requirements.

d. Outlet Structure Classification. Outlet structures can be classified according to their purpose, their physical and structural arrangement, or their hydraulic operation. EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works, provides information on basic hydraulics, conduits for concrete dams, and conduits for earth dams with emphases on flood control projects. Appendix IV of EM 1110-2-1602 provides an illustrative example of the computation of a discharge rating for outlet works.

e. Low-Level Outlets. Low-level outlets are provided to maintain downstream flows for all levels of the reservoir operational pool. The outlets may also serve to empty the reservoir to permit inspection, to make needed repairs, or to maintain the upstream face of the dam or other structures normally inundated.

f. Outlets as Flood Control Regulators. Outlet works may act as a flood control regulator to release waters temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows. In this case, the outflow capacity should be able to release channel capacity, or higher. The flood control storage must be evacuated as rapidly as safely possible, to maintain flood reduction capability.

14-2. Inflow Design Flood.

a. IDF Analyses. IDF analyses are performed to evaluate the adequacy of an existing spillway or to size a spillway. The IDF required in dam design is subject to the dam safety standards discussed in ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs. For a project involving high risk and consequences (i.e., Standard 1), the practice in the United States is to base the IDF on the PMP occurring over the upstream watershed. The PMP is based on the maximum conceivable combination of unfavorable meteorological events.

b. Dam Safety Standards. Flood hydrographs used in spillway design are dependent on the Dam Safety Standards of ER 1110-8-2. Refer to this ER for the appropriate IDF. Based on the risks and consequences associated with the site, the IDF could be the PMF, half PMF, or another magnitude of event determined from the base-safety condition.

14-3. Area and Capacity of the Reservoir.

a. Reservoir Capacity and Operations. Dam designs and reservoir operating criteria are related to the reservoir capacity and anticipated reservoir operations. The reservoir capacity and reservoir operations are used to properly size the spillway and outlet works. The reservoir capacity is a major factor in flood routings and may determine the size and crest elevation of the spillway. The reservoir operation and reservoir capacity allocations will determine the location and size of outlet works for the controlled release of water for downstream requirements and flood risk management.

b. Area-Capacity Tables. Reservoir area-capacity tables should be prepared before the final designs and specifications are completed. These area-capacity tables should be based on the best available topographic data and should be the final design for administrative purposes until superseded by a reservoir resurvey. To ensure uniform reporting of data for design and construction, standard designations of water surface elevations and reservoir capacity allocations should be used.

14-4. Routing the Inflow Design Flood.

a. Discharge Facilities. The facilities available for discharging inflow from the spillway design flood depend on the type and design of the dam and its proposed use. A single dam installation may have two or more of the following discharge facilities: uncontrolled overflow spillway, gated overflow spillway, regulating outlet, and power plant. With a reservoir full to the spillway crest at the beginning of the design flood, uncontrolled discharge will begin at once.

Surcharge storage is created when the outflow capacity is less than the inflow and the excess water goes into storage, causing the pool level to rise above spillway crest. The peak outflow will occur at maximum pool elevation, which should always be less, to some degree, than the peak inflow.

b. **Gated Spillway.** A gated spillway's main purpose is to maximize available storage and head, while at the same time limiting backwater damages by providing a high initial discharge capacity. In routing the IDF, an initial reservoir elevation at the normal full pool operating level is assumed. Operating rules for spillway gates must be based on careful study to avoid releasing discharges that would be greater than would occur under natural conditions before construction of the reservoir. By gate operation, releases can be reduced and additional water will be held in storage, which is called "induced surcharge storage." The release rates should be made in line with spillway gate regulation schedules developed for each gated reservoir. EM 1110-2-3600, Management of Water Control Systems, Section 4-5 describes induced surcharge storage and the development and testing of the regulation schedules.

c. **Surcharge Storage.** The important factor in the routing procedure is the evaluation of the effect of storage in the upper levels of the reservoir, surcharge storage, on the required outflow capacity.

d. **Wedge Storage.** Routing techniques divide storage computations into prism and wedge storage. These techniques always account for prism storage but simplified methods will often neglect wedge storage as its shape varies and is difficult to define. Prism storage accurately represents flood storage if the water surface is level. However, there will be a sloping water surface at the head of the reservoir due to backwater effects, and this condition creates the additional "wedge storage." Reservoir operation should be developed to account for wedge storage in instances where it represents a non-negligible proportion of available storage, and routing may have an impact on reservoir operation and required discharge. Wedge storage is particularly important for relatively small flood control pools. Refer to Chapter 9 of EM 1110-2-1417, Flood-Runoff Analysis, for more information.

e. **Drawdown.** If a reservoir is drawn down at the time of occurrence of the IDF, the initial increments of inflow will be stored with the corresponding reduction in ultimate peak outflow. Refer to ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs, for more information.

f. **Release Rates with Flood Warning.** Assuming a reservoir can be significantly drawn down in advance of the IDF by using a short-term flood warning system is generally not acceptable for several reasons. The volume that can be released is the product of the total rate of discharge at the dam times the warning time. Because the warning time is usually short, except on large rivers, the release rate must be the greatest possible without flood damage downstream. Even under the most favorable conditions, it is unlikely that the released volume will be significant, relative to the volume of the IDF.

14-5. Sizing the Spillway.

a. Storage and Spillway Capacity. In determining the best combination of storage and spillway capacity to accommodate the selected IDF, all pertinent factors of hydrology, hydraulics, design, cost, and damage should be considered. In this connection and when applicable, consideration should be given to the following factors:

- (1) The characteristics of the flood hydrograph.
- (2) The damage that would result if such a flood occurred without the dam.
- (3) The damage that would result if such a flood occurred with the dam in place.
- (4) The damage that would occur if the dam or spillway were breached.
- (5) Effects of various dam and spillway combinations on the probable increase or decrease of damages above or below the dam.
- (6) Relative costs of increasing the capacity of the spillways.
- (7) The use of combined outlet facilities to serve more than one function.

b. Outflow Characteristics. The outflow characteristics of a spillway depend on the particular device selected to control the discharge. These control facilities may take the form of an overflow weir, an orifice, a tube, or a pipe. Such devices can be unregulated, or they can be equipped with gates or valves to regulate the outflow.

c. Flood Routing. After a spillway control of certain dimensions has been selected, the maximum spillway discharge and the maximum reservoir water level can be determined by flood routing. Other components of the spillway can then be proportioned to conform to the required capacity and the specific site conditions, and a complete layout of the spillway can be established. Cost estimates of the spillway and dam can then be made. Estimates of various combinations of spillway capacity and dam height for an assumed spillway type, and of alternative types of spillways, will provide a basis for the selection of the most economical spillway type and the optimum relation of spillway capacity to the height of the dam.

d. Maximum Reservoir Level. The maximum reservoir level can be determined by routing the spillway design flood hydrograph using sequential routing procedures and the proposed operation procedures. This is a basic step in the selection of the elevation of the crest of the dam, the size of the spillway, or both.

e. Peak Rate of Inflow. Where no flood storage is provided, the spillway must be sufficiently large to pass the peak of the flood. The peak rate of inflow is then of primary interest,

and the total volume in the flood becomes less important. However, where a relatively large storage capacity above normal reservoir level can be made economically available by a higher dam, a portion of the flood volume can be retained temporarily in reservoir surcharge space, and the spillway capacity can be reduced considerably. If a dam could be made sufficiently high to provide storage space to impound the entire volume of the flood above normal storage level, theoretically, no spillway other than an emergency type would be required, provided the outlet capacity could evacuate the surcharge storage in a reasonable period of time in anticipation of a recurring flood. The maximum reservoir level would then depend entirely on the volume of the flood, and the rate of inflow would be of no concern. From a practical standpoint, however, relatively few sites will permit complete storage of the IDF by surcharge storage.

f. Overall Cost. The spillway length and corresponding capacity may have an important effect on the overall cost of a project because the selection of the spillway characteristics is based on an economic analysis. In many reservoir projects, economic considerations will necessitate a design using surcharge. The most economical combination of surcharge storage and spillway capacity requires flood routing studies and economic studies of the costs of spillway-dam combinations. Among the many economic factors that may be considered are damage due to backwater in the reservoir, cost-height relations for gates, and use in the dam of material excavated from the spillway channel. However, consideration must still be given to the minimum size spillway that must be provided for safety.

g. Comprehensive Study. The study may require many flood routings, spillway layouts, and spillway and dam estimates. Even then, the study is not necessarily complete because many other spillway arrangements could be considered. A comprehensive study to determine alternative optimum combinations and minimum costs may not be warranted for the design of some dams. Judgment on the part of the designer would be required to select for study only those combinations that show definite advantages, either in cost or adaptability. For example, although a gated spillway might be slightly cheaper than an ungated spillway, it may be desirable to adopt the latter because of its less complicated construction, its automatic and trouble-free operation, its ability to function without an attendant, and its less costly maintenance.

14-6. Outlet Works.

a. Definition. An outlet works consists of the equipment and structures that together release the required water for a given purpose or combination of purposes. Flows through river outlets and canal or pipeline outlets change throughout the year and may involve a wide range of discharges under varying heads. The accuracy and ease of control are major considerations and a great amount of planning may be justified in determining the type of control devices that can be best used. Note that if the outlet works are to be modeled in a computer software package to route the IDF, they must be designed to adequately accommodate the flow and tailwater conditions of the IDF. Refer to ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs, for more information.

b. Description. Usually, the outlet works consist of an intake structure, a conduit or series of conduits through the dam, discharge flow control devices, and an energy-dissipating device where required downstream of the dam. The intake structure includes a trash-rack, an entrance transition, and stop-logs or an emergency gate. The control device can be placed at the intake on the upstream face, at some point along the conduit and be regulated from galleries inside the dam, or at the downstream end of the conduit with the operating controls placed in a gate-house on the downstream face of the dam. When there is a power plant or other structure near the face of the dam, the outlet conduits can be extended farther downstream to discharge into the river channel beyond these features. In this case, a control valve may be placed in a gate structure at the end of the conduit.

c. Discharge. Discharges from a reservoir outlet works fluctuate throughout the year depending upon downstream water needs and reservoir flood control requirements. Therefore, impounded water must be released at specific regulated rates. Operating gates and regulating valves are used to control and regulate the outlet works flow and are designed to operate in any position from closed to fully open. Guard or emergency gates are designed to close if the operating gates fail, or where dewatering is desired to inspect or repair the operating gates.

d. Continuous Low-Flow Releases. Continuous low-flow releases are usually required to satisfy the needs of fish, wildlife and existing water rights downstream from the dam. When the low-flow release is small, one or two separate small bypass pipes, with high-pressure regulating valves, are provided to facilitate operations. Flood-regulating gates may be used for making low-flow releases when those low-flow releases require substantial gate openings (EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works).

e. Uses of an Outlet Works. An outlet works may be used for diverting the river flow or portion thereof during a phase of the construction period, thus avoiding the necessity for supplementary installations for that purpose. The outlet structure size dictated by this use rather than the size indicated for ordinary outlet requirements may determine the final outlet works capacity.

f. IDF. Reservoir regulating outlets should not be assumed operable during the occurrence of an IDF, unless they are specifically designed for such purpose. See ER 1110-8-2 for more information.

g. Intake Level. The establishment of the intake level is influenced by several considerations such as maintaining the required discharge at the minimum reservoir operating elevation, establishing a silt retention space, and allowing selective withdrawal to achieve suitable water temperature and/or quality. Dams that will impound waters for irrigation, domestic use, or other conservation purposes must have the outlet works intake low enough to be able to draw the water down to the bottom of the allocated storage space. Further, if the outlets are to be used to evacuate the reservoir for inspection or repair of the dam, they should be placed as low as practicable.

However, it is usual practice to make an allowance in a reservoir for inactive storage for silt deposition, fish and wildlife conservation, and recreation.

h. Elevation of Outlet Intake. Reservoirs become thermally stratified, and taste and odor vary between elevations. Therefore, the outlet intake should be established at the best elevation to achieve satisfactory water quality for the purpose intended. Downstream fish and wildlife requirements may determine the temperature at which the outlet releases should be made. M&I water use increases the emphasis on water quality and requires the water to be drawn from the reservoir at the elevation that produces the most satisfactory combination of odor, taste, and temperature. Water supply releases can be made through separate outlet works at different elevations if requirements for the individual water uses are not the same and the reservoir is stratified.

i. Energy-Dissipating Devices. The two types of energy-dissipating devices most commonly used in conjunction with outlet works on concrete dams are hydraulic jump stilling basins and plunge pools. On some dams, it is possible to arrange the outlet works in conjunction with the spillway to use the spillway-stilling device for dissipating the energy of the water discharging from the river outlets. Energy-dissipating devices for free-flow conduit outlet works are essentially the same as those for spillways.

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CHAPTER 15

Dam Freeboard Requirements

15-1. Basic Considerations.

a. Freeboard protects dam and embankment integrity from overflow and excessive wave overtopping caused by hydraulic modeling uncertainties, wave run-up and wind setup, and embankment settlement and subsidence. It is defined as the vertical distance between the crest of a dam and the maximum pool elevation of the IDF. Depending on the importance and potential hazard posed by the dam, the amount of freeboard will vary to maintain structural integrity and the estimated cost of damage repairs from overtopping. This chapter's focus is primarily on considerations of wind effects.

b. The contribution of wind to freeboard requirements is based on the probable wind characteristics (e.g., magnitude and direction) that are likely to occur coincident with an IDF event. In many situations, it is improbable that a high wind will occur simultaneously with a high pool level. However, this is not always the case as areas subjected to hurricanes and super storms may have a higher likelihood of high winds and high reservoir pools coinciding in time. Selection of appropriate wind characteristics can be based on statistical analysis of observed winds or stochastic modeling with appropriate consideration given to the joint probability of wind and reservoir stage.

c. Three basic considerations of wind effects are generally used in establishing freeboard allowance. These are wave characteristics, wind setup, and wave run-up (with consideration of overtopping). Other considerations to include in the estimation of the final freeboard include the settlement and subsidence of the structure foundation. Settlement and subsidence will not be discussed in this manual. Refer to EM 1110-1904, Settlement Analysis, for guidance on these geotechnical topics.

15-2. Wave Run-up and Wind Setup.

a. General. Wave run-up is the increase in water-level elevation at the point of interest (e.g., dam embankment) due to wave uprush on a structure above the still water pool elevation. Wind setup is the increase in water-level elevation at the point of interest due to the effects of the horizontal stress caused by wind along the reservoir surface. The point of interest is typically a dam embankment but it could be another location or structure. Figure 15-1 shows a conceptual drawing of wave run-up, wind setup, wave height and wave length.

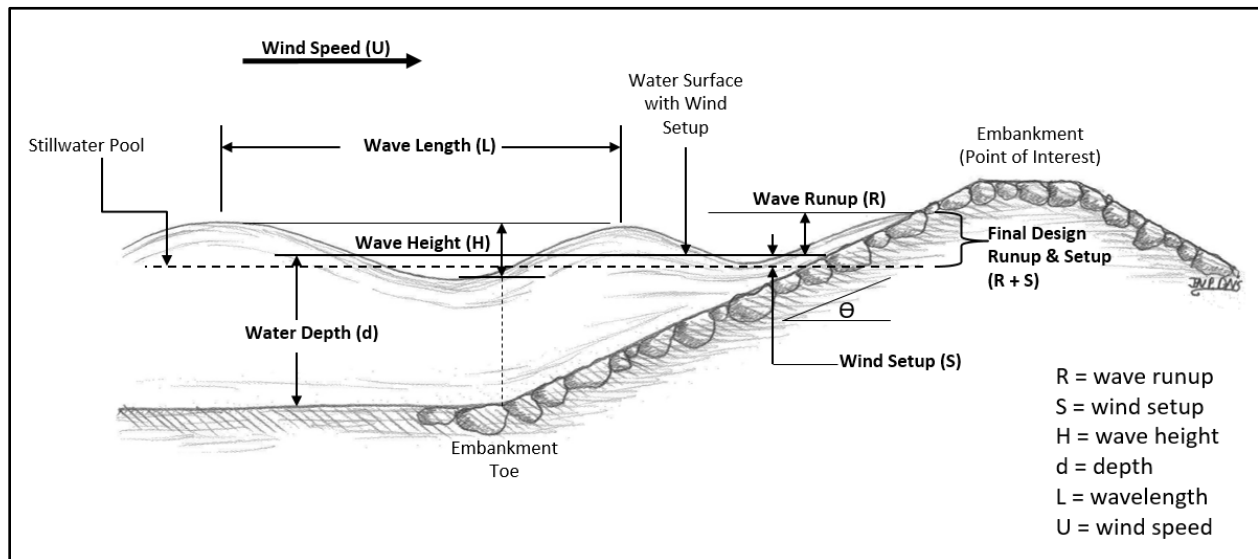


Figure 15-1. Wave Run-up and Wind Setup.

(1) Wave run-up and wind setup can be calculated at three main levels of detail depending on the risk involved in a project. These main levels include: hand calculations, the Automated Coastal Engineering System (ACES) software package, and advanced numerical models developed by the ERDC. The choice of the methodology depends on the level of risk inherent in the project, funding availability, Division guidance and the necessary design detail. Hand calculations may be used from feasibility to final design but they must be supported by documented references.

(2) The main purpose of a wave run-up and wind setup analysis is to size embankment riprap and (combined with an allowable overwash threshold) determine adequate dam freeboard. The deliverables of a wave run-up and wind setup analysis are: extreme wave height ($H_{2\%}$), wind setup (S), wave run-up (R), and freeboard requirements for wind effects ($S+R$). These values are determined through the calculation of a significant wave height (H_{m0} , H_s , $H_{1/3}$, or H_{33}) and its period (T_s). Differences between H_{m0} and H_s will be addressed later.

(3) The extreme wave height ($H_{2\%}$) is the wave height exceeded by 2% of the waves. This wave height is different from the significant wave height (H_s or H_{33}), which is the average wave height of the highest 1/3 of waves. Significant wave height also refers to the spectral wave height (H_{m0}). The spectral wave height is the energy-based significant wave height defined as four times the standard deviation of the surface elevation and is used in manuals like EM 1110-2-1100, Coastal Engineering Manual, and EurOtop (2007). The significant wave height (H_s) and the spectral wave height (H_{m0}) produce almost identical results in deep water conditions. In the case of shallow water conditions, H_s and H_{m0} can have a difference of 10-15%. All of these wave heights are measured from wave crest to wave trough at the location of the toe of the structure.

(4) In applications for inland reservoirs, it is necessary to give special consideration to the influences that reservoir surface configuration, surrounding topography, and ground roughness may have on wind velocities and directions over the water surface. The effects of shoreline irregularities on wave refraction and the influences of water depth on wave heights and must be taken into account. Although allowances can only be approximated, the estimates of wave and wind characteristics on inland reservoirs can be prepared with sufficient accuracy for engineering purposes.

(5) Appendix C contains detailed documentation on how to perform a wave run-up and wind setup analysis on a reservoir. Guidance using equations and figures from EM 1110-2-1100, Coastal Engineering Manual, is provided and a real-world example is shown to illustrate how to determine design parameters and use the guidance information provided. Appendix C shows the simplified hand-calculation methodology and sections are organized in the order recommended for calculation.

(6) Sources for additional information include the EurOtop Wave Overtopping of Sea Defenses and Related Structures Assessment Manual (EurOtop 2007) and EM 1110-2-1100, Coastal Engineering Manual (2014). Both these sources focus on coastal applications and do not address wave run-up and wind setup analysis on reservoirs directly. This is why Appendix C in this EM was included with more in-depth guidance on determining wind effects on inland reservoirs.

(7) Historic references drawn from include the Shore Protection Manual (USACE 1977 & 1984); ETL 1110-2-2211, Wave Runup and Wind Setup on Reservoir Embankments (1976) (Superseded by ETL 1110-2-305); ETL 1110-2-305, Determining Sheltered Water Wave Characteristics (1984) (Expired); EM 1110-2-1412, Storm Surge Analysis and Design Water Level Determinations (1986) (Superseded by EM 1110-2-1100); and EM 1110-2-1414, Water Levels and Wave Heights for Coastal Engineering Design (1989) (Superseded by EM 1110-2-1100). Note that historic superseded and expired publications are not endorsed by the USACE but are provided here to provide background and history. See the most recent approved USACE publications on the U.S. Army Corps of Engineers Official Publications website (USACE 2015b, Appendix B, Entry 29, p B-4).

(8) Wind speed data may be retrieved from a variety of sources; however, it is important for the wind speed data to be defined by its frequency, direction, and duration. For instance, building code wind speeds (ASCE) are typically defined as 3-seconds in duration (gusts) and hurricane wind speeds (NOAA) are typically defined as 1-minute in duration (maximum sustained). If a return frequency analysis is not performed on data records retrieved (period of record and measurement location are considerations, particularly in mountainous areas where micro-climates commonly exist), than conversion to different return frequencies of interest on standard data (e.g., ASCE and NOAA) may be performed using tabular conversion data from ASCE (ASCE 7-02) or other professional references. In addition to the wind speed, wind direction generally toward the dam or embankment needs to be considered within realm of feasibility in potentially causing or having an impact, given topographic and other physical limitations. One last parameter that needs to be considered with wind speed is total duration of the peak wind speed (adjusted for fetch length) that will be used for the

analysis. A generated frontal wind may have a much longer duration than that of a tropical/subtropical storm. Once a wave overtopping rate is determined, this duration is a multiplier to calculate the total volume of water that overtopped the embankment.

(9) The amount of overflow a structure can tolerate over time is predominantly based on volumes rate of flow, type of material (and cover if earthen) receiving the flow, and velocities achieved that can generate both significant erosional and or uplift damages. For earthen dams/embankments, research by the USDA Agricultural Research Service (USDA-ARS) is considerable. However, other references (e.g., EurOtop) also enjoy common usage. Limitations of other material types (e.g., concrete, articulated concrete matt-types, roller compacted concrete (RCC), turf reinforcement matt, etc.) are readily available in common industry literature.

(10) The amount of wave overtopping a structure can tolerate over time is based in similar manner as that of overflow. For structures other than earthen (e.g., concrete, RCC) limitations are similar to overflow as well. However, the distribution of waves (irregular, i.e., wind generated) may be of primary importance as the maximum wave heights can generate a considerably higher flow rate than the often calculated average overtopping rate. This factor likely has a larger impact on uplift damages than erosion due to the limited time of exposure the higher waves have over the duration of the storm event. For earthen structures, the degree of soil cohesiveness and quality of cover (grass density, root mass and root length) are large factors in determining resistance to erosional damages. For cohesive soils, common references are EurOtop and others based on research by the Netherlands. Another source often referenced is the Design of Reinforced Grass Waterways by Construction Industry Research and Information Association (CIRIA 1987). For non-cohesive soils, there is little information; however, the research in this field is growing as the United States and other countries have to manage aging infrastructure beyond the expected design life cycle. A reference for sandy soils can be sourced from research performed by Colorado State University and Dr. Steven Hughes (Full-Scale Wave Overwash Resiliency Testing of Dikes and Embankments on Florida Sandy Soils, draft October 2013 and final January 2014). The draft version focuses more on the accumulative work aspect of erosion and the final version focuses more on grass characteristics that have an effect on resistance to erosional forces. Current application of these two methods is that the design of dams revolves around accumulative work and the effectiveness of maintenance revolved around grass characteristics. Both documents can be located at or retrieved from the Jacksonville District.

(11) Determination of an acceptable overtopping rate during design is optimally dependent on the structure maintenance costs or what can be afforded with frequency versus cost for a higher freeboard design. For example, increasing the freeboard of an embankment with a long reach can add a significant cost to the design and maintenance for rare overtopping events and design for overtopping by more frequent design storm events may be more acceptable, becoming an optimized design decision. For non-cohesive earthen embankments, an example of acceptable damage may be loss of cover (grass) and material to the point of progressive erosion (e.g., headcutting or crest width reduction); whereby, failure may become imminent if wave overtopping continues for a significantly longer duration of time.

b. The following are general steps for simplified hand calculations of wave run-up and wind setup on inland reservoirs. Refer to Appendix C for details and examples.

- (1) Set the pool elevation at the maximum level of the IDF pool for dam freeboard calculations.
- (2) Estimate the reservoir effective fetch (F_e)².
- (3) Collect site-specific or general wind speed data.
- (4) Adjust the wind speed data to the standard 10 meter (33 foot) elevation.
- (5) Adjust overland wind speeds to represent over water wind speeds.
- (6) Adjust over water wind speeds for atmospheric conditions for reservoirs with effective fetches over 10 miles.
- (7) Determine wind speeds at several durations (e.g., 1, 5, 10, 60, 180, and 360 minutes).
- (8) Assume deep water reservoir conditions and determine the design wind speeds and durations by plotting the site wind speed-durations determined previously against the fetch-limited wind speed duration curve.
- (9) Determine the average depth (d) of the reservoir along the effective fetch length.
- (10) Calculate wind setup (S) with a depth-appropriate method.
- (11) Determine the significant wave heights and their periods (H_s and T) using wave forecasting equations. Many wave forecasting methods consider deep water and shallow water conditions separately. Assume deep water conditions initially unless the pool is obviously shallow (below ~ 5 feet).
- (12) Calculate the theoretical deep water wavelength (L_o) using the wave period of the significant wave height and determine if the assumption of deep water conditions holds. Do this by comparing the average depth (d) along the effective fetch (F_e) to $\frac{1}{2}$ of the theoretical deep water wavelength (L_o). If d/L_o is greater than 0.5, the reservoir is considered to be deep. If conditions are deep, keep the design wind speeds and durations and the significant wave heights and their periods as solutions and move to the next step. If conditions are shallow, use the shallow-water forecasting methods to estimate new design wind speeds and durations and significant wave heights and periods.
- (13) Calculate the extreme wave height (typically $H_{2\%}$).

² Wind fetch is the unobstructed length of water over which a given win blows in a single direction. The effective fetch is a modified wind fetch that accounts for physical restrictions on wave growth at the project site (See Appendix C for details).

(14) Determine the slope of the dam embankment, spillway, or shoreline over which the wave will pass.

(15) Determine the wave run-up (R) on the embankment or at the point of interest.

(16) The total raise in water height above the still water elevation at the embankment/point of interest is the wave run-up plus the wind setup (R+S).

(17) Calculate the amount of wave overtopping to determine if it exceeds structure tolerances.

15-3. Freeboard Allowances for Wave Action.

a. A freeboard estimate is required to establish allowances needed to provide for wave action that is likely to affect various project elements. These project elements include:

- (1) Main embankment of the dam, and supplemental dike sections.
- (2) Levees that protect areas within potential flowage limits of the reservoir.
- (3) Highway and railroad embankments that intersect the reservoir limits.
- (4) Structures located within the reservoir area.
- (5) Shoreline areas that are subject to adverse effects of wave action.

b. Freeboard allowances for wave action on embankments and structures within reservoir flowage limits must also take into account the following:

(1) Wave action effects must be taken into account in establishing design grades and slope protection measures for highway, railroad, levee, and other embankments that intersect or border a reservoir. The design of operating structure, boat docks, recreational beaches, and shoreline protection measures at critical locations involves the consideration of wave characteristics and frequencies under a range of conditions. Estimates of wave characteristics affecting the design of these facilities can have a major influence on the adequacy of design and costs of relocations required for reservoir projects, and in the development of supplemental facilities.

(2) The freeboard reference level selected as a base for estimating wave effects associated with each of the several types of facilities referred to above will be governed by considerations associated with the particular facility. Otherwise, procedures generally as described with respect to the determination of freeboard allowances for dams should be followed, and stage hydrographs and related wave run-up elevations corresponding to the selected wind criteria should be prepared. However, the free-

board reference level and coincident wind velocity-duration relations selected for these studies usually correspond to conditions that would be expected with moderate frequency, instead of the rare combinations assumed in estimating the height of a dam required for safety.

(3) In estimating the effects of wave action on embankments and structures, the influences of water depths near the facility should be carefully considered. If the shallow depths prevail for substantial distances from the embankment or structure under study, wave effects may be greatly reduced from those prevailing in deep water areas. However, under the same circumstances, wind setup can increase, leading to higher water levels and increased wave energy. Additionally, facilities located where sudden reductions in water depths cause waves to break are likely to be subjected to greater dynamic forces than would be imposed on similar facilities located in deep water. This consideration is particularly important in estimating the effects waves may have on bridge structures that are partially submerged under certain reservoir conditions.

(4) Systematic analyses of wave effects associated with various key locations along embankments that cross or border reservoirs provide a practical basis for varying design grades and erosion protection measures to establish the most economical plan to meet pertinent operational and maintenance standards.

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CHAPTER 16

Dam-Break Analysis

16-1. Introduction.

a. USACE Policy. It is the policy of the Corps of Engineers to design, construct, and operate dams safely as required by ER 1110-8-2, Inflow Design Floods for Dams and Reservoirs and engineering ethics. When a dam is breached, catastrophic flash flooding occurs as the impounded water escapes through the gap into the downstream channel. Usually, the response time available for warning is much shorter than that for precipitation-runoff floods, so the potential for loss of life and property damage is much greater.

b. Potential Hazard Evaluation. A potential hazard evaluation is the basis for selecting the performance standards to be used in dam design or in evaluating existing dams. ER 1110-8-2 provides dam safety standards with respect to the appropriate selection of an IDF.

c. Safety Design. Safety design includes studies to determine areas that would be flooded during the IDF and in the event of dam failure. The areas downstream from the project should be evaluated to determine the need for land acquisition, flood plain management, or other methods to prevent major damage. Information should be developed and documented on the risks of flooding to provide guidance for downstream releases. Reference ER 1110-2-1451, Acquisition of Lands Downstream from Spillways for Hydrologic Safety Purposes, for guidance in acquiring land downstream of spillways.

d. National Dam Safety Act. The potential for catastrophic flooding due to dam failures in the 1960s and 1970s brought about passage of the National Dam Safety Act, Public Law 92-367. The Corps of Engineers became responsible for inspecting U.S. Federal and non-Federal dams, which met the size and storage limitations of the act, to evaluate their safety. The USACE inventoried dams; surveyed each State and Federal agency's capabilities, practices, and regulations regarding the design, construction, operation, and maintenance of the dams; developed guidelines for the inspection and evaluation of dam safety; and formulated recommendations for a comprehensive national program.

e. Flood Emergency Documents. Emergency Actions Plans (EAPs) are formal documents that identify potential emergency conditions (either dam failure or large spillway releases) at a dam and specifies preplanned actions to be followed to minimize loss of life and property damage. Refer to ER 1110-2-1156, Safety of Dams – Policy and Procedures, for more information. The Federal Emergency Management Agency (FEMA) also has publications including Federal Guidelines for Emergency Action Planning for Dams (FEMA Publication No. P-64 (FEMA 2015, Appendix B, Entry 5, p B-1).

f. Risk Assessment Procedures for Existing Dams. Risk is comprised of three elements: the likelihood of loading, probability of failure, and the consequences of failure. Risk assessment, risk management, and risk communication are the three components for which the dam safety risk-informed program is managed.

(1) Risk assessment. Risk assessments include the recognition of safety issues, the evaluation of remediation options, and the assessment of the effectiveness of repairs. Assessments are performed on a continuous basis as risk is subject to change. From ER 1110-2-1156, Safety of Dams – Policy and Procedures, risk assessment is a broad term that encompasses a variety of analytic techniques that are used in different situations, depending upon the nature of the risk, the available data, and the needs of decision makers. It is a systematic, evidence-based approach for quantifying and describing the nature, likelihood, and magnitude of risk associated with the current condition and the values of the risk resulting from a changed condition due to some action.

(2) Risk management. Once a problem is recognized, action is initiated to reduce the level of risk. Management decisions fall into routine and non-routine activities. Routine activities include instrumentation monitoring, inspections, and assessments. Non-routine activities occur during a special event such as an unusually large flood and may include lowering reservoir pools, stockpiling emergency material, updating emergency action plans and inundation maps, and installing additional instrumentation. Risk management according to ER 1110-2-1156 is the process of identifying a problem and initiating action to identify, evaluate, select, implement, monitor and modify actions taken to alter levels of risk, as compared to taking no action. The purpose of risk management is to choose and implement technically sound and integrated actions to reduce risks after the costs are considered of each increment of risk reduction. Environmental, social, cultural, ethical, political and legal considerations all factor into the decision made on how much cost will be incurred for each increment of risk reduction (how safe is safe enough?). Risk management for dams includes short-term Interim Risk Reduction Measures, long-term structural risk reduction measures, and strengthening recurrent ER 1110-2-1156 activities - such as monitoring and surveillance, emergency action planning, operations and maintenance, and staff training.

(3) Risk communication. Risk communication is the two-way exchange of potential hazard information with the public and other project stakeholders. Communication must be accurate, timely, and clear so individuals can understand the risks and make decisions accordingly. From ER 1110-2-1156, risk communication is the open, two-way exchange of information and opinion about hazards and risks leading to a better understanding of the risks and better risk management decisions. Risk communication is integrated into the assessment and management processes. It is not a task that occurs only after decisions have been made. Risk communication ensures that the decision makers, other stakeholders and affected parties understand and appreciate the process of risk assessment and in so doing can be fully engaged in and responsible for risk management.

(4) For more information visit the U.S. Army Corps of Engineers Dam Safety Program website at the address shown in Appendix B-3.

16-2. Dam Breach Analysis.

a. Causes of Dam Failures. Dam failures can be caused by overtopping a dam due to insufficient spillway capacity during large inflows to the reservoir, by seepage or piping through the dam or along internal conduits, slope embankment slides, earthquake damage and liquefaction of earthen dams from earthquakes, or landslide-generated waves within the reservoir. Hydraulics, hydrodynamics, hydrology, sediment transport mechanics, and geotechnical aspects are all involved in breach formation and eventual dam failure. An internal USACE reference for basic dam breach analysis is the Modeling, Mapping, and Consequences Center (MMC) Standard Operating Procedures (SOP) for Dams (USACE 2015c), which is available upon request from the MMC. HEC Research Document 13, Flood Emergency Guidelines for Corps Dams (HEC 1980), and which lists the prominent causes as follows:

- (1) Earthquake.
- (2) Landslide.
- (3) Extreme storm.
- (4) Piping.
- (5) Equipment malfunction.
- (6) Structural damage.
- (7) Foundation failure.
- (8) Sabotage.

b. Dam Breach Characteristics. The breach is the opening formed in the dam when it fails. Despite the fact that the main modes of failure have been identified as piping or overtopping, the actual failure mechanics are not well understood for either earthen or concrete dams. In previous attempts to predict downstream flooding due to dam failures, it was usually assumed that the dam failed completely and instantaneously. These assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam-break flood waves. The presumptions are somewhat appropriate for concrete archtype dams, but they are not suitable for earthen dams and concrete gravity-type dams.

(1) Earthen dams, which exceedingly outnumber all other types of dams, do not tend to completely fail, nor do they fail instantaneously. Once a developing breach has been initiated, the discharging water will erode the breach until either the reservoir water is depleted or the breach resists further erosion. Breach widths for earthen dams are usually much less than the total length of the

dam as measured across the valley. Also, the breach requires a finite interval of time for its formation through erosion of the dam materials by the escaping water. Piping failures occur when initial breach formation takes place at some point below the top of the dam due to erosion of an internal channel through the dam by escaping water. As the erosion proceeds, a larger and larger opening is formed. This is eventually hastened by caving-in of the top portion of the dam. Refer to HEC Technical Document 39, Using HEC-RAS for Dam Break Studies (Brunner 2014), and the MMC Standard Operating Procedures (SOP) for Dams (USACE 2015c) for more information.

(2) Concrete gravity dams also tend to have a partial breach as one or more monolith sections formed during the dam construction are forced apart by the escaping water. The time for breach formation is in the range of a few minutes.

(3) Poorly constructed earthen dams and coal-waste slag piles that impound water tend to fail within a few minutes and have average breach widths in the upper range or even greater than those for the earthen dams mentioned above.

c. Dam Breach Parameters. The parameters of failure depend on the dam and the mode of failure. For flood hydrograph estimation, the breach is modeled assuming weir conditions, and the breach size, shape, and timing are the important parameters. The larger the breach opening and the shorter the time to total failure, the larger the peak outflow. Refer to Technical Paper 39 (TP-39), A Method for Analyzing Effects of Dam Failures in Design Studies (Brunner 2014), and the USACE MMC SOP for Dams (USACE 2015c) for more information. There are three basic approaches used to determine possible breach sizes and times.

(1) The first approach uses statistically derived regression equations, like those formulated by MacDonald and Langridge-Monopolis (1984) and by Froelich (1987). Both sets of equations are based on actual data from dozens of historic dam failures. The MacDonald and Langridge-Monopolis study was based on data from 42 constructed earth- and rockfill dams. The Froelich study included data from constructed and landslide-formed earthen dams. Both studies resulted in a set of graphs and equations that can be used to predict the approximate size of the breach and the time it takes for the breach to reach its full size.

(2) The second approach is a physically based computer model. Several computer models are available including NWS-Breach, developed by Dr. Danny Fread (1989) for the NWS.

(3) The third approach is the Simplified Physical Breach method in HEC-RAS for dam-break studies. This approach was developed primarily for levee breaches and is less applicable to dams. Erosion rates are recommended to be estimated from past levee and dam breaches. Refer to Technical Paper 39 (TP-39), A Method for Analyzing Effects of Dam Failures in Design Studies (Brunner 2014), for more information on using HEC-RAS for dam-break studies. This document is available on the HEC Publications website (Appendix B, Entry 14, p B-2).

16-3. Dam Failure Hydrograph.

a. Flow Hydrograph. The flow hydrograph from a breached dam may be computed using traditional methods for flow routing through a reservoir and downstream channel. The reservoir routing approach is the same as routing for the spillway design flood, described in Chapter 14 of this EM. Generally, a short time step is required because the breach formation and resulting reservoir outflow change rapidly with time.

b. Routing Methods. The choice between hydraulic and hydrologic routing depends on many factors, including the nature of available data and accuracy required. The hydraulic method is the more accurate method of routing the unsteady flow from a dam failure flood through the downstream river. This technique simultaneously computes the discharge, water surface elevation, and velocity throughout the river reach. Chapter 9 of EM 1110-2-1417, Flood-Runoff Analysis, describes the routing methods and applicability of routing techniques. Chapter 5 of EM 1110-2-1416, River Hydraulics, describes unsteady flow computations.

c. Geometry and Surface Area. The geometry and surface area of the reservoir can also affect the choice of method. For very narrow and long reservoirs where the dam is relatively large, the change of water level at the failed dam is rapid, and the unsteady flow method is useful. However, for very large reservoirs where the dam is small compared to the area of the lake, the change in water level is relatively slow and the storage routing method (Modified Puls) is economical in developing the failure hydrograph. Small time steps are required for both methods due to the rapid change in water level.

d. Height of Downstream Water. The height of the water downstream of a dam (tailwater) also affects the outflow hydrograph in a failure analysis. It also affects the formation or nonformation of a bore in front of the wave.

e. Deriving the Peak Outflow. By assuming a rectangular cross section, zero bottom slope, and an instantaneous failure of a dam, the peak outflow can be derived by the mathematical expression originally developed by St. Venant, as follows:

$$Q_{\max} = \frac{8}{27} W_b \sqrt{g} Y_o^{3/2} \quad (16-1)$$

where:

Y_o = the initial depth.

W_b = the width of the breach.

G = the gravity coefficient.

water depth y , just downstream of the dam is:

$$y = \frac{4}{9}Y_o \quad (16-2)$$

f. Equation 16-2 is applicable only for relatively long and narrow rectangular channels where the dam is completely removed. Refer to Technical Paper 39 (TP-39), A Method for Analyzing Effects of Dam Failures in Design Studies (Brunner 2014).

g. Potential for Overtopping. The Hydrologic Engineering Center River Analysis System (HEC-RAS, HEC 2018a) and Hydrologic Modeling System (HEC-HMS, HEC 2017b) can be used to determine the potential for the overtopping of dams by runoff resulting from various proportions of the PMF. Of the two models, HEC-RAS allows for more sophisticated reservoir (outlet works) operation that would more accurately represent real-world operation. HEC-RAS is better able to address situations where the level-pool routing assumption is not valid and is the more appropriate of the two models for simulating breaches in earthen dams caused by overtopping.

h. Peak Flow Values. With several calculations of theoretical flood peaks from assumed breaches, peak flow values may seem either too low or too high. One way of checking the reasonableness of the assumption is to compare the calculated values with historical failures. An envelope of estimated flood peaks from actual dam failures prepared by the Bureau of Reclamation is a good means of comparing such values. HEC Research Document No. 13, Figure 2, provides an envelope of experienced outflow rates from breached dams, as a function of hydraulic depth.

16-4. Dam-Break Routing.

a. Dam-Break Flood Hydrographs. Dam-break flood hydrographs are dynamic, unsteady flow events. Therefore, the preferred routing approach is to use a full unsteady flow routing model. The HEC-HMS computer model provides the capability to compute and route the IDF and compute the breach and resulting hydrograph, but its channel routing is limited to hydrologic methods. The most appropriate HEC-HMS channel-routing approach is the Muskingum-Cunge option. This option uses a simple cross section plus reach slope and length to define a routing reach. No downstream backwater effects are considered. If simplified representations of the downstream river reaches are acceptable, an adequate routing may be obtained.

b. St. Venant Equations. The St. Venant equations apply to gradually varied flow with a continuous profile. If features that control or interrupt the water surface profile exist along the main stem of the river or its tributaries, internal boundary conditions are required. These features include dams, bridges, roadway embankments, etc. If the structure is a dam, the total discharge is the sum of spillway flow, flow over the top of the dam, gated spillway flow, flow through turbines, and flow through a breach, should a breach occur. The spillway flow and dam overtopping are treated as weir flow, with corrections for submergence. The gated outlet can represent a fixed gate or one in which the gate opening can vary with time. These flows can also

be specified by rating curves that define discharge passing through the dam as a function of upstream water surface elevation.

c. **HEC-RAS Unsteady Flow Modeling.** The unsteady portion of the HEC-RAS software package has dam-break routing capabilities. HEC-RAS can be used to route the outflow hydrograph computed in an HEC-HMS runoff-dam-break model or can be used with internal dam-break routines. Both programs can read and write hydrographs using the HEC Data Storage System, HEC-DSSVue (HEC 2009). Refer to Technical Document 39 (TD-39), A Method for Analyzing Effects of Dam Failures in Design Studies (HEC 2014), available on the HEC website for more guidance (Appendix B, Entry 14, p B-2). HEC-RAS also has the ability to perform two-dimensional (2D) hydrodynamic flow routing within the unsteady flow analysis option of HEC-RAS. The program can solve either the 2D Saint Venant equations (with momentum additions for turbulence and Coriolis effects) or the 2D Diffusion Wave equations. Refer to the Hydrologic Engineering Center Computer Program Documentation (CPD-68A) titled HEC-RAS River Analysis System 2D Modeling User's Manual Version 5.0 (HEC 2015) for more information.

16-5. Inundation Mapping.

a. **Preparation of Maps.** To evaluate the effects of dam failure, maps should be prepared delineating the area that would be inundated in the event of failure. Land uses and significant development or improvements within the area of inundation should be indicated. Refer to ER 1110-2-1156, Safety of Dams – Policy and Procedures, and the USACE internal MMC Standard Operating Procedures (SOP) for guidance.

b. **Evaluation of Hazard Potential.** To assist in the evaluation of hazard potential, the degree of occupancy and hazard potential within the delineated areas should be considered. Loss of life potential and damages to structures and contents are the primary hazard categories evaluated.

(1) To perform analyses relating to potential loss of life and damages to structures and their contents, the Hydrologic Engineering Center Flood Damage Reduction Analysis and Flood Impact Analysis software packages are commonly used (HEC-FDA and HEC-FIA, respectively).

(2) The HEC-FDA software package is used primarily to compute expected or equivalent annual damages to structures and contents. The software, which works by using H&H data in conjunction with a study area structure inventory to compute an EAD, is commonly used in various USACE projects. It implements the risk analysis procedures described in EM 1110-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies. It also computes the annual exceedance probability and conditional nonexceedance probability as required for levee certification.

(3) The HEC-FIA software package uses multiple metrics to compute potential loss of life in the event of a flood event or dam or levee failure. The software is typically used to perform dam and levee failure scenario analysis to support consequence estimates to determine the risk posed or prevented by USACE projects.

c. Hazard Potential for Recreation Areas. The hazard potential for affected recreation areas varies greatly, depending on the type of recreation offered, intensity of use, communications facilities, and available transportation. The potential for loss of life may be increased where recreationists are widely scattered over the area of potential inundation because they would be difficult to locate on short notice.

d. Industries and Utilities. Many industries and utilities requiring substantial quantities of water are located on or near rivers or streams. Flooding of these areas and industries, in addition to causing the potential for loss of life, can damage machinery, manufactured products, raw materials and materials in process of manufacture, plus interrupt essential community services.

e. Least Hazard Potential. Rural areas usually have the least hazard potential. However, the potential for loss of life exists, and damage to large areas of intensely cultivated agricultural land can cause high economic loss.

f. Evacuation Plans.

(1) Evacuation plans should be prepared and implemented by the local jurisdiction controlling inundation areas. The assistance of local civil defense personnel, if available, should be requested in preparation of the evacuation plan. State and local law enforcement agencies usually will be responsible for the execution of much of the plan and should be represented in the planning effort. State and local laws and ordinances may require that other state, county, and local government agencies have a role in the preparation, review, approval, or execution of the plan. Before finalization, a copy of the plan should be furnished to the dam agency or owner for information and comment.

(2) Evacuation plans will vary in complexity depending on the type and degree of occupancy in the potentially affected area. The plans may include delineation of the area to be evacuated; routes to be used; traffic control measures; shelter; methods of providing emergency transportation; special procedures for the evacuation and care of people from institutions such as hospitals, nursing homes, and prisons; procedures for securing the perimeter and for interior security of the area; procedures for the lifting of the evacuation order and reentry to the area; and details indicating which organizations are responsible for specific functions and for furnishing the materials, equipment, and personnel resources required. Refer to Chapter 16 of ER 1110-2-1156, Safety of Dams – Policy and Procedures, for more information on creating Emergency Action Plans and the USACE MMC SOP for Dams (USACE 2015c) for more information. The MMC Standard Operating Procedures (SOP) document is available upon request from the MMC.

CHAPTER 17

Channel Capacity Studies

17-1. Introduction.

a. General. Channel capacity studies tend to focus on high flows. Flood operations for a reservoir will require operational downstream targets for nondamaging flows when excess water must be released. Nondamaging capacities may be defined at several locations, and the target flow may be defined at several levels. There may be lower targets for small flood events and, under extreme flood situations, the nondamaging target may cause some minor damage. Also, the nondamaging flow target may vary seasonally and depend on floodplain land use.

b. Withstanding Release Rates. Channel capacity is also concerned with the capability of the channel to withstand reservoir release rates. Of particular concern is the reach immediately downstream from the reservoir. High release rates for hydropower or flood risk management could damage channel banks and cause local scour and channel degradation.

c. Channel Capacity. While flood operation may focus on nondamaging flood capacity, planning studies usually require stage-discharge information over the entire range of expected operations. Also, low-flow targets may be concerned with maintaining minimum downstream flow depth for navigation, recreation, or environmental goals. Channel capacity studies typically provide information on safe channel capacity and stage-discharge (rating) curves for key locations.

17-2. Downstream Channel.

a. Downstream Channel Erosion. Water flowing over a spillway or through a sluiceway is capable of causing severe erosion of the stream bed and banks below the dam. Consequently, the dam and its appurtenant works must be so designed that harmful erosion is minimized. The outlet works for a dam usually require an energy-dissipating structure. The design may vary from an elaborate multiple-basin arrangement to a simple head wall design, depending on the number of conduits involved, the erosion resistance of the exit channel bed material, and the duration, intensity, and frequency of outlet flows. A stilling basin may be provided for outlet works when such downstream uses as navigation, irrigation, and water supply, require frequent operation or when the channel immediately downstream is easily eroded. Chapter 4 of EM 1110-2-3600, Management of Water Control Systems, provides a general discussion of energy dissipaters for spillways and outlet works, respectively.

b. Adequate Capacity. The channel downstream should have adequate capacity to carry most flows from reservoir releases. After the water has lost most of its energy in the energy-dissipating devices, it is usually transported downstream through the natural channel to its destination points. With the expected release rates, the channel should be able to resist excessive erosion and scour, and have a large enough capacity to prevent downstream flooding except during large floods.

c. River Surveys. River surveys of various types provide the basic physical information on which river engineering planning and design are based. Survey data include information on the horizontal configuration (planform) of streams; characteristics of the cross sections (channel and overbank); stream slope; bed and bank materials; water discharge; sediment characteristics and discharge; water quality; and natural and cultural resources.

d. Evaluating Bank Stability. It is essential to understand the complex historic pattern of channel migration and bank recession of the stream and the relationship of channel changes to streamflow in the evaluation of bank stability. Studies of bank caving, based on survey data and aerial photographs, provide information on the progressively shifting alignment of a stream and are basic to laying out a rectified channel alignment. The concepts and evaluation procedures presented in Stability of Flood Control Channels (USACE 1990) are applicable to the channel capacity evaluation.

e. Interrupted Sediment Flow. Dam and reservoir projects tends to interrupt the flow of sediment, which can have a significant impact on the downstream channel capacity. If the project is relatively new, the affect may not be seen by evaluating historic information or current channel conditions. The future channel capacity will depend on the long-term trends in aggradation and degradation along the river. General concepts on sediment analysis are presented in Chapter 9 of this EM. EM 1110-2-400, Sediment Investigations of Rivers and Reservoirs, is the primary reference for defining potential problems and analyses procedures.

f. Downstream Floodplain Land Use. Channel capacity also depends on the long-term trends in downstream floodplain land use. While it is not a hydrologic problem, channel capacity studies should recognize the impact of floodplain encroachments on what is considered the non-damaging channel capacity. Anecdotal history has shown that many USACE projects are not able to make planned channel-capacity releases due to development and encroachments downstream.

17-3. Stream Rating Curve.

a. Stage-Discharge Relationship. The relationship between stage and discharge—the “rating” at a gauging station—is based on field measurements with a curve fitted to plotted data of stage versus discharge. For subcritical flow, the stage-discharge relationship is controlled by the stream reach downstream of the gauge; for supercritical flow, the control is upstream of the gauge. The stage-discharge relationship is closely tied to the rate of change of discharge with time, and the rating curve for a rising stage can be different from that for the falling stage in alluvial rivers.

b. Tailwater Rating Curve. The tailwater rating curve, which gives the stage-discharge relationship of the natural stream below the dam, is dependent on the natural conditions along the stream and ordinarily cannot be altered by the spillway design or by the release characteristics. Degradation or aggradation of the river below the dam, which will affect the ultimate stage-discharge conditions, must be recognized in selecting the tailwater rating curves to be used for design. Usually, river flows that approach the maximum design discharges have never occurred,

and an estimate of the tailwater rating curve must either be extrapolated from known conditions or computed on the basis of assumed or empirical criteria. Thus, the tailwater rating curve at best is only approximate, and factors of safety to compensate for variations in tailwater must be included in dependent designs. Tailwater elevations for various conditions are also important and required for the structural stability analyses of concrete gravity dams and geotechnical studies as well as assessments of earthen embankment dams.

c. Extrapolation. Extrapolation of rating curves is necessary when a water level is recorded below the lowest or above the highest gauged level. Where the cross section is stable, a simple method is to extend the stage-area and stage-velocity curve and, for given stage values, take the product of velocity and cross section area to give discharge values beyond the stage values that have been gauged. Generally, water surface profiles should be computed to develop the rating beyond the range of observed data.

d. Rating Curve Shifts. The stage-discharge relationship can vary with time, in response to degradation, aggradation, or a change in channel shape at the control section, deposition of sediment causing increased approach velocities in a weir pond, vegetation growth, or ice accumulation. Shifts in rating curves are best detected from regular gauging and become evident when several gaugings deviate from the established curve. Sediment accumulation or vegetation growth at the control will cause deviations that increase with time, but a flood can flush away sediment and aquatic weed and cause a sudden reversal of the rating curve shift.

e. Flow Magnitude and Bed Material. Stream bed configuration and roughness in alluvial channels are a function of the flow magnitude and bed material. Bed forms range from ripples and dunes in the lower regime (Froude number < 1.0) to a smooth plane bed, to antidunes with standing waves (bed and water surface waves in phase) and with breaking waves and, finally, to a series of alternative chutes and pools in the upper regime as the Froude number increases.

f. Upper and Lower Rating Portions. The large changes in resistance to flow that occur as a result of changing bed roughness affect the stage-discharge relationship. The upper portion of the rating is relatively stable if it represents the upper regime (plane bed, transition, standing wave, or antidune regime) of bed form. The lower portion of the rating is usually in the dune regime, and the stage-discharge relationship varies almost randomly with time and season. Continuous definition of the stage-discharge relationship at low flow is a very difficult problem, and a mean curve for the lower regime is frequently used for gauges with shifting control.

g. Break up of Surface Material. In gravel-bed rivers, a flood may break up the armoring of the surface gravel material, leading to general degradation until a new armoring layer becomes established and ratings tend to shift between states of quasi-equilibrium. It may then be possible to shift the rating curve up or down by the change in the mean-bed level, as indicated by plots of stage and bed level versus time.

h. Ice. Ice at the control section may also affect the normal stage-discharge relationship. Ice effects vary with the quantity and the type of ice (surface ice, frazil ice, or anchor ice). When ice forms a jam in the channel and submerges the control or collects in sufficient amounts between the control and the gauge to increase resistance to flow, the stage-discharge relationship is affected; however, ice may form so gradually that there is little indication of its initial effects. Surface ice is the most common form and affects station ratings more frequently than frazil ice or anchor ice. The major effect of ice on a rating curve is due to backwater and may vary from day to day.

17-4. Water Surface Profiles.

a. Appropriate Methods. For most channel capacity studies, water surface profiles will be computed to develop the required information. Given the technical concerns described in the preceding section on rating curves, the selection of the appropriate method requires some evaluation of the physical system and the expected use of the information. The modeling methods are described in Chapter 8 of this EM and are presented in EM 1110-2-1416, River Hydraulics. While steady-flow water surface profiles are used in a majority of profile calculations, the unsteady flow aspects of reservoir operation or the long-term effects of changes in sediment transport may require the application of methods that capture those aspects.

b. Further Information. The USACE, and other agencies, have accumulated considerable experience with river systems. Appendix D of EM 1110-2-1416 titled, River Modeling – Lessons Learned, provides an overview of technical issues and modeling impacts that apply to profile calculations. Stability of Flood Control Channels (USACE 1990) provides case examples of stream stability problems, causes, and effects. While the focus is not on reservoirs, the experience reflects the high flow conditions that are a major concern with reservoir operation. EM 1110-2-4000, Sedimentation Investigations of Rivers and Reservoirs, provides procedures for problem assessment and modeling. All of these documents should be reviewed prior to formulating and performing technical studies.

CHAPTER 18

Real Estate and Right-of-Way Studies

18-1. Introduction.

a. General. This chapter provides guidance on the application of real estate acquisition requirements for reservoir projects. Previous chapters discussed land acquisition floods and their water surface profiles.

b. Related Guidance. Real estate requirements associated with feasibility reports, General Design Memoranda, and Real Estate Design Memoranda are set forth in Title 43 Code of Federal Regulations (CFR) Part 8 and other applicable real estate guidance and regulations including ER 405-1-12, the Real Estate Handbook. Real estate requirements associated with the acquisition of lands downstream from spillways are set forth in ER 1110-2-1451, Acquisition of Lands Downstream from Spillways for Hydrologic Safety Purposes and other applicable guidance and regulations.

18-2. Real Estate Acquisition Policies for Reservoirs.

a. Basic Policies. See Army Regulation 405-10, Real Estate Acquisition of Real Property and Interests. Also, land acquisition policies of the Department of the Army governing acquisition of land for reservoir projects are published in ER 405-1-12, Real Estate Handbook, Title 43 CFR Part 8 and other applicable guidance and regulations. The CFR contains information specific to the acquisition of land for reservoirs in the following sections: 8.0 Acquisitions of land for reservoir projects, 8.1 Land for reservoir construction and operation, 8.2 Additional lands for correlative purposes, 8.3 Easements, 8.4 Blocking out, 8.5 Mineral rights, and 8.6 Building.

b. Considering Factors. Factors such as estimated frequency, depth, and duration of occurrences, probable accuracy of estimates, and relocation costs will be taken into consideration. Projected impacts to lands may also rise to the level of a taking under the Fifth Amendment to the U.S. Constitution. A Takings Analysis by a qualified attorney, discussed in 18-4 below, may be necessary. Sufficient land should be acquired to meet the requirements of 43 CFR Part 8 and other applicable real estate regulations guidance, as well as the Fifth Amendment to the U.S. Constitution takings, to ensure that impacts would not result from floods up to the magnitude of the SPF. In such circumstances, however, consideration may be given to easements rather than fee acquisition for select sections if found to be in the public interest and legally sufficient for the proposed project purposes. However, when the project design provides a high-level spillway, the crest of which for economy of construction is considerably higher than the storage elevation required to regulate the reservoir inflow design flood (IDF), the upper level of fee acquisition will normally be at least equal to the top elevation of spillway gates or crest elevation of ungated spillway, and may exceed this elevation if necessary to conform with other criteria prescribed.

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18-3. Acquisition of Lands Downstream from Spillways for Hydrologic Safety Purposes. Real estate interests must be obtained for downstream areas where spillway discharges create or significantly increase a hazardous condition, or where projected impacts to lands may also rise to the level of a taking under the Fifth Amendment to the U.S. Constitution. Refer to ER 1110-2-1451, Acquisition of Lands Downstream from Spillways for Hydrologic Safety Purposes, for more information.

18-4. Takings Analysis. Individual property ownership in the studied area should be evaluated through a comparison of pre-project and post-project flooding. If additional flooding appears to be induced by the proposed project, the increase should be assessed in terms of frequency, velocity, depth, and duration. A qualified attorney should conduct a Takings Analysis that assesses whether the expected induced flooding would rise to the level of a compensable taking under the U.S. Constitution's Fifth Amendment. Such Takings Analysis constitutes a privileged legal opinion, which may not be shared without the permission of the Chief Counsel (CECC-ZA).

APPENDIX A

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APPENDIX B

Websites

1. CE-QUAL-W2 Model

<http://www.cee.pdx.edu/w2/>

Portland State University CE-QUAL-W2 Hydrodynamic and Water Quality Model

2. Dam Safety Program

<http://www.usace.army.mil/Missions/CivilWorks/DamSafetyProgram.aspx>

U.S. Army Corps of Engineers Dam Safety Program

3. Engineering Research and Development Center (ERDC) Library

<http://acwc.sdp.sirsi.net/client/default/>

Engineering Research and Development Center Library

A source for additional CE-QUAL-W2 documentation

4. Extreme Storm Database

<https://maps.crrel.usace.army.mil/apex/f?p=765:2>

U.S. Army Corps of Engineers Extreme Storm Database

A source for extreme storm information including total rainfall, storm area, dew points, etc.

5. FEMA Website

<https://www.fema.gov/media-library/assets/documents/3357>

Federal Emergency Management Agency website

A source for emergency action plans.

6. GIS Data from the USGS

<http://nationalmap.gov/viewer.html>

USGS National Map Viewer and Download Platform

A source of GIS data.

7. GIS Land Cover Data

<http://www.mrlc.gov/nlcd2006.php>

2006 National Land Cover Database for the Conterminous United States.

8. GIS Land Cover Definitions

<http://landcover.usgs.gov/classes.php>

National Land Cover Data Class Definitions.

9. GIS NEXRAD Precipitation Grids

<http://dipper.nws.noaa.gov/hdsb/data/nexrad/nexrad.html>

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NEXRAD gridded precipitation for 1995 to the present.

10. GIS PRISM Precipitation Grids

<http://www.prism.oregonstate.edu/>

PRISM precipitation grids developed by Oregon State.

11. GIS River Data

<http://www.nohrsc.noaa.gov/gisdatasets/>

National Operational Hydrologic Remote Sensing Center

A source for river and stream coverages for the United States.

12. GIS gSSURGO and Other Data

<http://datagateway.nrcs.usda.gov/>

U.S. Department of Agriculture (USDA) and National Resources Conservation Service (NRCS)

A source for gSSURGO soil's data, digital elevations models, National Hydrology Dataset streamlines, etc.

13. GIS SSURGO and STATSGO Data

<http://soildatamart.nrcs.usda.gov/>

U.S. Department of Agriculture (USDA) and National Resources Conservation Service (NRCS)

A source for SSURGO and STATSGO soil's data.

14. Hydrologic Engineering Center (HEC)

<http://www.hec.usace.army.mil/>

Hydrologic Engineering Center (HEC)

A source of USACE models, manuals, technical documents, and expertise.

15. National Bridge Inventory (NBI)

<http://www.fhwa.dot.gov/bridge/nbi.cfm>

U.S. Department of Transportation Federal Highway Administration Bridges and Structures Inventory.

16. National Inventory of Dams (NID)

<http://geo.usace.army.mil/pgis/f?p=397:1:0>

U.S. Army Corps of Engineers database of national dams.

17. NCEI Climate Data Images and Publications System

<http://www.ncdc.noaa.gov/IPS/>

National Centers for Environmental Information (NCEI) Images and Publications System (IPS)

Location of Local Climatological Data and Climatological Data publications.

18. NCEI Meteorological Data

<http://www.ncdc.noaa.gov/>

National Centers for Environmental Information (NCEI)

The primary source of meteorological data in the United States.

19. NOAA Advanced Hydrologic Prediction Service

<http://water.weather.gov/ahps/>

National river conditions at stream gauges through an interactive map.

20. NOAA Atlas 14 Precipitation Frequency Data Server

<http://hdsc.nws.noaa.gov/hdsc/pfds/>

NOAA National Weather Service Hydrometeorological Design Studies Center Precipitation Frequency Data Server (PFDS).

Point precipitation frequency data available through interactive maps for the United States.

21. NOAA GIS-Based Map Interface

<https://www.ncdc.noaa.gov/customer-support/tools>

[A source for climatic data including hourly wind speeds.](#)

22. NOAA Hydrometeorological Reports (HMRs)

<http://www.nws.noaa.gov/oh/hdsc/>

NOAA's National Weather Service Hydrometeorological Design Studies Center

A source of Hydrometeorological Reports (HMRs) used to quantify the probable maximum precipitation.

23. NOAA National Operational Hydrologic Remote Sensing Center (NOHRSC)

<http://www.nohrsc.noaa.gov/>

National Weather Service National Operational Hydrologic Remote Sensing Center (NOHRSC)

Source for interactive snow maps of the United States.

24. NOAA National Operational Model Archive & Distribution System (NOMADS)

<http://nomads.ncdc.noaa.gov/>

A source of meteorological model data.

25. NOAA Next Generation Radar (NEXRAD) Gridded Precipitation Data

<http://dipper.nws.noaa.gov/hdsb/data/nexrad/nexrad.html>

National Oceanic Atmospheric Administration archive of River Forecast Center Operational NEXRAD Data.

26. NOAA Storm Event Database

<http://www.ncdc.noaa.gov/stormevents/>

National Oceanic Atmospheric Administration Storm Event Database.

27. NWS Upper-Air Data

<http://weather.noaa.gov/fax/barotrop.shtml>

National Weather Station's (NWS) Telecommunication Operations Center

A source for publications of standard barotropic level fax charts and upper-air soundings.

28. Probable Maximum Precipitate (PMP)

www.wmo.int

World Meteorological Organization

Location of the "Manual on Estimation of Probable Maximum Precipitation (PMP)." This is a source for the estimation of site-specific probable maximum precipitates.

29. USACE Publications

<http://www.publications.usace.army.mil/>

U.S. Army Corps of Engineers engineering manuals, regulations, and other references

30. USEPA National Service Center for Environmental Publications (NSCEP)

<http://www.epa.gov/nscep/>

Source for information of drinking water standards, methods, and emerging contaminants

31. USGS National Hydrologic Dataset

<http://nhd.usgs.gov/data.html>

National Hydrologic Dataset (NHD)

Source of hydrography for GIS processing.

32. USGS Publications Warehouse

<http://pubs.er.usgs.gov/>

A source of past USGS publications.

33. USGS Stream Gage Data

<http://waterdata.usgs.gov/nwis>

U.S. Geological Survey (USGS) National Water Information System

A source of streamflow records for the United States.

34. USGS StreamStats

<http://water.usgs.gov/osw/streamstats/>

USGS regional regression equations for ungauged streams

35. USGS Techniques of Water-Resources Investigations Reports

<http://pubs.usgs.gov/twri/>

Publications for planning and conducting specialized work in water-resources investigations

APPENDIX C

Wave Run-up and Wind Setup Guidance and Examples

C-1. Wave Run-up and Wind Setup Guidance and Examples. This appendix discusses guidance for the wave run-up and wind setup simplified calculations on reservoirs in depth. Two examples are provided at the end of the guidance as well as nomograms for wave forecasting that can be used to check values calculated from the provided equations. EM 1110-2-1100, Coastal Engineering Manual, is the main source of the equations in this appendix.

C-2. Effective Fetch.

a. Effective wind fetch (F_e) for wave run-up. The characteristics of wind-generated waves are influenced by the distance wind moves over the water surface in the “fetch” direction. The generally narrow irregular shoreline of inland reservoirs will have lower waves than an open coast because there is less water surface to be acted upon by the wind. The method to compensate for the reduced water surface for an enclosed body of water is the estimation of an effective fetch.

b. Effective fetch (F_e) is the radial average distance that wind moves over a reservoir surface leading to the point of interest that creates the largest wave run-up. This fetch may be centered on a wind direction (i.e., SW, WSW, W etc.) or it may not depending on engineering judgment of what conditions will produce the highest wave run-up at the point of interest and if available wind speeds are directional. The fetch selected as the effective fetch is not always the longest distance across the reservoir. Wind speeds and their directions are also important and may result in a fetch of shorter length creating a larger wave height.

(1) Effective fetch is estimated by drawing centerlines along applicable wind directions from the point of interest to the far shore. Nine radials separated by 3-degrees and spanning 12 degrees from each side of the centerline are drawn, creating a total fan of 24 degrees. The lengths of the radials are averaged for each fetch to determine the effective fetch length. This procedure is shown in using the Fort Peck embankment (Figure C-1).

(2) The longest of the average effective fetches centered on a wind direction will likely produce the largest waves. However, wind speeds are also extremely important and the magnitudes of directional wind speed data should also be considered when these data are available. It is often necessary to consider several effective fetches and their wind speeds in analysis to determine the most critical wave height.

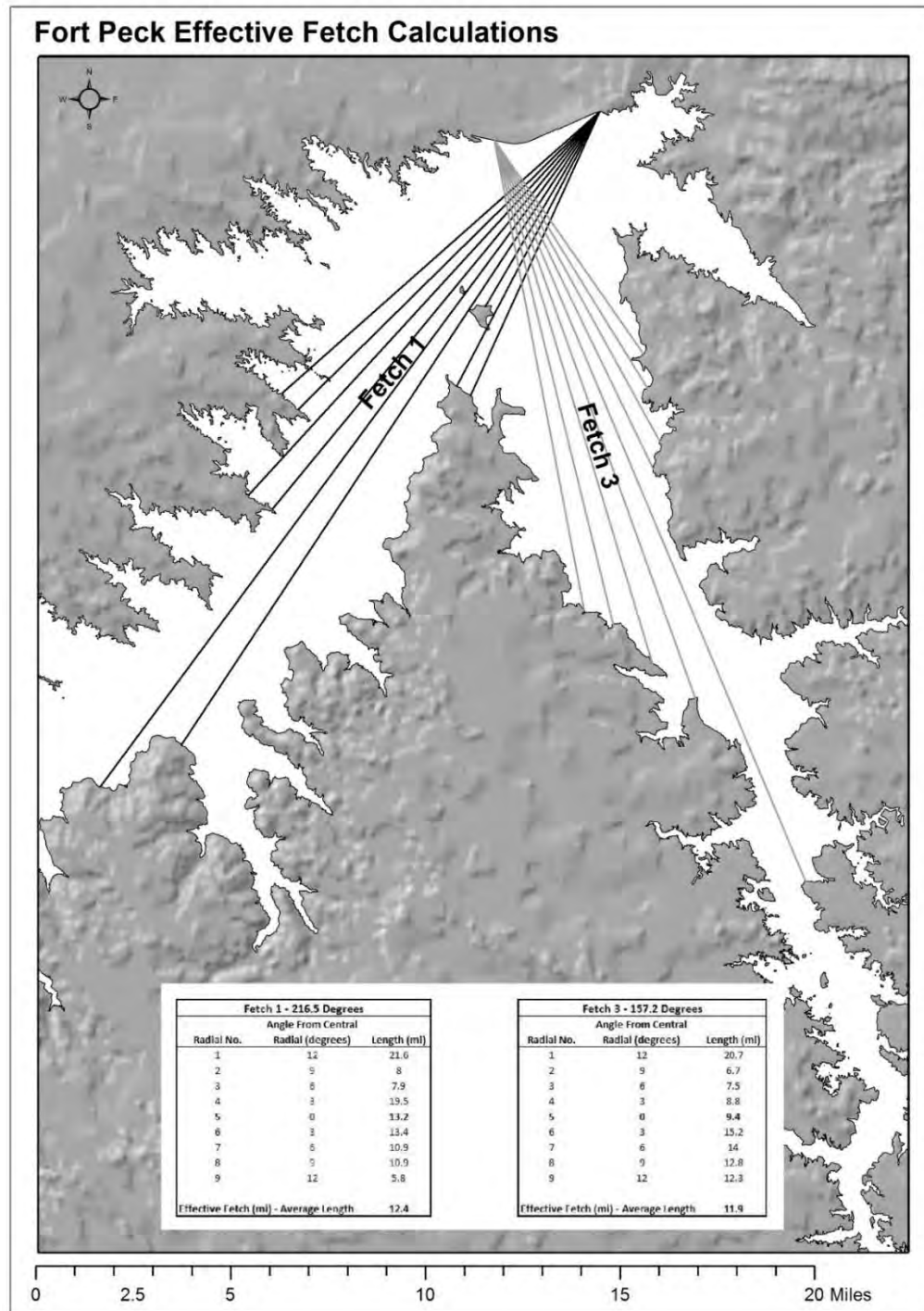


Figure C-1. Effective Fetch Calculations for Fort Peck, MT Embankment.

C-3. Wind Speeds.

a. General.

(1) Tropical storms (hurricanes) and tornadoes produce some of the most violent windstorms in the United States. Hurricane wind characteristics may affect reservoir projects located near Atlantic and Gulf coastlines, but winds associated with tornadoes are of short duration and not applicable to the determination of freeboard allowances for wave action.

(2) In mountainous regions, the flow of air is influenced by topography as well as meteorological factors. These “orographic” wind effects, when augmented by critical meteorological patterns, may produce high wind velocities for relatively long periods of time. Therefore, they should be given special consideration in estimating wave action in reservoirs located in mountainous regions.

(3) In areas not affected by major topographic influences, air movement is generally the result of horizontal differences in pressure, which in turn are due primarily to large-scale temperature differences in air masses. Wind velocities and durations associated with these meteorological conditions, with or without major influences of local topography, are of major importance in estimating wave characteristics in reservoirs.

(4) Isovel patterns can be used to estimate wind velocities and directions near a water surface at successive intervals of time such as when a windstorm passes the area. Sequence relations can represent wind velocities at, say, one-half hour intervals during periods of maximum winds, and one-hour or longer intervals thereafter. The “isovel” lines connect points of equal wind speeds, resembling elevation contour maps. Wind directions are indicated by arrows. EM 1110-2-1100, Section II-5-5 describes types of storms and the storm surge generation process. Figure II-5-23 shows an example wind isovel pattern and pertinent parameters for a hurricane.

(5) If wind velocity over a particular fetch remains constant and the fetch is long enough, wave heights will progressively increase until a limiting maximum value called a “fully-developed wave height” is attained. This fully-developed wave height is dependent on fetch distance, wind velocity, wind duration, and in shallow water reservoirs possibly by water depth. Accordingly, wind velocity-duration relations applicable to effective reservoir fetch areas are needed for use in computing wave characteristics in reservoirs.

b. Site-specific wind speed data.

(1) Site-specific wind speed data are preferred in analysis as they capture local effects such as orographic influences. Data on actual windstorms of record are maintained at many NCEI stations. Index values, such as the fastest mile, 1-min average or 5-min average velocities, with direction indications, are usually presented in climatological data publications. Wind data are summarized in the “Local Climatological Data” and “Climatological Data” publications available on the NCEI IPS (NOAA 2013c). Winds can be estimated using direct measurement or synoptic weather charts.

(2) Wind velocity data and their exceedances are sometimes available at U.S. Army Corps of Engineers project sites from past studies, even Standard Project Storms (SPS) with wind and or Standard Project Hurricanes (SPH) may have been previously determined.

(3) Some data collected by other agencies and private observers may be available in published or unpublished form. However, information regarding wind velocities sustained for several hours or days is not ordinarily published in detail.

(4) Special studies are usually required to determine wind velocity-duration relations applicable to specific effective fetch areas involved in wave computations. Basic records for such studies are usually available from the NCEI website. Some summaries of wind velocities over relatively long periods of time have been published by various investigators, and others may be available in project reports related to water resources development.

c. Generalized wind speed data.

(1) General all-directional wind statistics may be used in cases where better data are not available and where studies are preliminary. A source of this generic data is the Coastal Engineering Manual, EM 1110-2-1100 (2014). Figure II-8-7 provides a map of fastest-mile speeds (1-minute duration) at 33 feet (10 meters) with an annual probability of 0.02 (50-year return period). This figure is reproduced in Figure C-2 of this document.

(2) Generalized wind velocity-duration relations are considered to be fairly representative of maximum values that are likely to prevail over a reservoir in largely a single direction for periods up to 6 hours (excluding projects located in regions that are subject to severe hurricanes or orographic wind-flow effects). Special studies of wind characteristics associated with individual project areas should be made when determinations of unusual importance, or problems requiring consideration of wind durations exceeding 6 hours, are involved.

d. Elevation adjustments. All wind information should be adjusted to the standard level of 10 meters (33 feet). In the case of most recorded data, this adjustment has already been made, but it should always be checked as it has a large effect on the final deliverables. If the collected data are within 12 meters (39 feet) of the 10-meter (33-feet) level and neutral atmospheric conditions exist, then Equation II-2-9 from EM 1110-2-1100 may be used. This equation is reproduced as Equation C-1 in this document and is known as the “1/7th power expression.” If a more accurate estimation of wind speed at the 10-meter level is needed, Figure II-2-6 in EM 1110-2-1100, Coastal Engineering Manual (2014), can also be used.

$$U_c = U_{10} = U_z \left(\frac{10}{z} \right)^{1/7} \quad (C-1)$$

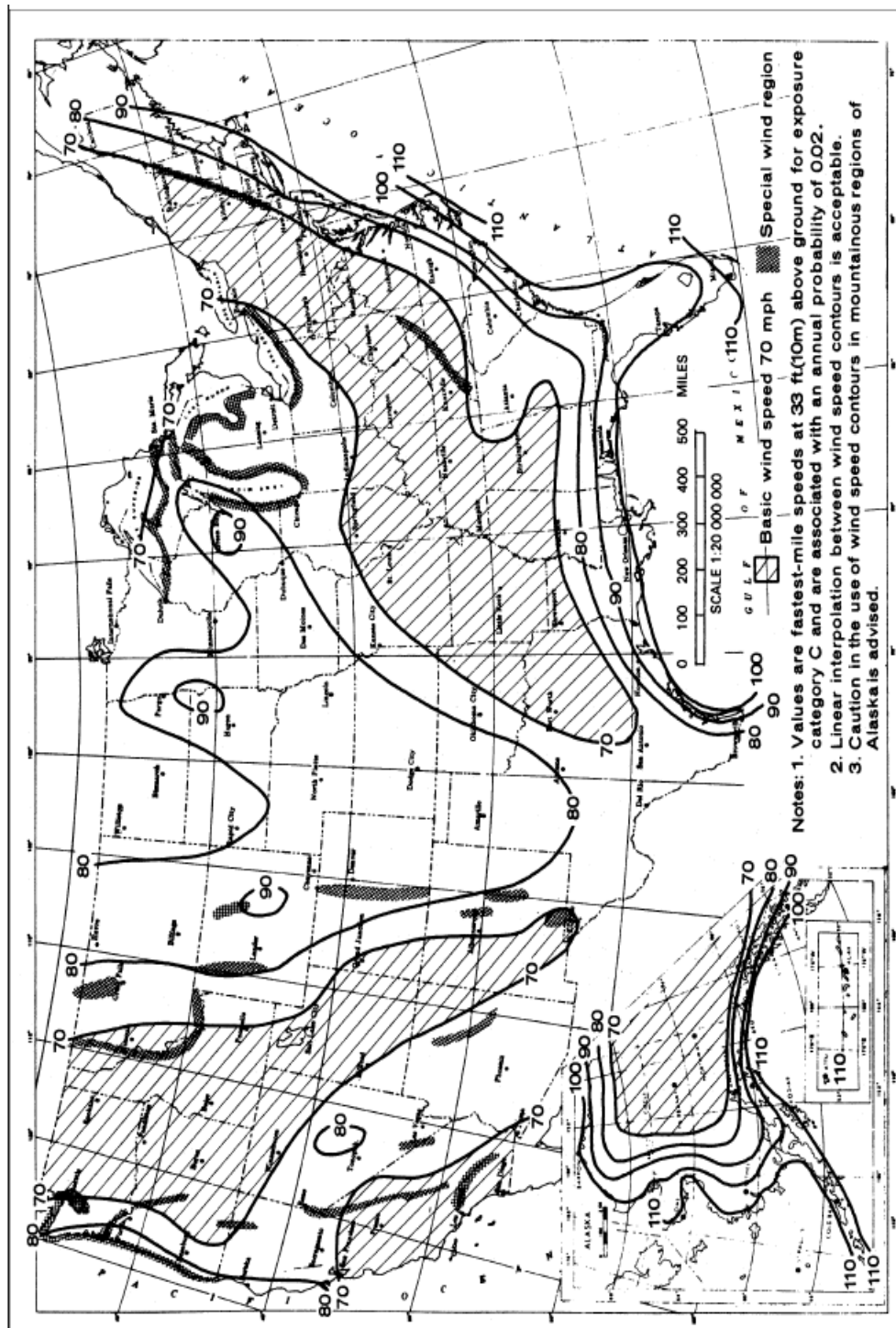


Figure C-2. General Extreme Fastest-Mile Wind Speeds with 50-yr Return Period across the United States at 10-meter Elevation.

e. Overwater wind speed adjustments. Most collected wind speeds will be from gauges over land. Gauges on the banks of reservoirs or very close to a shoreline can be considered overwater gauges. Under comparable meteorologic conditions, wind velocities over water are higher than over land surfaces due to smoother and more uniform surface conditions. Winds blowing from land tend to increase with passage over reservoir areas. The relationships are not constant but vary with topographic and vegetative cover of the land areas involved, reservoir configurations, and other conditions affecting air flow (e.g., wind speeds over a prairie grassland are likely to be very similar to that of overwater). Figure C-4 reproduces Figure II-2-7 in EM 1110-2-1100 that can be used to convert overland wind speeds (U_L) to overwater (U_w) speeds. Refer to EM 1110-2-1100 III-4-2-2 “Measurement of Wind Speed and Direction” for more information.

f. Atmospheric stability adjustments. This adjustment is only applicable to reservoirs with an effective fetch over 10 miles. These adjustments account for differences in air-water temperatures, which affect the stability of the atmosphere and the velocity profile of the wind near the water surface. In the case of inland reservoirs, the difference in air and water temperatures and the condition of the atmospheric boundary layer are typically not known.

(1) According to EM 1110-2-1100 (2014), the air-water temperature difference can significantly affect light and moderate winds, but it has minimal impact (5% or less) on high wind speeds typical of wind-wave design.

(2) Adjustments to wind speeds for atmospheric stability are recommended as flow characteristics within the atmospheric boundary layer are influenced by thermal stratification and horizontal density gradients. Use Figure II-2-6 in EM 1110-2-1100 to estimate wind speed adjustments due to temperature differences between the water and air. The figure from EM 1110-2-1100 is reproduced in Figure C-3 of this document.

g. Site wind speed frequencies and durations. The collected and corrected (U_c) site wind speed data will have their own frequencies and durations (e.g., 1% 1-minute wind speed). Wind speeds of other frequencies and durations will often need to be estimated from these data. The wind speed duration that produces the largest wave height is dependent on both the effective fetch length available for the wave’s development and the duration the wind blows. This is discussed further in Section C-4 part c. The estimation of wind speeds for several durations is important because it is not obvious what wind speed will produce the highest wave height for a site. Several frequencies of wind speeds should also be considered to help determine the most appropriate design.

(1) Wind speeds for the 50-year frequency determined from Figure C-2 can be adjusted to other return periods using Table II-8-6 in EM 1110-2-1100 (2014). This table is reproduced in Table C-1. If another frequency not provided in the table is needed, refer to ASCE 7-02 Table C6-3 for more conversion factors.

(2) EM 1110-2-1100 Figure II-2-1 provides the ratio of wind speed of any duration to the 1-hour wind speed. This figure is reproduced in Figure C-5. The wind speed duration, t , in the figure is in units of seconds.

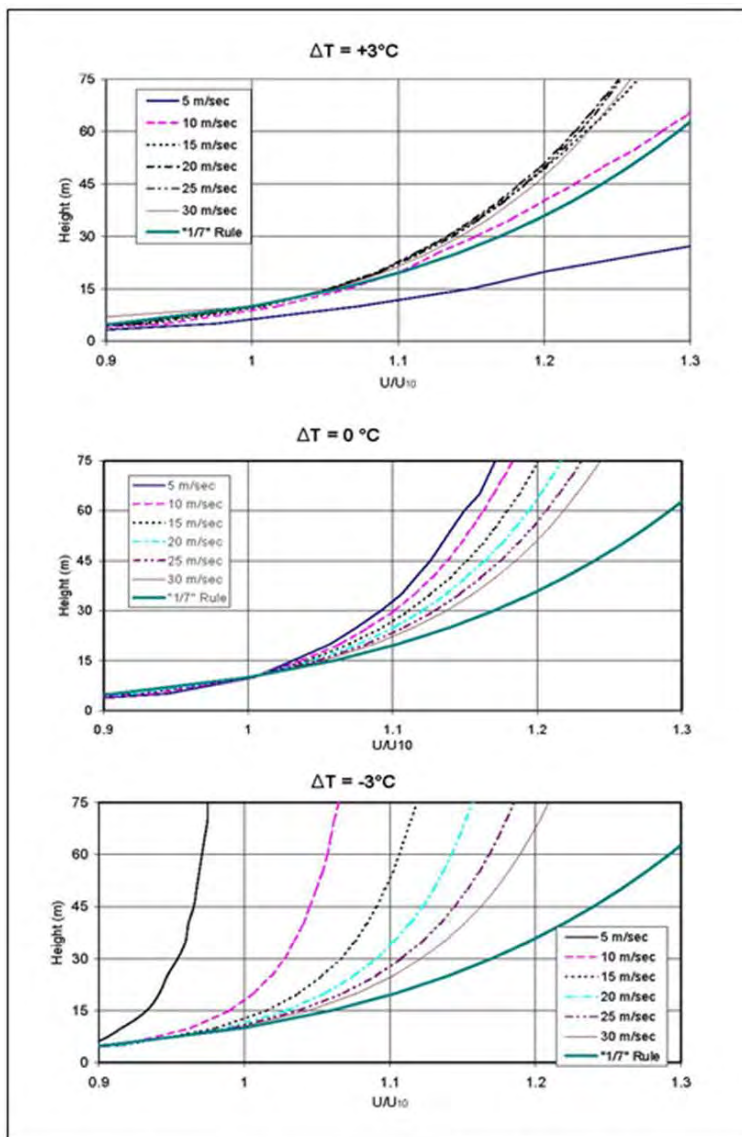


Figure II-2-6. Ratio of wind speed at any height to the wind speed at the 10-m height as a function of measurement height for selected values of air-sea temperature difference and wind speed: (a) $\Delta T = +3^\circ\text{C}$; (b) $\Delta T = 0^\circ\text{C}$; (c) $\Delta T = -3^\circ\text{C}$. Plots generated with following conditions: duration of observed and final wind = 3 hrs; latitude = 30°N ; fetch = 42 km; wind observation type: over water; fetch conditions: deep open water.

Figure C-3. Atmospheric Stability Adjustments.

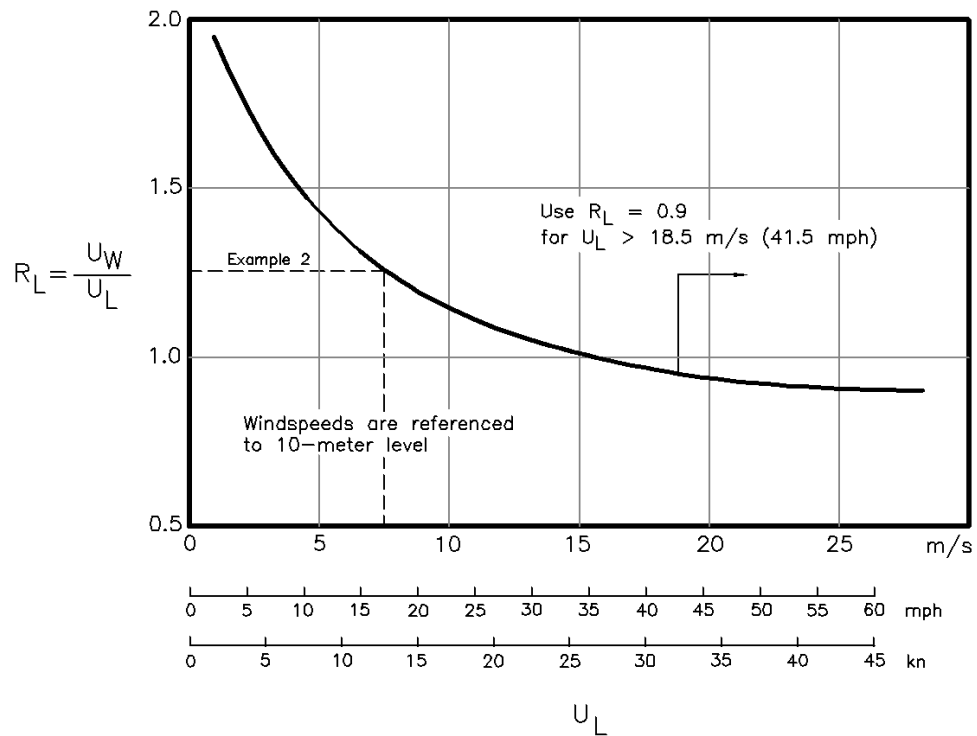


Figure C-4. Conversion of Overland Wind Speeds to Overwater Wind Speeds.

Table C-1. Return Period Adjustment Factor (Table II-8-6 in EM 1110-2-1100).

Return Period, % Exceedance	Adjustment Factor	
	Other Regions	Hurricane Region (Gulf and Atlantic)
4	0.95	1.00
2	1.00	1.05
1	1.07	1.11

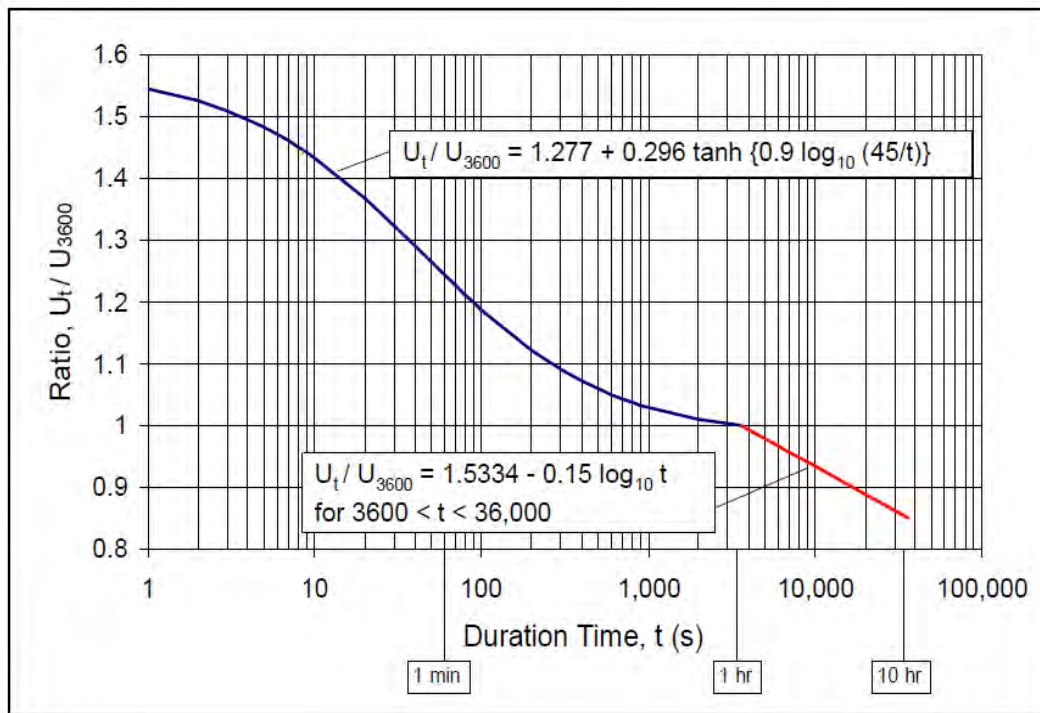


Figure C-5. Ratio of Wind Speed to Any Duration U_t to the 1-hour Wind Speed.

C-4. Wind Setup.

a. Wind setup in inland waters. When wind blows over a water surface, it exerts a horizontal stress on the water, driving it in the direction of the wind. In an enclosed body of water, this wind effect results in a piling up of water at the leeward end, and lowering of water level at the windward end. This effect is called “wind tide” or “wind setup” and is often represented as a wedge of water as shown in Figure C-6.

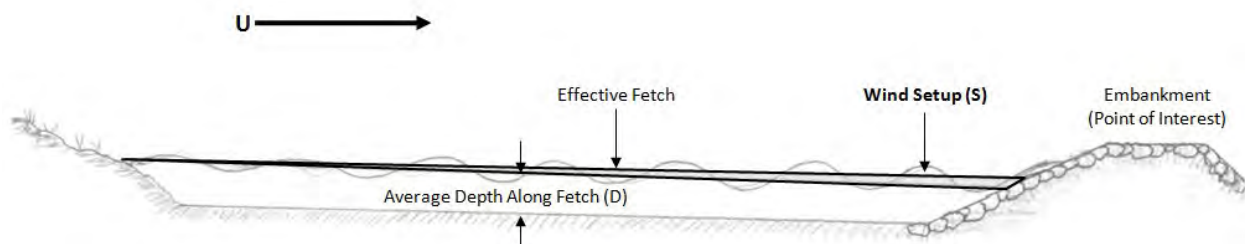


Figure C-6. Wind Setup.

b. Wind setup is a required component in the determination of freeboard. For shallow reservoirs (less than 16 feet in depth for the purpose of determining setup), wind setup is a particularly important component of freeboard and may even exceed the wave run-up component in magnitude. Two methods are available for determining wind setup: the Zeider Zee equation, which is applicable to reservoirs with average depths greater than 16 feet, and the Bretschneider Method, which is applicable to reservoirs with average depths less than 16 feet.

(1) The most widely used wind setup equation as applied to USACE reservoirs is the Zeider Zee equation. The Zeider Zee equation is described in both EM 1110-2-1414 and EM 1110-2-1420. It is presented here as Equation C-2. Zeider Zee normally results in the highest wind setup determination of the three methods.

$$S = \frac{U^2 F}{CD} \quad (C-2)$$

where:

S = wind setup above the stillwater elevation that would prevail with zero wind action, feet or meters

U = average wind speed over the fetch distance F, miles per hour or kilometers per hour

F = fetch distance, miles or kilometers

C = 1,400 for English units and 62,000 for Metric units

D = average depths of water generally along the fetch line

The fetch distance (F) used in the above formula should be the longest fetch distance to the structure (usually somewhat longer than the effect fetch, F_e).

(2) Although the Zeider Zee equation is commonly suggested for determining wind setup in reservoirs, the Zeider Zee equation may yield results that excessively overestimate wind setup at shallower water depths. The recommended alternative to the Zeider Zee equation at lower impoundment depths (average stillwater depth less than 16 feet including surcharge depth), is the Bretschneider method (Bretschneider 1953). For an enclosed reservoir the effective stress parameter in the Bretschneider method is presented in an integrated form in Equation C-3.

$$\frac{\kappa U^2 F}{gd^2} = \sum_{i=1}^{i=M} \left(\frac{\kappa U^2 \Delta x}{g(d_i)^2} \right) \quad (C-3)$$

where:

κ = a constant equal to 3.3×10^{-6} (dimensionless)

- U = average wind velocity (feet per second) over the fetch
F = fetch length (feet)
G = the gravitational constant (32.17 feet / sec²)
d = average depth (feet) of water generally along the fetch line
i = section number
x = horizontal distance (feet)

(3) Based on the calculated value of this stress parameter, wind setup can be determined from the tabulated data presented in Ippen (1966). If conditions are such that the stress parameter values fall beyond those presented in the tabulated data, it is recommended that the computations be completed manually (spreadsheet or numerical code based on Equation C-3).

(4) Refer to EM 1110-2-1414, Section 3-2 for a discussion of prediction models. Wind setup will typically be very small in the case of inland reservoirs.

C-5. Significant Wave Height and Period.

a. General. Wave height produced by winds is affected by the average wind speed and its duration and the reservoir effective fetch (F_e) available for wave growth. Water depth is also an important parameter in cases where the reservoir is considered to be shallow (average total depth is equal to or less than half the deep water wave length, $d/L_o < 0.5$) or total depth in the vicinity of the location of interest limits wave growth or causes the wave to break and reform before interacting with the structure. In general breaking will occur when the wave height reaches 0.78 times the total water depth, $0.78(d + S)$. Not all sources agree that shallow water factors such as bottom friction and percolation should be considered in wave growth analyses. The universal assumption of deep water wave growth may be applied for a conservative design. Forecasting equations and charts relate fetch, corrected wind speed, wind speed duration, and sometimes depth to estimate wave height and period. These charts are used to determine fetch-limited wind speeds that will be used to estimate the winds and durations that produce the significant wave heights (H_s). The previously determined effective fetch is used in the equations and charts along with a selection of wind speeds to produce the fetch-limited wind-speed duration curve of the reservoir.

b. Deep water wave forecasting. The EM 1110-2-1100 Equations II-2-35 and II-2-36 (2014) apply to deep water conditions. These formulas, and supporting equations for critical parameters, are reproduced in Equations C-4 through 8. Equation C-8 is the fetch-limited wind duration (time required for the wave to become fetch-limited). Equations C-9 and C-10 are the combined versions of the preceding equations in terms of the corrected wind speed. Use Equation C-9 to determine the specific wave height (H_s) and Equation C-10 to determine its period. It is recommended that analytical results from these equations be checked against those of the nomograms shown in Figures C-7 (English units) and C-8 (Metric units).

$$C_D = 0.001(1.1 + 0.035U_c) \quad (C-4)$$

$$u_* = (C_D U_c^2)^{1/2} \quad (C-5)$$

$$H_{mo} = \frac{4.13 \times 10^{-2} u_*^2 \left(\frac{gF_e}{u_*^2} \right)^{1/2}}{g} \quad (C-6)$$

$$T_p = \frac{0.651 u_*}{g} \left(\frac{gF_e}{u_*^2} \right)^{1/3} \quad (C-7)$$

$$t_{x,u} = 77.23 \frac{F_e^{0.67}}{U_c^{0.34} g^{0.33}} \quad (C-8)$$

$$H_{mo} = \frac{4.13 \times 10^{-2}}{g} \left[0.001 U_c^2 (1.1 + 0.035 U_c) \right] \cdot \left[\frac{gF_e}{(0.001 U_c^2 (1.1 + 0.035 U_c))} \right]^{1/2} \quad (C-9)$$

$$T_p = \frac{0.651}{g} \left[0.001 (1.1 + 0.035 U_c) U_c^2 \right]^{1/2} \cdot \left[\frac{gF_e}{(0.001 (1.1 + 0.035 U_c) U_c^2)} \right]^{1/3} \quad (C-10)$$

where:

F_e = effective fetch in meters.

H_{mo} = energy-based wave height in meters.

C_D = drag coefficient.

U_c = elevation-corrected wind speed in meters per second.

u_* = friction velocity in meters per second.

g = acceleration of gravity, 9.81 meters per second.

$t_{x,u}$ = fetch-limited wind duration at which waves become fetch-limited in growth in seconds.

T_p = wave period in seconds.

c. Shallow water wave forecasting.

(1) EM 1110-2-1100 recommends that deep water wave growth formulae be used for all depths. However, application of deep water equations to shallow-wave waves have the constraint that no wave period can grow past a limited value approximated by Vincent (1985) and shown in Equation II-2-39 from EM 1110-2-1100. The limiting equation is reproduced here as Equation C-11.

(2) Additionally, wave growth is assumed to continue only up to a point where an asymptotic depth-dependent wave height is attained. The depth limited wave height is traditionally defined as 0.78 times the total depth. If the reservoir of interest is defined as shallow and deep water wave formulae are applied, maximum wave height and wave period should be limited as discussed here. Note that wind setup should be included in the total water depth.

$$T_p \approx 9.78 \left(\frac{d_{Total}}{g} \right)^{1/2} \quad (C-11)$$

where:

- T_p = wave period, in seconds
- d_{Total} = total water depth, in meters
- g = acceleration due to gravity, 9.81 meters/second

(3) If the T_p calculated by the forecasting equations (C-2 through C-8) is larger than the period calculated in Equation C-11, the T_p should be decreased to the wave period limit calculated in Equation C-11.

(4) If a less conservative approach is desired, shallow water wave forecasting equations can be found in several references:

- U.S. Army Corps of Engineers (USACE). 1984. "Shore Protection Manual". Coastal Engineering Research Center, Waterways Experience Station, Second Printing.
- U.S. Army Corps of Engineers (USACE). 1989. EM 1110-2-1414, "Water Levels and Wind Heights for Coastal Engineering Design".
- Khandekar, M.L. 1989. "Operational Analysis and Prediction of Ocean Wind Waves". Springer, New York.
- Sorensen, R.M. 1993. "Basic Wave Mechanics for Coastal and Ocean Engineers". John Wiley & Sons, Inc.

d. Fetch-limited or duration-limited conditions. Wave growth can be either fetch-limited or duration-limited. It is important to determine if the wave growth at a site is limited by the length of open water to act upon or the duration of the wind speed acting on the water.

(1) Under fetch-limited conditions, wave heights are limited by the length of the fetch available for their development. Under duration-limited conditions, the wave heights are limited by the length of time the wind has blown at a specific wind speed.

(2) To illustrate fetch- and duration-limited conditions, refer to Figure C-7. For a wind speed of 20 miles per hour and wind duration of 8 hours, a wave height of 3.8 feet is possible at the point of interest (i.e., embankment). However, if the effective fetch length of the reservoir is only 10 miles, then the wave height is fetch-limited and will only be about 1.9 feet.

(3) The site wind speeds and durations that produce the largest significant wave heights can be determined by plotting the fetch-limited wind-speed duration against the wind-speed frequency-durations of the site. The intersections of the curves result in the wind speeds and durations of the significant wave height. These wind speeds and durations are called the design wind speeds and durations. This analysis is shown in Figure C-9 for the Fort Peck embankment. In this specific case, the design wind speed for the 1% frequency is 44 miles per hour and the design duration is 2.7 hours (Figure C-8).

(4) Equation C-6 provides the duration required for the waves to become fetch-limited. If the design wind condition has a determined duration of less than this value, wave growth is considered to be duration limited.

(5) Duration limited wave growth equations can be obtained by using duration to determine an equivalent fetch. EM 1110-2-1100 Equation II-2-38 (reproduced as Equation C-12) provides a method for converting duration to equivalent fetch (X). This estimated equivalent fetch can then be substituted into the fetch-limited wave growth formulae to obtain duration limited estimates of wave height and period.

$$\frac{gX}{u_*^2} = 5.23 \times 10^{-3} \left(\frac{gt}{u_*} \right)^{3/2} \quad (C-12)$$

e. Design wind speeds and durations. Design wind speeds and durations are determined by plotting a fetch-limited duration curve on the same graphs as the wind speed frequency-duration curves for the site. The design wind speeds and durations are those that produce the largest significant wave heights at the site. Figure C-8 shows an example of these curves for the Fort Peck embankment.

(1) The wind speed frequency curves for the site were determined earlier either from site-specific data or from Figure C-2. Site-specific data is highly recommended. All the wind-frequencies to be analyzed should be graphed.

(2) Fetch-limited wind speed duration curves are determined using Equation C-6. Wind speeds and durations should be chosen that envelop the wind speed frequency curves of the site.

(3) The intersections fetch-limited wind speed duration curves and the wind speed frequency-duration curves of the site results in the design wind speeds and durations used to calculate the significant wave height (H_s).

f. Significant wave height and its period. Significant wave heights (H_s) and their corresponding periods (T_s) are determined by using the design wind speeds and durations determined in (d) and the forecasting equations and/or charts. In the case of deep water conditions, H_{mo} and H_s are nearly identical, H_{mo} is H_s and T_p is T_s . In the case of shallow-water conditions, H_{mo} and H_s can vary as much as 10-15%.

g. Deep water wavelength.

(1) The theoretical estimate of deep water wavelength (L_o) is calculated using Equation C-13. While depth is considered negligible in the EM 1110-2-1100 forecasting equations, L_o should be calculated to check that a reduction in T_s is not required.

(2) The reservoir is considered deep if the average depth of the reservoir along the effective fetch length is larger than one-half of the theoretical deep water wavelength ($D > 1/2L_o$). If the reservoir average depth along the effective fetch is not larger than half the theoretical deep water wavelength, than shallow conditions prevail and the wave period should be checked as described in the following section.

$$L_o = \frac{gT_s^2}{2\pi} \quad (C-13)$$

$$L_o = 1.56T^2 \text{ for Metric units, } L_o = 5.12T^2 \text{ for English units.}$$

where:

L_o = theoretical deep water wavelength in feet or meters.

T_s = period of the significant wave height in seconds.

g = acceleration of gravity, 32.2 feet/sec² for English units or 9.81 meters/sec² for Metric units.

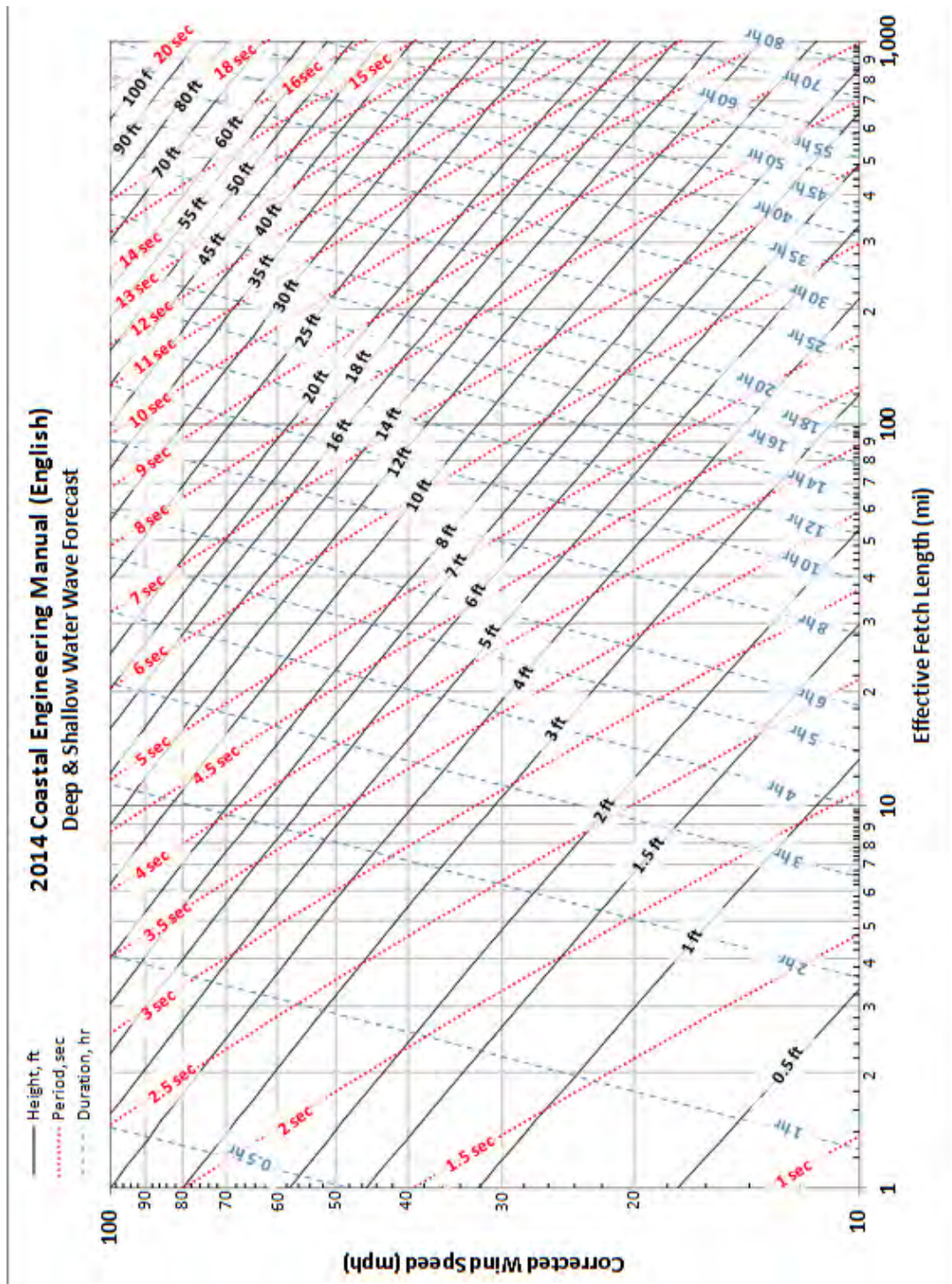


Figure C-7. Deep and Shallow Wave Forecast (English Units).

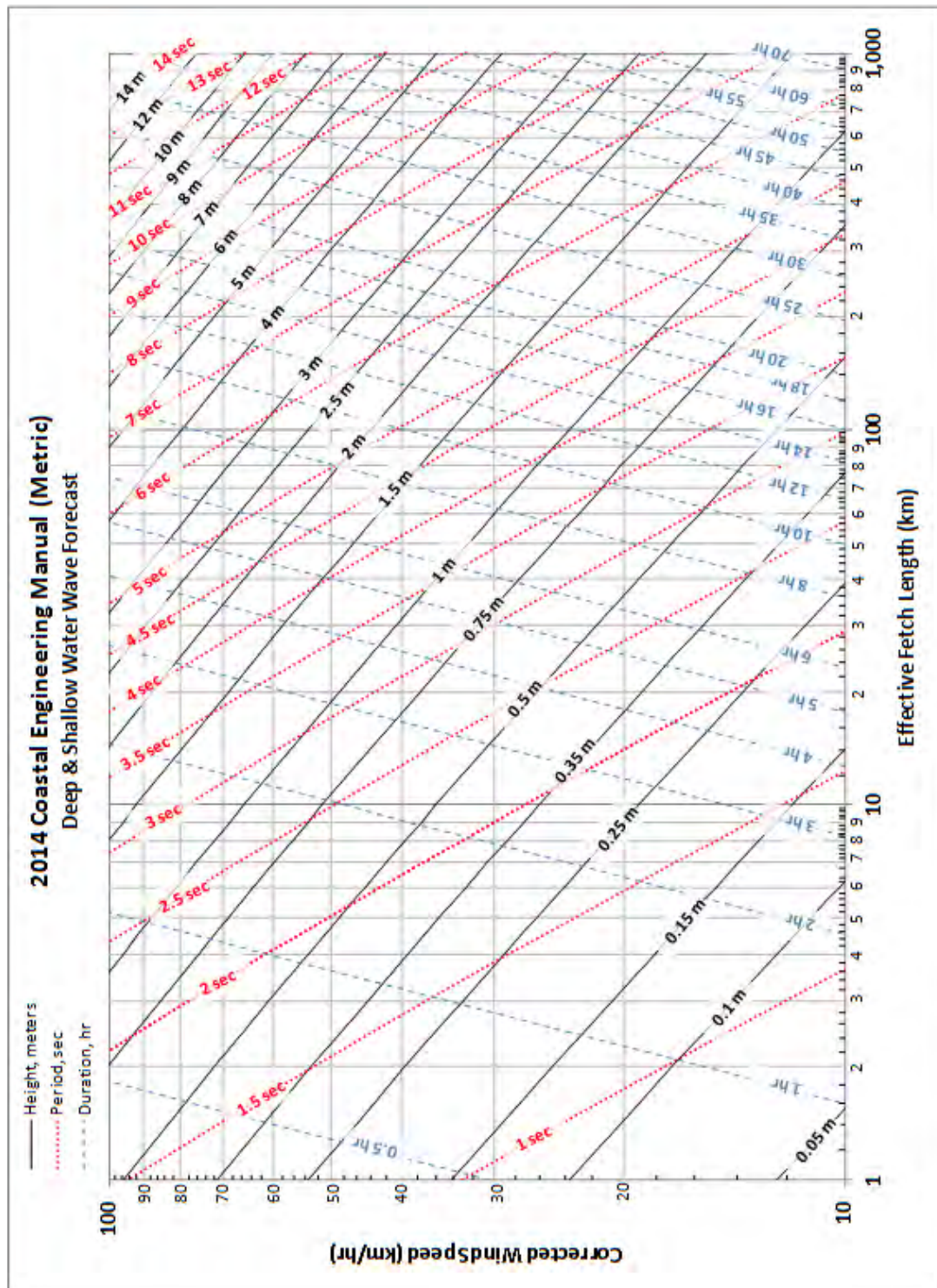


Figure C-8. Deep and Shallow Wave Forecast (Metric Units).

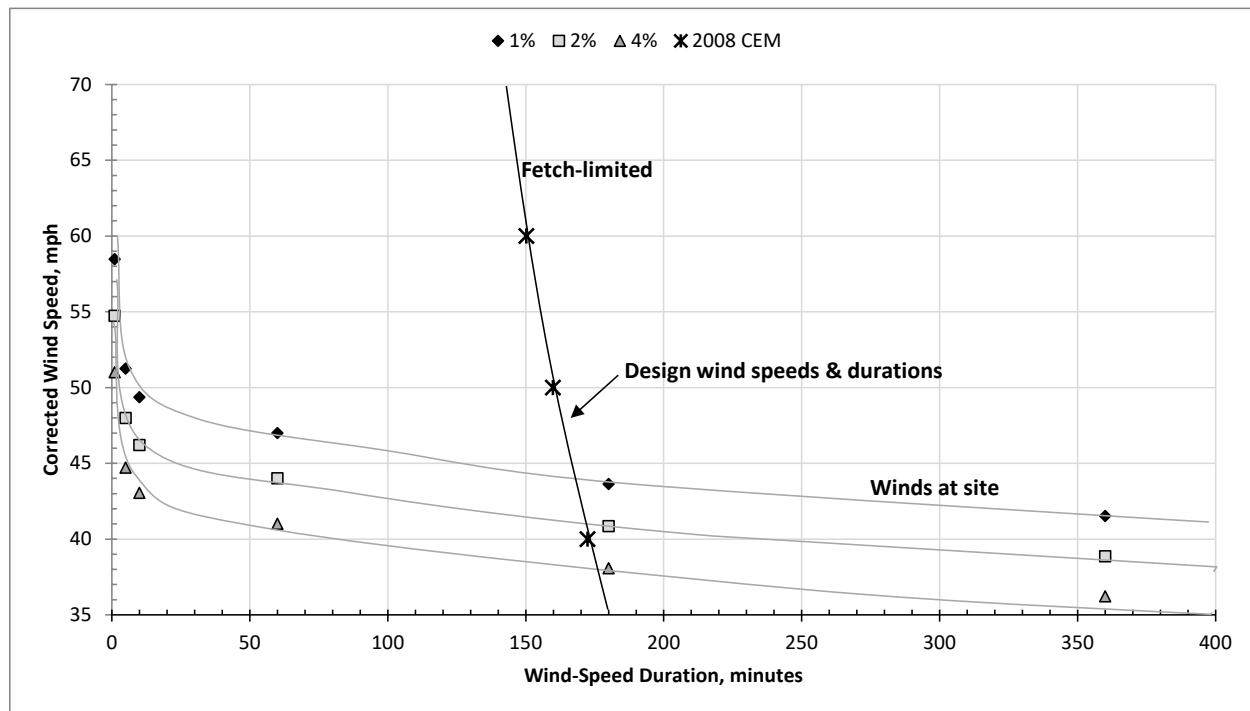


Figure C-9. Fort Peck Dam Embankment Wind Speed Frequency Duration.

h. Extreme wave heights. Wind-generated waves in a large body of water are not uniform but consist of waves with a range of heights and lengths. Refer to Section 15-2 in this manual for a definition of extreme wave height. The extreme wave height is estimated as shown in Equation C-14.

$$H_{2\%} = 1.4H_s \quad (C-14)$$

A factor of 1.67 times the significant wave height should be used to obtain the wave height exceeding 0.4% of the waves in the wave train.

C-6. Wave Run-Up on Sloping Embankments.

a. Introduction. Most dam embankments are fronted by deep water, have slopes between 1 on 2 and 1 on 4, and are armored with riprap. However, some shallow water reservoirs may be protected by grass covered earthen embankments with slopes as shallow as 1 on 6. Rockfill dams are considered permeable rubble slopes and earthfill dams with riprap armor are considered impermeable. Laboratory tests of many slopes, wave conditions, and embankment porosity provide sufficient data to make estimates of wave run-up on a prototype embankment.

b. Wave run-up on impermeable slopes. Impermeable slopes belong to embankments covered with impermeable surfaces like asphalt or concrete, rough surfaces like rubble stones or concrete ribs

on fine core materials and riprap on earth embankments. Chapter 5 of EM 1110-2-1100, Coastal Engineering Manual, has several equations used to calculate run-up on impermeable slopes. Only a few equations are presented here.

(1) Equation VI-5-3 of EM 1110-2-1100 can be used to estimate wave run-up using the significant wave height ($H_{33\%}$, H_s) and period (T_s). The same equation can be applied to determine run-up levels exceeded by other percentages of incident waves as well, such as $H_{2\%}$. This equation is shown in Equation C-15.

(2) Equation VI-5-12 of EM 1110-2-1100 can be used to estimate wave run-up for impermeable rock slopes fronted by deep water. Examples of these slopes are shown in Figure VI-5-11 of that manual. This equation is shown in Equation C-16.

(3) Both Equations C-15 and C-16 are functions of the surf-similarity parameter. The surf-similarity parameter (ξ) characterizes the form (i.e., spilling, plunging, collapsing, surging) of the incoming waves and how they are likely to break. Refer to Part IV Chapter 5 of EM 1110-2-1100 for variable values and more information if needed. Equation C-17 shows how to calculate the surf-similarity parameter.

(4) Equation C-15 can be used to calculate the significant wave run-up (H_s , $H_{1/3}$ or $H_{33\%}$) and the runup exceeded by $i\%$ of the incident waves ($H_{i\%}$) for a variety of impermeable slopes. The reduction factors (γ) for surface roughness and other embankment features are found in Table VI-5-3 of the EM 1110-2-1100. This table is partially reproduced in

(5) Table C-2. The coefficients A and C for the equation are documented in Table VI-5-2 of EM 1110-2-1100 for impermeable slopes. Note that all units are Metric.

(6) Equation C-16 can be used to calculate the significant wave runup (H_s , $H_{1/3}$ or $H_{33\%}$) and the run-up exceeded by $i\%$ of the incident waves ($H_{i\%}$) for impermeable rock slopes. The coefficients A, B and C are provided in Table VI-5-5 of EM 1110-2-1100.

$$\frac{R_{i\%}}{H_s} = (A\xi + C)\gamma_r\gamma_b\gamma_h\gamma_\beta \quad (C-15)$$

$$\frac{R_{i\%}}{H_s} = \begin{cases} A\xi_{om} & \text{for } 1.0 < \xi_{om} \leq 1.5 \\ B(\xi_{om})^C & \text{for } \xi_{om} > 1.5 \end{cases} \quad (C-16)$$

$$\xi = \frac{\tan \theta}{\sqrt{\frac{2\pi H_s}{gT_s^2}}} \quad (C-17)$$

where:

$R_{i\%}$ = run-up level exceeded by $i\%$ of the incident waves in meters.

H_s = significant wave height in meters.

T_s = wave period of the significant wave height in seconds.

ξ or ξ_{om} = surf-similarity or break-water parameter.

A, B, C = coefficients dependent on the surf-similarity parameter and i . See Table IV-5-2 (EM 1110-2-1100) for values for equation C-15 and Table IV-5-5 for values for equation C-16.

γ_r = reduction factor for influence of surface roughness (1 for smooth slopes); see Table VI-5-3 (EM 1110-2-1100) for values.

γ_b = reduction factor for influence of a berm (1 for non-bermed profiles).

γ_h = reduction factor for influence of shallow-water conditions where the wave height distribution deviates from the Rayleigh distribution (1 for Rayleigh-distributed waves).

γ_β = factor for influence of angle of incidence β of the waves (1 for head-on long-crested waves, i.e., $\beta = 0^\circ$). See EM 1110-2-1100 (Part VI) Chapter 5 for more values.

$\tan \theta$ = rise/run at break water location (i.e., if 1v:3h then $\tan \theta = 1/3$).

g = acceleration of gravity, 9.81 meters/s².

Table C-2. Run-Up Reduction Factors.

Surface Roughness Factor for Equations C-15 (EM 1110-2-1100 Table VI-5-3)	
Type of Slope Surface	γ_r
Smooth, concrete, asphalt	1.0
Smooth block revetment	1.0
Grass (3 cm length)	0.90 -1.0

Surface Roughness Factor for Equations C-15 (EM 1110-2-1100 Table VI-5-3)	
1 layer of rock, diameter D, ($H_s/D = 1.5-3.0$)	0.55 - 0.60
2 or more layers of rock, ($H_s/D = 1.5-6.0$)	0.50 - 0.55

c. Wave run-up on permeable sloped embankments. Permeable slopes are those with substantial structure pores, typically rubble-mound structures with secondary armor layers, filter layers, and quarryrun cores. Rockfill dams are considered to be permeable rubble slopes. The storage capacity of the structure pores in a permeable embankment result in maximum run-ups that are smaller than those on impermeable structures of the same dimensions.

(1) Significant (R_s) and extreme ($R_{2\%}$) wave run-up can be estimated using the calculated surf-similarity parameter and Equations VI-5-13 from EM 1110-2-1100 Part VI Chapter 5. The surf-similarity parameter is calculated through Equation C-17 shown previously. Note that units are Metric. Refer to Table VI-5-5 in EM 1110-2-1100 for coefficient values A, B, C and D.

$$\begin{aligned}
 R_{ui} \% / H_s &= A \xi_{om} && \text{for } 1.0 < \xi_{om} \leq 1.5 \\
 &= B (\xi_{om})^C && \text{for } 1.5 < \xi_{om} \leq (D/B)^{1/C} \\
 &= D && \text{for } (D/B)^{1/C} \leq \xi_{om} < 7.5
 \end{aligned}
 \tag{C-18}$$

where:

ξ_{om} = surf-similarity or break-water parameter.

H_s = significant wave height in meters.

d. Adjustments in wave run-up estimates for variations in riprap.

(1) A rough riprap layer on an embankment tends to reduce the height of run-up after a wave breaks. If the riprap layer thickness is small in comparison with wave magnitudes and the underlying surface is relatively impermeable, so that the void spaces in the riprap remain mostly filled with water between successive waves during severe storm events, the height of run-up may closely approach heights attained on smooth embankments of comparable slope. However, if the riprap layer is sufficiently rough, thick, and free draining to quickly absorb the water that impinges on the embankment as each successive wave breaks, further wave run-up will be almost completely eliminated.

(2) The design of riprap to absorb most of the energy of breaking waves is practicable if waves involved will be relatively small or moderate, but costs and other practical considerations usually preclude such design where large waves are encountered. Accordingly, the design characteristics of riprap layers are usually somewhere between the two extremes described above.

C-7. Wave Run-Up on Vertical Embankments.

a. Introduction. Run-up on vertical embankments can be calculated using the Sainflou or Goda formulae described in Chapter 5 of EM 1110-2-1100 Part VI.

C-8. Allowable Wave Overtopping.

a. Wave overtopping occurs when wave run-up and wind setup levels combine to produce a water level greater than the crest elevation of the embankment. When waves over-wash a structure it occurs intermittently, not continuously, as individual high waves among a multitude of storm waves attack the face of the structure. A structure that is built to prevent all overtopping is often cost prohibitive. Instead of designing a structure with no overtopping, typically it is more practical to design the height of the structure to a given design condition (e.g., H_s , $H_{2\%}$ or total overtopped volume), understanding that a certain percentage of waves in the incident wave field will generate run-up levels that will overtop the structure.

b. Wave overtopping is an important design element both in terms of predicting backside flooding and safeguarding structural integrity of the embankment. Several methods exist for predicting the over-wash flow rate in a given situation. Over-wash is generally measured in one of two ways, as a volume over time per unit length of structure or as a mean rate of over-wash volume per unit length over the duration of the storm event.

c. Allowable wave overtopping is the amount or rate of over-wash that is permissible on a given embankment. This rate is a function of the construction of the embankment. An unprotected earthen embankment will not tolerate a high over-wash rate. The resulting intermittent flow on the backside will quickly lead to erosion and failure. For these situations, the embankment elevation and freeboard may be increased to minimize the wave discharge over the structure. Grass, turf reinforced mats (TRM) or artificial turf mats add an element of protection to earthen slopes, allowing for higher over-wash rates and lower freeboards. Riprap, concrete, asphalt and other hard armor types result in the highest allowable discharge rates and the lowest freeboard requirements.

d. The wave overtopping discharge varies considerably from wave to wave. The majority of the discharge during a storm is due to just a small fraction of the waves. These largest, less frequent waves, can produce discharge that is 100 times the average. For practicality, most information on wave overtopping is given as the time averaged over-wash (discharge) rate, expressed as a volume per second per unit length of structure.

e. Critical values of average wave overtopping discharges. EM 1110-2-1100 Part VI, Table VI 5-6 (recreated in Figure C-9) provides general information for identifying allowable discharge rates for different structure types. These values should be considered only as rough guidelines as the same discharge rates may occur from difference intensities of wave action that could contribute to an attack on the structure integrity. Allowable wave overtopping rates must be selected based on site specific conditions including storm wave climate, geotechnical makeup, construction materials, ability to route overtopping volumes, and proximity to other structures and/or population centers.

f. Average overtopping discharge formulae. Formulae for overtopping are empirical because they are fitted to hydraulic model test results for specific structure geometries. In general, the average overtopping discharge per unit length of structure, q , is a function of the following:

- (1) H_s – significant wave height
- (2) T_{op} – wave period associated with the spectral peak in deep water (alternatively, T_{om})
- (3) σ – spreading of short-crested waves
- (4) β – angle of incidence for the waves
- (5) R_c – freeboard
- (6) H_s – water depth in front of structure
- (7) G – gravitational acceleration

g. EM 1110-2-1100 Part VI, Tables VI-5-7 through VI-5-13 provide a number of overtopping models and formulae for impermeable and permeable structures of different configurations. Using the wave climate and wave run-up information determined from previous sections, variable freeboard heights (R_c) can be entered into the formulae presented in EM 1110-2-1100 to determine corresponding discharge rates. Freeboard values that result in wave overtopping discharge rates equal to or less than the allowable discharge rate for the structure are viable design freeboard values.

Critical Values of Average Overtopping Discharges

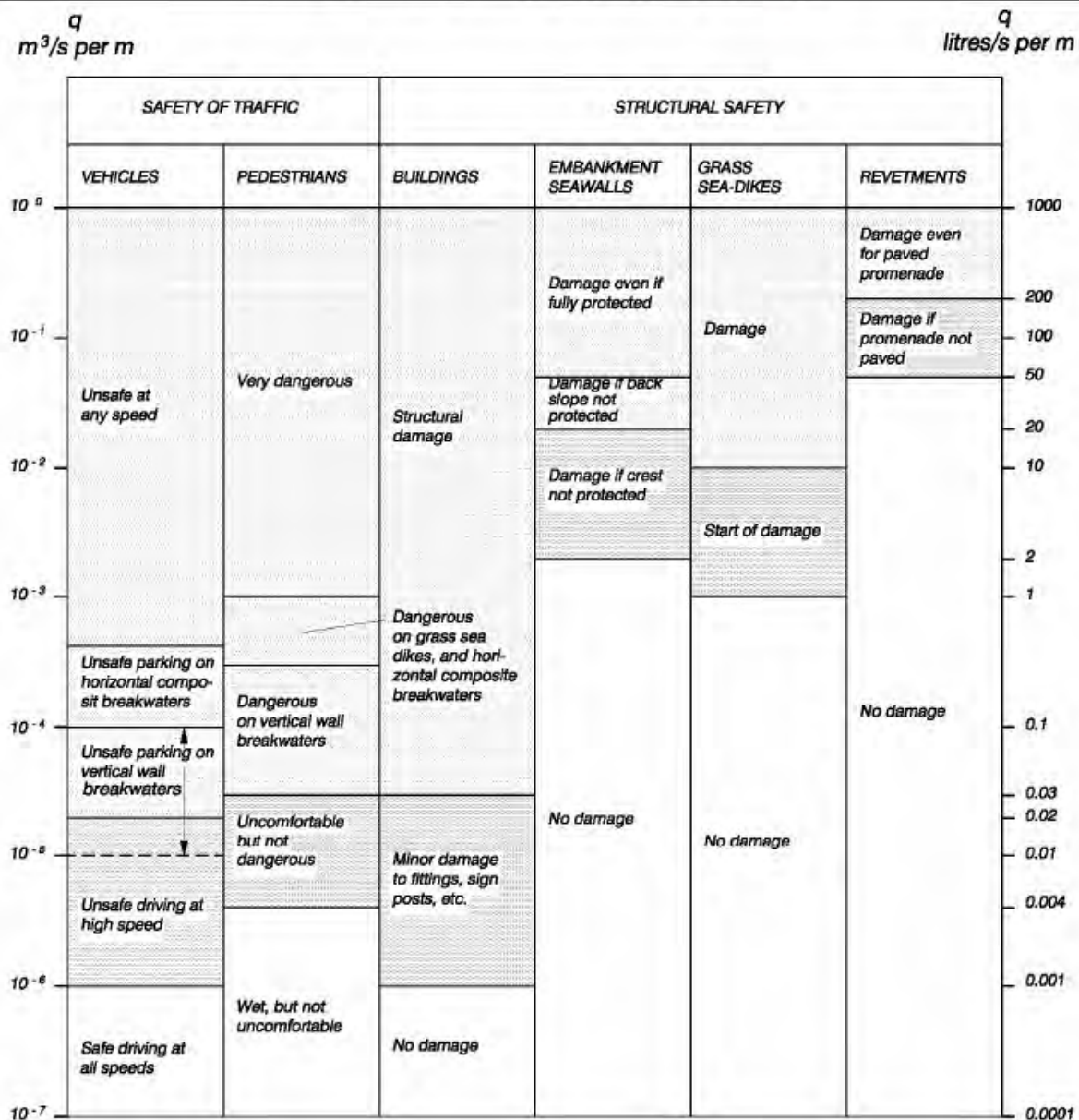


Figure C-10. Critical Values of Overtopping Discharges.

h. Final design elevation. The final elevation of an embankment affected by wave and wind action is determined by adding the top of the inflow design flood pool, wind setup, and wave run-up, while considering the resulting overtopping discharge rate, estimated project cost, and acceptable level of risk. See Figure C-11 for an additional example using equations from EM 1110-2-1100.

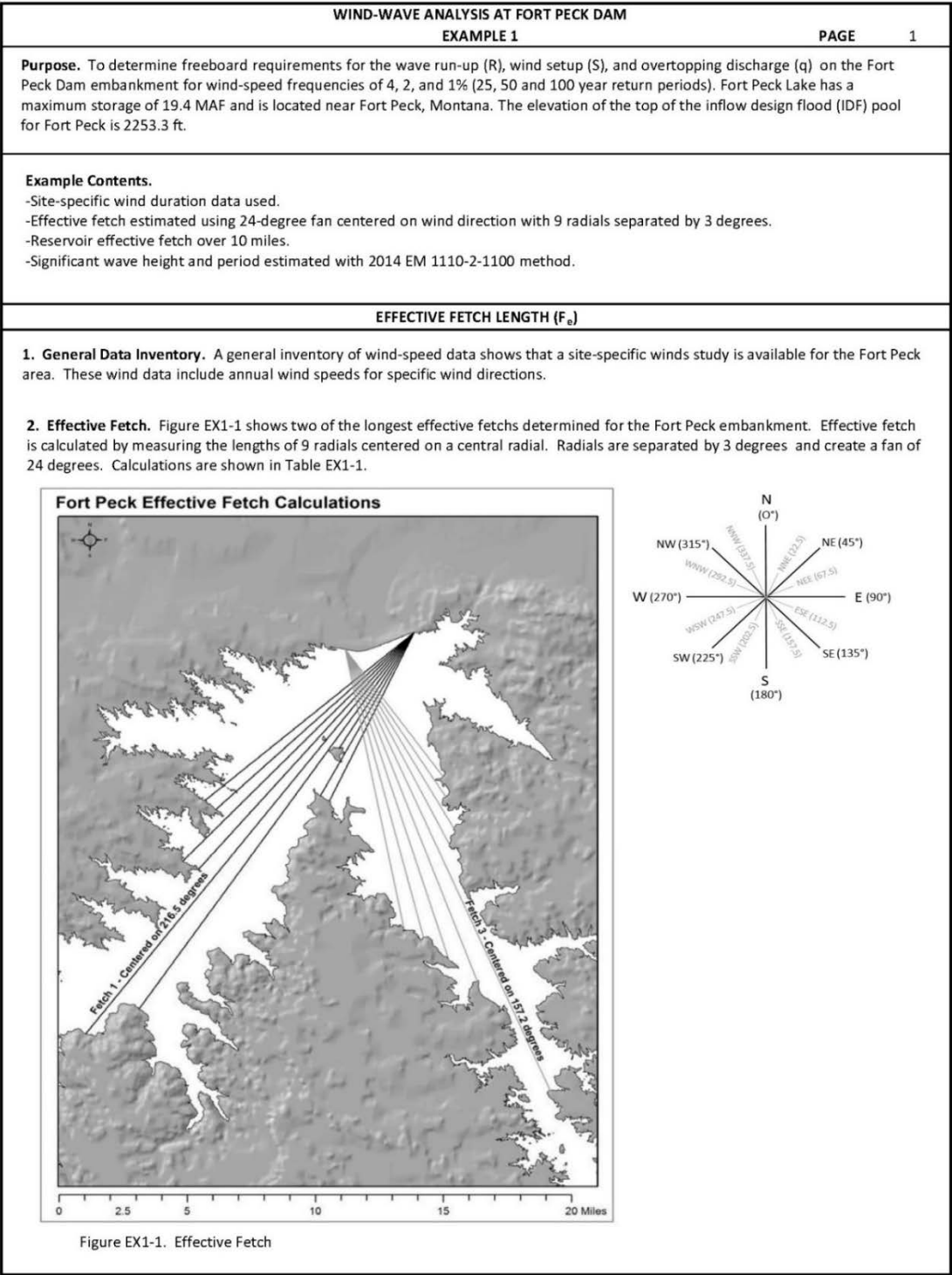


Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1.

WIND-WAVE ANALYSIS AT FORT PECK DAM			
EXAMPLE 1			
			PAGE 2
Table EX1-1. Effective fetch calculations			
Fetch 1 - 216.5 degrees			
Radial No.	Angle from Central Radial	Length of Radial, mi	
	Radial	Radial, mi	
1	12	21.6	
2	9	8.0	
3	6	7.9	
4	3	19.5	
5	0	13.2	
6	3	13.4	
7	6	10.9	
8	9	10.9	
9	12	5.8	
Average (mi) =		12.4	
Fetch 3 - 157.2 degrees			
Radial No.	Angle from Central Radial	Length of Radial, mi	
	Radial	Radial, mi	
1	12	20.7	
2	9	6.7	
3	6	7.5	
4	3	8.8	
5	0	9.4	
6	3	15.2	
7	6	14.0	
8	9	12.8	
9	12	12.3	
Average (mi) =		11.9	
The longest effective fetch calculated is 12.4 miles. Consider the directional wind speeds before selecting this as the effective fetch for calculations.			
WIND SPEEDS			
3. Wind speeds. Annual one-hour wind frequency data are available for the 4, 2 and 1% frequency wind events. These data were derived from several gages on land. Applicable wind speeds are shown in Table EX1-2.			
The longest effective fetch is centered on 216.5 degrees (Fetch 1), which falls between the SSW and SW directions. The largest wind speeds of these two directions were selected. Wind speeds around Fetch 3 were also checked but determined to be lower. Because the effective fetch and wind speeds for the Fetch 3 direction are lower than those for Fetch 1, only Fetch 1 is considered in this example.			
Table EX1-2. Referenced overland wind speeds			
Wind Direction (degrees)	1-Hr Overland Wind Speed (mph)		
	4%	2%	1%
ESE - 112.5	34	36	38
SE - 135	31	33	35
SE - 157.5	30	33	35
S - 180	32	35	38
SSW - 202.5	38	41	45
SW - 225	41	44	47
WSW - 247.5	45	48	51
Average	36	39	41
Max	45	48	51
Min	30	33	35
Selected	41	44	47
4. Elevation adjustments. It is stated in the site-specific wind speed study referenced that these wind speeds are already corrected to the standard 33 ft (10 meter) elevation.			
5. Overwater wind speed adjustments. The wind speed data were recorded at a gage on land. Speeds need to be adjusted to wind speeds over water. Use Figure II-2-7 in EM 1110-2-1100 for the conversion (Figure EX1-2). Table EX1-3 shows the converted wind speeds.			

Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1 (Continued).

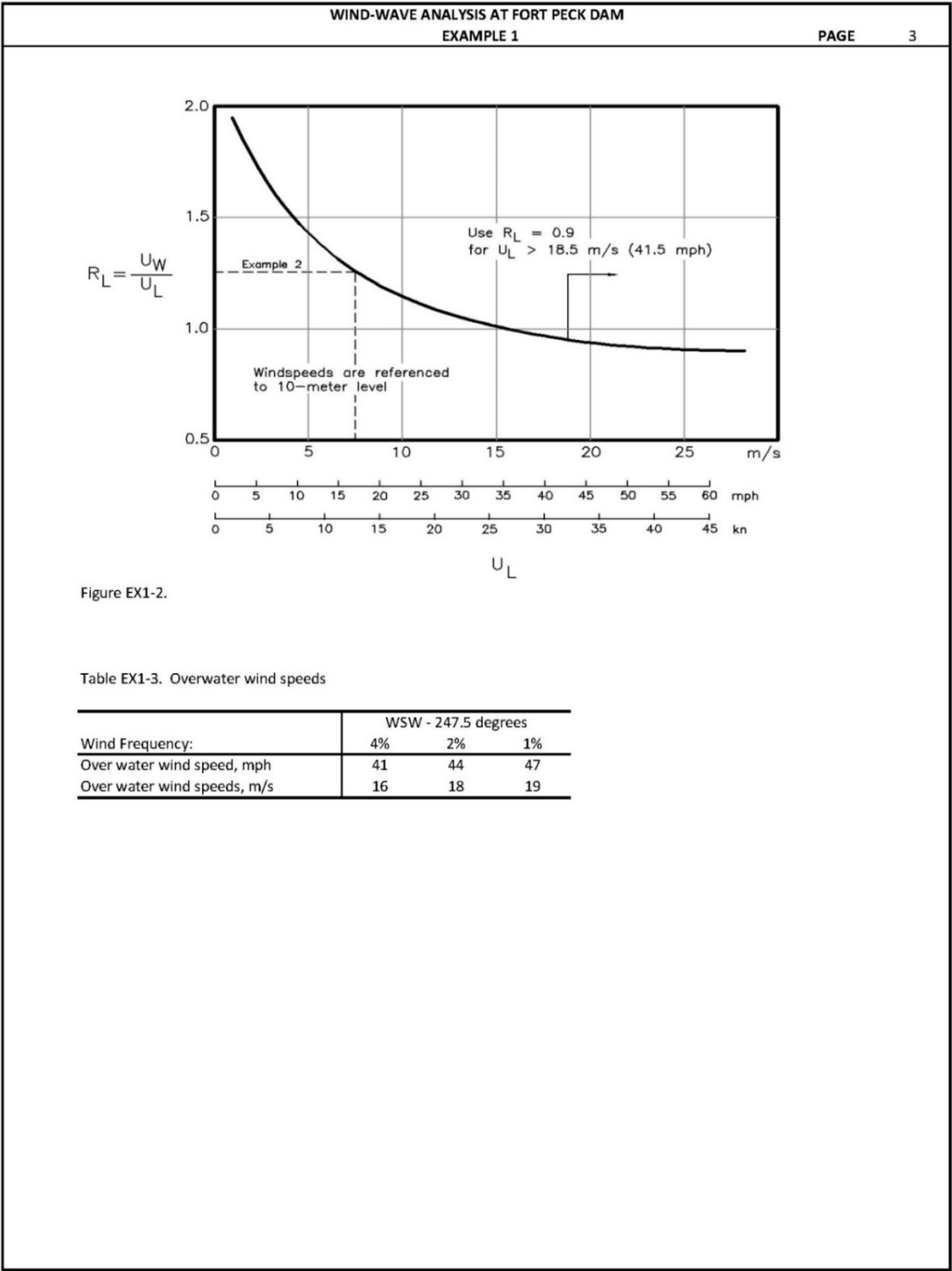


Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1 (Continued).

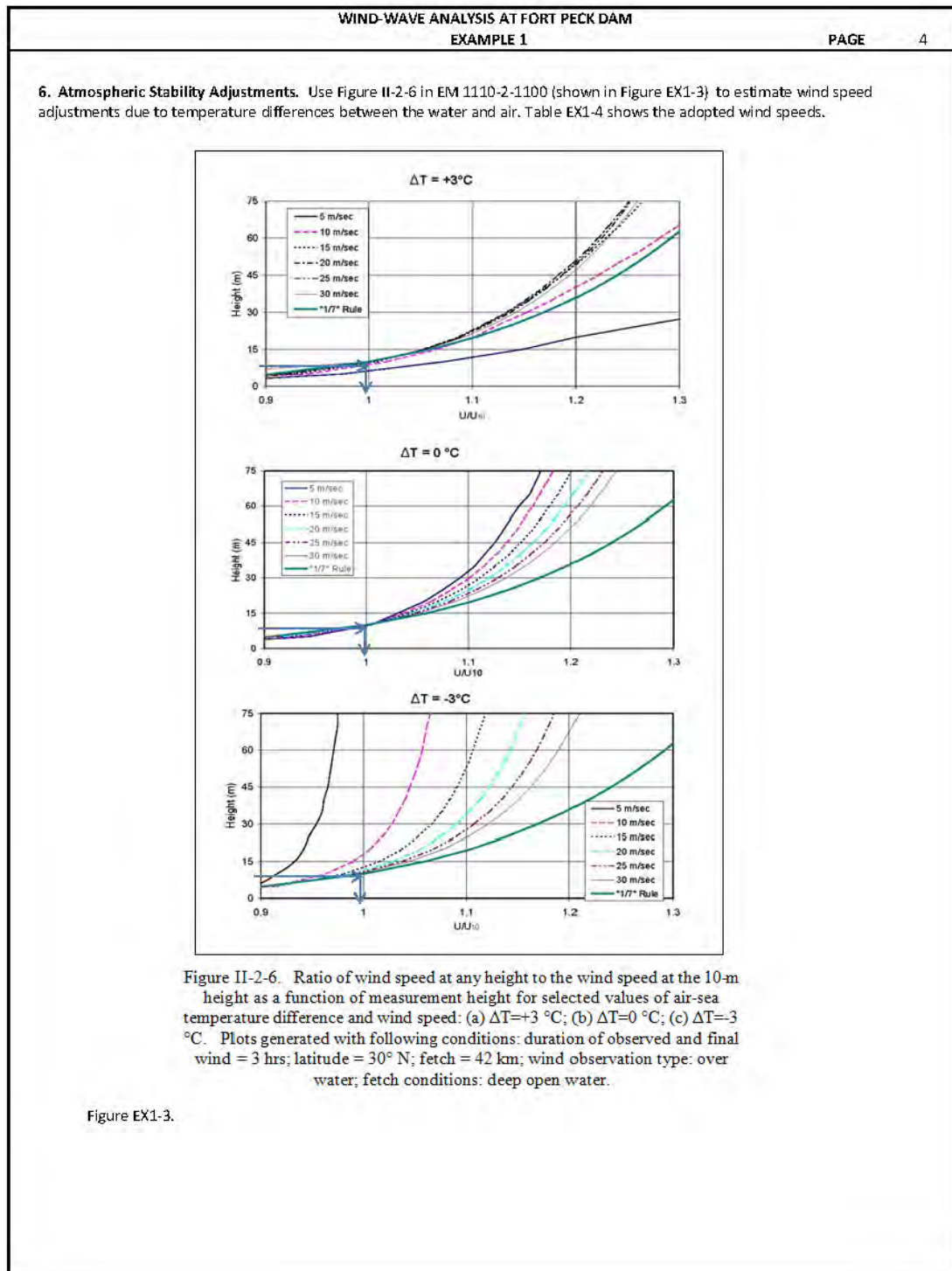


Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1 (Continued).

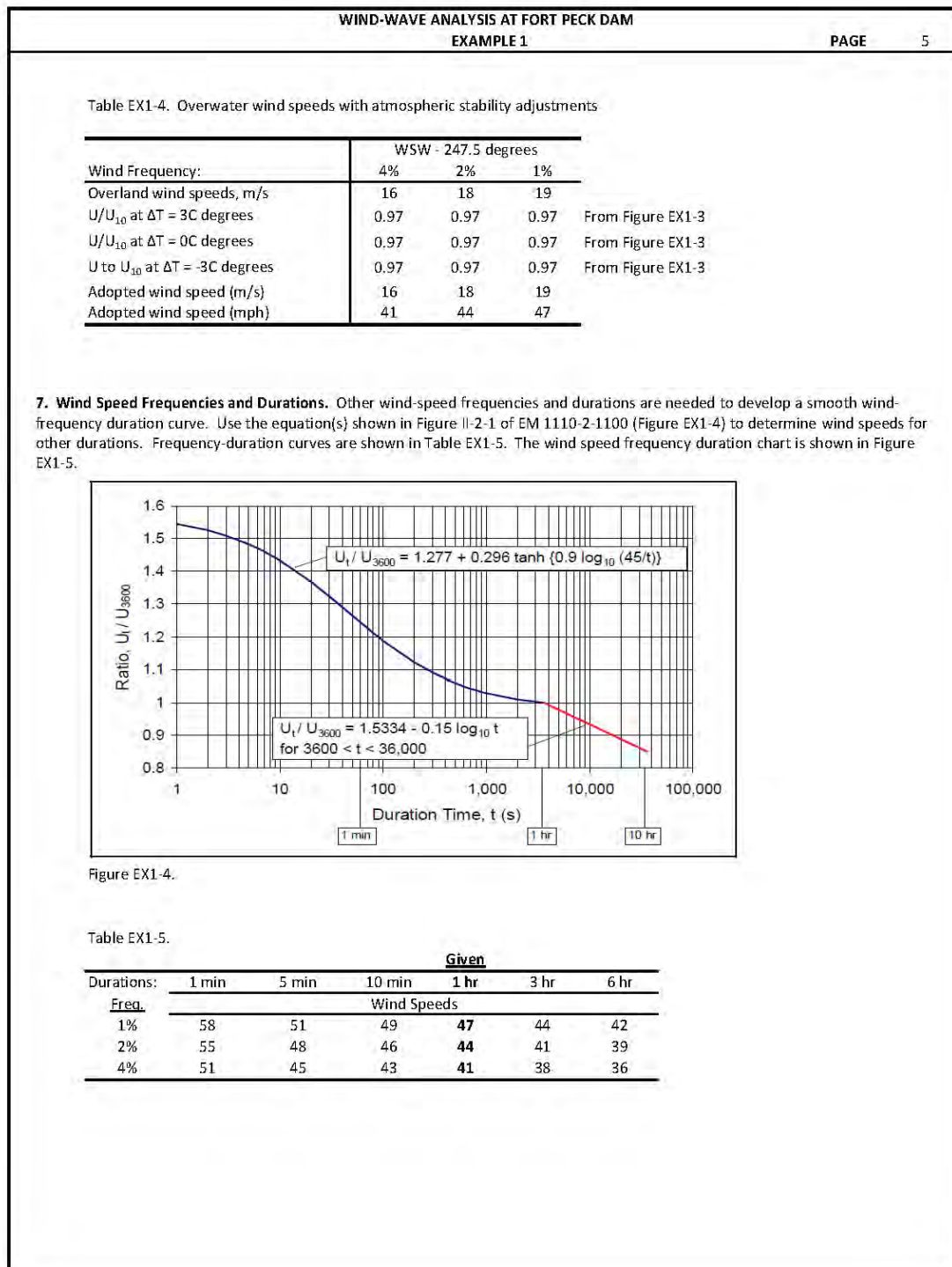


Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1 (Continued).

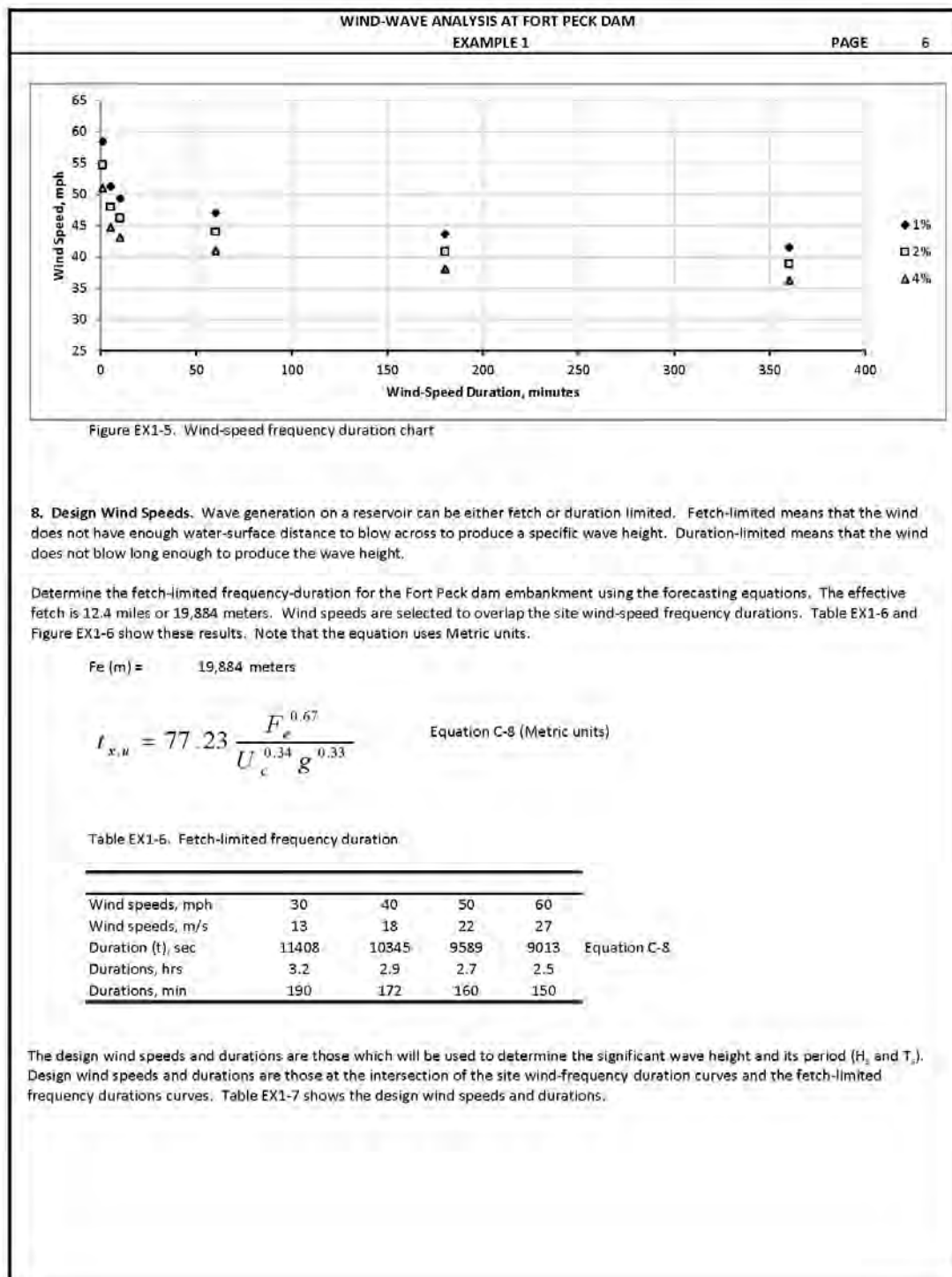


Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1 (Continued).

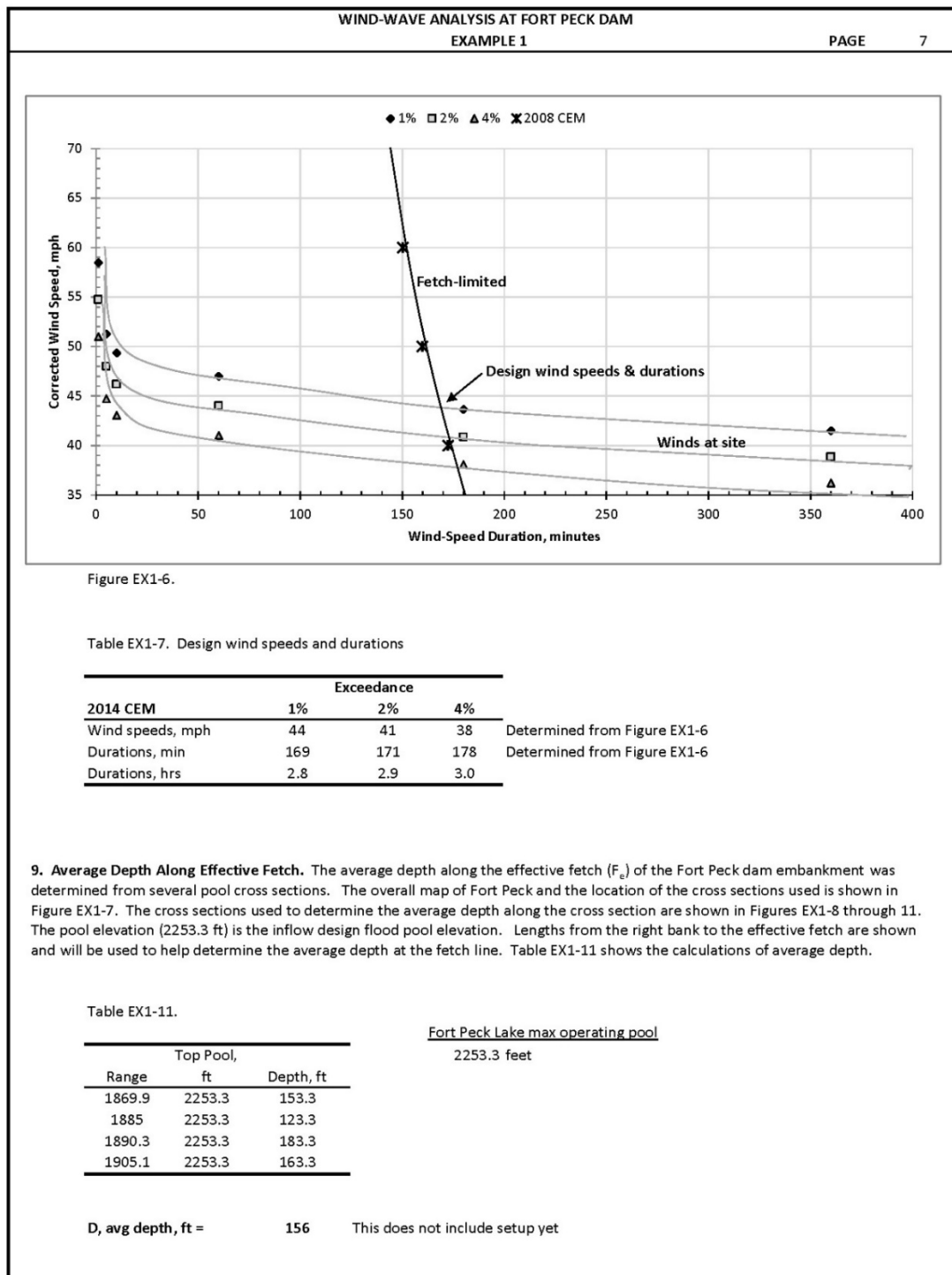


Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1 (Continued).

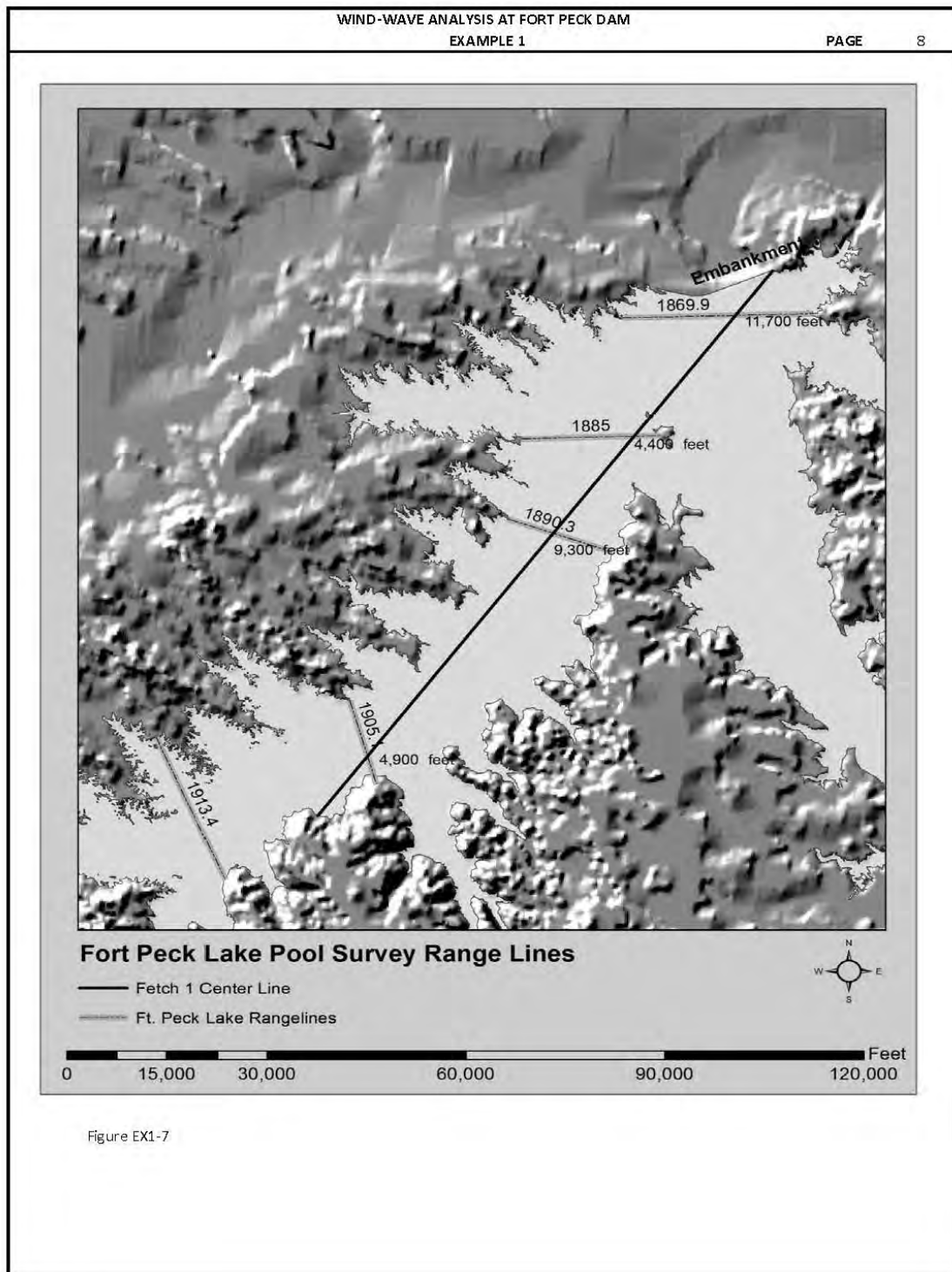


Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1 (Continued).

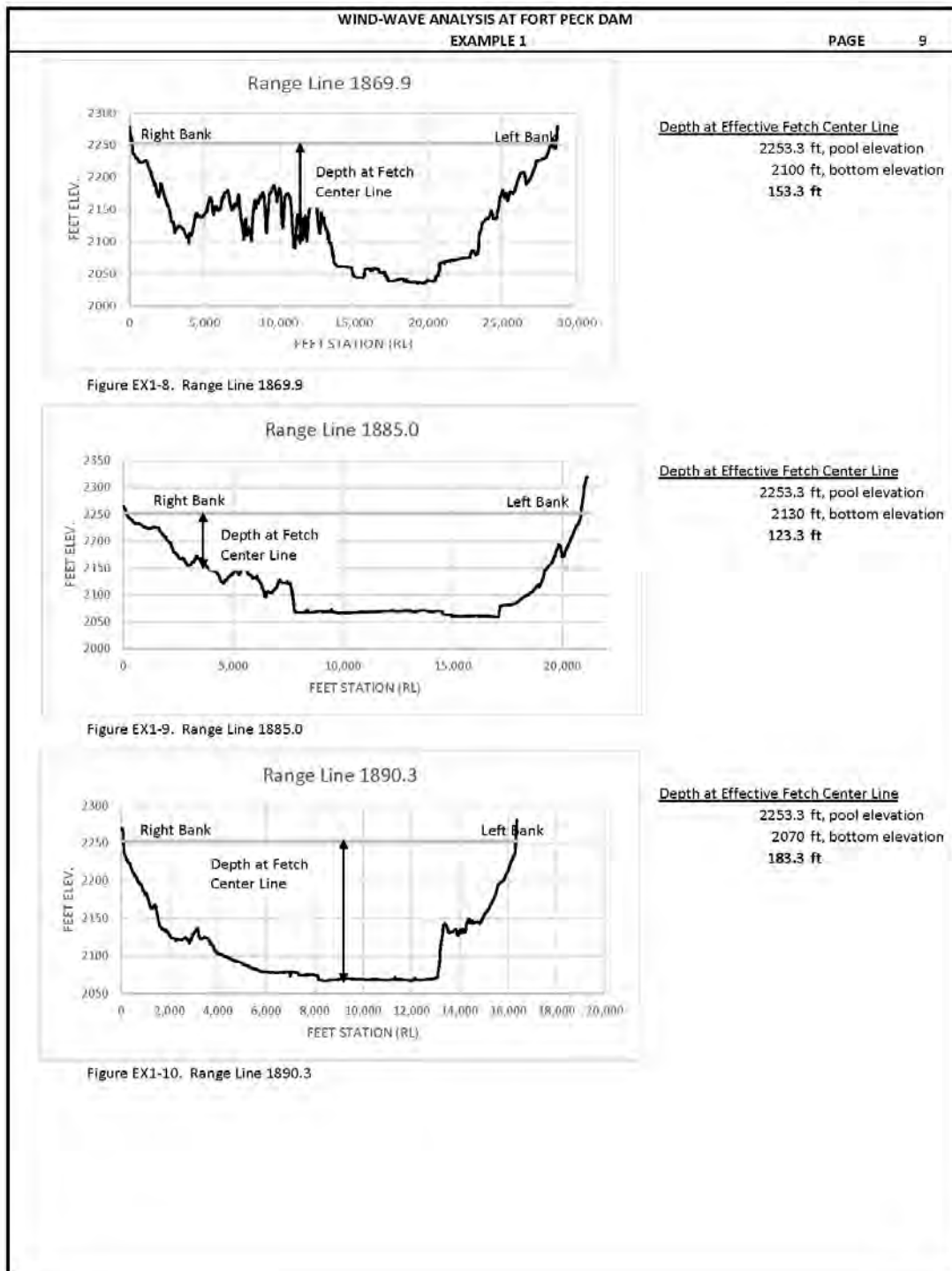


Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1 (Continued).

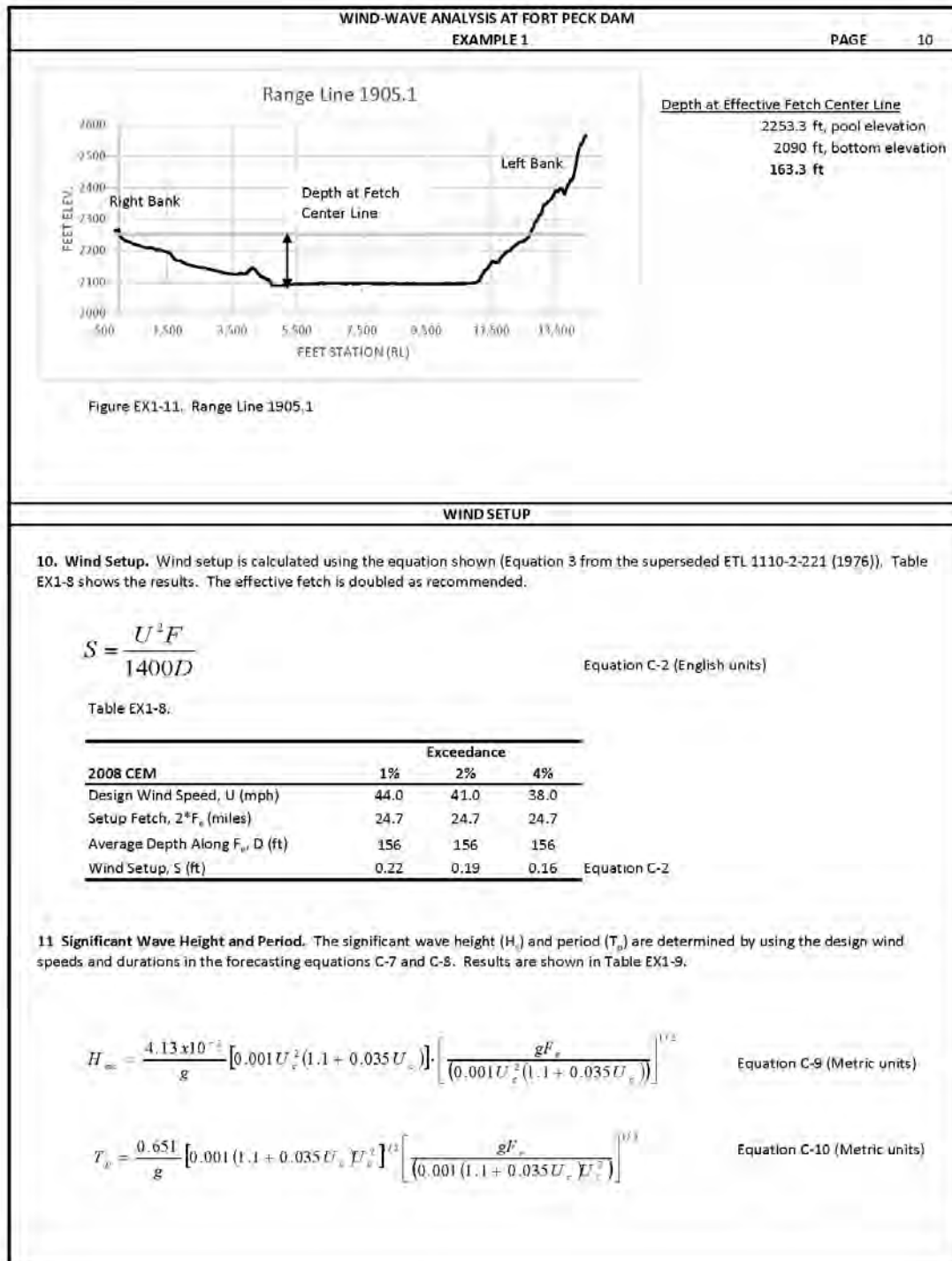


Figure C-11. Wind-Wave Analysis at Fort Peck Dam, Example 1 (Continued).

WIND-WAVE ANALYSIS AT FORT PECK DAM				
EXAMPLE 1				PAGE 11
Table EX1-9. Significant wave height and period				
2014 CEM	Exceedance			
	1%	2%	4%	
Design wind speeds, mph	44.0	41.0	38.0	
Design wind speeds, m/s	19.7	18.3	17.0	
Design durations, hrs	2.8	2.9	3.0	
H_w , m	1.55	1.42	1.30	Equation C-9
H_w , ft	5.07	4.67	4.27	
T_w , sec	3.6	3.5	3.4	Equation C-10

12. **Theoretical Deep Water Wavelength.** The theoretical deep water wavelength, L_w , is calculated to make sure that the assumption of deep-water conditions was correct. This calculation uses the significant wave period determined previously.

The reservoir is considered to be deep if the average depth of the reservoir along the effective fetch length is larger than one-half of the theoretical deep-water wavelength ($D > 1/2 L_w$). Table EX1-10 shows the calculated theoretical deep water wavelengths. Table EX1-11 shows the check for deep water conditions.

$$L_w = \frac{gT_w^2}{2\pi} \quad \text{Equation C-13}$$

Table EX1-10. Theoretical deep water wavelength

2014 CEM	Exceedance			
	1%	2%	4%	
T_w , sec	3.6	3.5	3.4	
L_w , ft	67	63	60	Equation C-13

Equation C-9 used for L_w

Table EX1-11. Check of Deep Water Assumption

2014 CEM	Exceedance			
	1%	2%	4%	
$D + S$ (ft)	156.02	155.99	155.96	
$1/2 L_w$	33.6	31.7	29.9	

$D > 1/2 L_w$, therefore deep water conditions

Figure C-11. Wind-Wave Example at Fort Peck Dam, Example 1 (Continued).

WIND-WAVE ANALYSIS AT FORT PECK DAM

EXAMPLE 1

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13. Extreme wave height. The extreme wave height, $H_{2\%}$, is estimated using Table 1 of the superseded ETL 305 (1984). It is recommended that the 2% of wave exceeding the specific wave height, H_s , be used in the calculation. The factor of $H_{2\%}/H_s$ is 1.40, Table EX1-12.

Table EX1-12.

Significant Wave Height, H_s			Extreme Wave Height, $H_{2\%}$		
1%	2%	4%	1%	2%	4%
5.1	4.7	4.3	7.1	6.5	6.0

Equation C-14

WAVE RUNUP ON SLOPED EMBANKMENTS

14. Embankment Slope. The slope of the embankment affects wave run-up. Determine the slope of the dam embankment, spillway, or shoreline, etc., over which the wave will pass. In the case of Fort Peck Lake, the embankment slope varies between 1 on 7, 1 on 45 and 1 on 3.5 (v:h) according to the Operation and Maintenance manual. The slope of 1 on 3.5 will be used.

15. Wave Run-up. The wave run-up on the Fort Peck Lake embankment is determined using the two equations from EM 1110-2-1100. Values used in the calculations are summarized in Table EX1-13. Results are shown in Table EX1-14.

Table EX1-13.

2014 CEM	Exceedance			
	1%	2%	4%	
H_s , ft	5.1	4.7	4.3	Note that these are converted to metric units for the calculations.
T_g , sec	3.6	3.5	3.4	
L_o , ft	67	63	60	

Table EX1-14. Run-up calculations (Metric units used in calculations and then converted)

2014 CEM	Exceedance			
	1%	2%	4%	
Surf-similarity parameter	1.0	1.1	1.1	Equation C-17
Significant wave A parameter	1.35	1.35	1.35	
Significant wave C parameter	0	0	0	From Table VI-5-3 in EM 1110-2-1100
Extreme wave A parameter	1.60	1.60	1.60	
Extreme wave C parameter	0	0	0	Equation C-15
Reduction factor	0.55	0.55	0.55	
Significant wave run-up, R_s (ft)	3.9	3.7	3.4	Equation C-15
Extreme wave run-up, $R_{2\%}$ (ft)	4.6	4.3	4.0	

$$\xi = \frac{\tan \theta}{\sqrt{\frac{2\pi H_s}{g T_s^2}}}$$

Note that these are Metric units equations.

Equation C-17 (Metric units)

$$\frac{R_{m*0}}{H_s} = (A\xi + C) \gamma_r \gamma_b \gamma_n \gamma_\beta$$

Equation C-15 (Metric units)

Figure C-11. Wind-Wave Example at Fort Peck Dam, Example 1 (Continued).

WIND-WAVE ANALYSIS AT FORT PECK DAM

EXAMPLE 1

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Dam Freeboard Needed for Wave Run-up and Wind Setup

Final results for runup and setup are summarized in Table EX1-15.

Table EX1-15.

	Exceedance		
	1%	2%	4%
Effective Fetch Length (mi)	12.4	12.4	12.4
Design Wind Speed (mph)	44.0	41.0	38.0
Significant Wave Height, H_s (ft)	5.1	4.7	4.3
Extreme Wave Height, H_{2+10} (ft)	7.1	6.5	6.0
Significant Wave Run-up, R (ft)	3.9	3.7	3.4
Extreme Wave Run-up, R_{2+10} (ft)	4.6	4.3	4.0
Wind Setup, S (ft)	0.22	0.19	0.16
Final Design Value (ft)	4.9	4.5	4.2

Top of inflow design flood pool (ft)	2,253.3	ft
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	Exceedance		
	1%	2%	4%
Top of Dam (ft)	2,258.2	2,257.8	2,257.5

Results show that Fort Peck dam needs at least 4.9 ft of freeboard for wave run-up and wind setup.

The final freeboard required depends on guidance in ER 1110-8-2 which must be referenced.

Overtopping Discharge

16. Overtopping Discharge. Use the van der Meer and Janssen formula to calculate overtopping. This is from Table VI-5-11 of EM 1110-2-1100. Table EX1-16 shows the overtopping results.

For a surf similarity parameter less than 2,

$$q = 0.06 \cdot \exp \left(-5.2 \frac{R_s}{H_s} \sqrt{\frac{2\pi H_s}{gT_s}} \frac{1}{\tan \alpha} \frac{1}{\gamma_s \gamma_b \gamma_h \gamma_p} \right) \cdot \sqrt{gH_s^3} \cdot \sqrt{\frac{\tan \alpha}{\left(\frac{2\pi H_s}{gT_s} \right)}}$$

application range:

$$0.3 < \frac{R_s}{H_s} \sqrt{\frac{2\pi H_s}{gT_s}} \frac{1}{\tan \alpha} \frac{1}{\gamma_s \gamma_b \gamma_h \gamma_p} < 2$$

Figure C-11. Wind-Wave Example at Fort Peck Dam, Example 1 (Continued).

WIND-WAVE ANALYSIS AT FORT PECK DAM

EXAMPLE 1

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Table EX1-16.

	Exceedance		
	1%	2%	4%
Freeboard, R_c (m)	1.5	1.4	1.3
Significant Wave Height, H_s (m)	1.5	1.4	1.3
Wave Period, T (sec)	3.6	3.5	3.4
Angle alpha, α (radians)	0.2783	0.2783	0.2783
Reduction factor	0.55	0.55	0.55
Overtopping Discharge, q (m^3/s per meter)	8.7E-05	8.4E-05	8.1E-05

Range Check

1.7 Equations are valid

Based on information in Table VI-5-6 in EM 1110-2-1100 this overtopping discharge would make it unsafe to drive at high speed on the embankment during overtopping.

Figure C-11. Wind-Wave Example at Fort Peck Dam, Example 1 (Continued).

GLOSSARY

Term	Definition
ACES	Automated Coastal Engineering System
AEP	Annual Exceedence Probability
AF	Acre-Feet
ASCE	American Society of Civil Engineers
AWWA	American Water Works Association
CECW	Directorate of Civil Works, US Army Corps of Engineers
CFR	Code of the Federal Regulations
COOP	Cooperative Observer Network
CPD	Computer Program Documentation
CRREL	Cold Regions Research and Engineering Laboratory
CWMS	Corps Water Management System
DEM	Digital Elevation Model
EAD	Expected Annual Damage
EAP	Emergency Action Plan
EC	Engineer Circular
EM	Engineer Manual
USEPA	U.S. Environmental Protection Agency
ER	Engineer Regulation
ERDC	U.S. Army Engineer Research and Development Center
EROS	Earth Resources Observation and Science
ETL	Engineer Technical Letter
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FRA	Flood Risk Analysis
GIS	Geographic Information System
GSSHA	Gridded Surface Subsurface Hydrologic Analysis (model)
GTL	Guide Taking Lines

Term	Definition
GUI	Graphical User Interface
H&H	Hydrologic and Hydraulic
HEC	Hydrologic Engineering Center
HEC-FDA	Hydrologic Engineering Center Flood Damage Reduction Analysis
HEC-FIA	Hydrologic Engineering Center Flood Impact Analysis
HEC-GeoHMS	Hydrologic Engineering Center Geospatial Hydrologic Modeling Extension
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-PRM	Hydrologic Engineering Center Prescriptive Reservoir Model
HEC-RAS	Hydrologic Engineering Center River Analysis System
HEC-SSP	Hydrologic Engineering Center Statistical Software Package
HEC-WAT	Hydrologic Engineering Center Watershed Analysis Tool
HMR	Hydrometeorological Report
HEC-ResSim	Hydrologic Engineering Center Reservoir System Simulation
HQUSACE	Headquarters, U.S. Army Corps of Engineers
HUC	Hydrologic Unit Code
IDF	Inflow Design Flood
IfSAR	Interferometric Synthetic Aperture Radar
IHD	International Hydrological Decade
InSAR	Interferometric Synthetic Aperture Radar
IPS	(National Oceanic and Atmospheric Administration) Images and Publications System
LCI	(U.S. Geological Survey) Land Cover Institute
LiDAR	Light Detection and Ranging
M&I	Municipal and Industrial
MAF	Million Acre-Feet
MCL	Maximum Contaminant Level
MMC	Modeling, Mapping, and Consequences
MOVE	Maintenance of Variance Extension

Term	Definition
MRLC	Multi-Resolution Land Characteristics Consortium
NAPP	National Aerial Photography
NATSGO	National Soil Geographic
NBI	National Bridge Inventory
NCEI	National Centers for Environmental Information
NED	National Elevation Dataset
NEXRAD	Next-Generation Radar
NHD	National Hydrography Dataset
NID	National Inventory of Dams
NLCD	National Land Cover Data
NOAA	National Oceanic and Atmospheric Administration
NOHRSC	(National Weather Service) National Operational Hydrologic Remote Sensing Center
NOMADS	(National Oceanic and Atmospheric Administration) National Operational Model Archive & Distribution System
NRCS	Natural Resources Conservation Service
NSCEP	(U.S. Environmental Protection Agency) National Service Center for Environmental Publications
NSM	Nutrient Sub Module
NWIS	(U.S. Geological Survey) National Water Information System
NWS	National Weather Service
PE&RS	Photogrammetric Engineering & Remote Sensing
PFDS	(Hydrometeorological Design Studies Center) Precipitation Frequency Data Server
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PRISM	Parameter-elevation Regressions on Independent Slopes Model
PRM	Prescriptive Reservoir Model
PSU	Portland State University

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Term	Definition
SCA	Snow Covered Area
SES	Senior Executive Service
SNODAS	Snow Data Assimilation System
SNOTEL	Snowpack Telemetry
SOP	Standing Operating Procedure
SPF	Standard Project Flood
SPS	Standard Project Storm
SSARR	Streamflow Synthesis and Reservoir Regulation
SSURGO	Soil Survey Geographic (database)
STATSGO	State Soil Geographic (database)
SWE	Snow Water Equivalent
TD	Technical Document
TP	Technical Paper
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
UTM	Universal Transverse Mercator
WES	World Education Studies
WMO	World Meteorological Organization