D.9. U.S. Bureau of Reclamation

conditions between borings requiring additional treatment and/or design modification.

9. It is time that the civil engineering profession in the area of dam design respond with action programs rather than reacting to disasters and analyzing why the situation occurred. This can only be achieved with adequate funding to provide for the proposed Phase I and following Phase II dam inspection and remedial works as outlined by the speakers from the Corps of Engineers. These funds must be generated so that the public safety can be adequately protected.

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BUREAU OF RECLAMATION'S CRITERIA FOR THE DESIGN OF SPILLWAYS

By Erwin Thomas Scherich[1/]

Of the Bureau of Reclamation's 340 plus storage dams about 30 meet our definition of small dams - up to 50 feet in height above streambed and up to 1,000,000 cubic yards embankment volume. We also have designed over 136 diversion dams - nearly all meeting our definition of small dams. Of the small storage dams, storage capacities range from 100 acre-feet to 527,000 acre-feet. The dam impounding a 527,000 acre-foot reservoir is only 37 feet high. From a safety standpoint volume of storage, in most cases, should be of more concern than height of dam or volume of embankment, and we have no quarrel with the limitation of 1,000 acre-foot storage for the small dam category used by other agencies.

Our earliest storage dam, Minidoka, was completed in 1906. It is 48 feet high, has an embankment volume of 257,300 cubic yards, and creates a reservoir of 210,000 acre-foot capacity. I mention this early dam to focus attention on the fact that when design criteria are evaluated we must keep in mind that dams are not designed for a certain number of years of useful life. After a service of two, three, or many times the repayment period, dams are expected to continue to function in a safe manner. Dams and their appurtenant

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structures should be designed to give satisfactory and safe service for an indefinite period. Federal agencies continue to have responsibility for dams they design, or are placed under their supervision, after operation and maintenance have been transferred to the owners, perhaps an irrigation district. This accounts, at least in part, for a tendency toward conservatism in design.

Even with this tendency to be conservative in our design assumptions, we have 86 dams under study primarily to determine if they can safely handle newly prepared inflow design floods. Inadequate spillway capacity has proved to be a major cause for concern. I am convinced that the greatest single deficiency in dams throughout the world is inadequate spillway capacity.

In small as well as large dams the first and most important concern of the designer should be the capability, by temporary surcharge storage in conjunction with releases through spillways and outlet works, to safely handle the inflow design flood. The inflow design flood may be the maximum probable flood, a less than maximum probable flood, or even a definite frequency flood, for example, 100-year frequency.

The flood selected for the spillway design is obviously the key to the dam safety, and to a great extent, the structural cost, and in many cases, the project cost. The selection of the inflow design flood involves consideration of dam height, storage capacity of the reservoir behind the dam, type of development - present and projected - in the floodplain downstream from the dam, potential hazard to life

in case of failure, and importance of the value of water stored behind the dam. The following criteria are used by the Bureau of Reclamation for selection of the inflow design flood:

1. Where failure of the dam would endanger human life and/or would cause excessive property damage, a maximum probable flood is used to size surcharge space and spillway capacity.

2. Where failure of the dam would not endanger human life, and would not cause heavy property damage to present or to projected future development below the dam but would impair the usefulness of the project, a less than maximum probable flood may be used to size surcharge space and spillway capacity provided a minimum freeboard will result (depending on type and construction of dam) when the maximum probable flood is routed through the reservoir.

3. Where failure of the dam would not endanger human life, would not cause significant property damage, or seriously impair the usefulness of the project such as the supply of water to a municipality or other critical use or where repair or replacement would not involve major costs or delays in the supply of water, such as a diversion dam, a specific frequency flood (50- or 100-year frequency) may be used to size the surcharge space and the spillway.

Criterion No. 2 is currently (1974) being used for the design of

Palmetto Bend Dam in Texas, a dam of intermediate height - 64 feet above streambed - but large capacity and embankment volume - 170,500 acre-feet and 5,436,000 cubic yards. The less than maximum probable flood was used to establish the dam crest with a normal 8 feet of freeboard allowance for a gated spillway. The maximum probable flood results in a 3-foot encroachment on the freeboard - a 5-foot encroachment would have been considered the maximum for the existing conditions.

After having decided on the appropriate inflow design flood to use in establishing surcharge space and spillway capacity, the second step is to determine a freeboard allowance that will satisfy conditions that exist for the proposed dam. These conditions include computed wave heights based on fetch and wind velocity assumptions, provisions for settlement where poor foundation conditions exist, an arbitrary allowance for improper gate operation where applicable, and perhaps an arbitrary allowance for contingencies to provide an increased factor of safety.

The third step is to determine the combination of surcharge storage and spillway capacity which will result in the least total project cost. This is the least total cost for the level of safety established for conditions that exist for a particular dam and involves comparative cost estimates for various sizes and types of spillways and combinations with outlet works of various sizes and types. Ungated spillways should be favored because of greater reliability over gated spillways where the economic choice is not clearly indicated by the cost difference. It is acceptable Bureau of

Reclamation practice to use auxiliary spillways and flood outlets in combination with service spillways where site and operating conditions are favorable. It is also acceptable practice to store the volume of the inflow design flood provided flood outlet capacity is adequate to evacuate the volume of a 100-year frequency flood antecedent to or subsequent to the inflow design flood, and provided further that the flood outlet is considered to be highly reliable. Where no spillway is provided evacuation of surcharge space must be accomplished at the maximum rate possible for the outlet works. In some cases an emergency spillway may be added to increase the safety against overtopping. Outlet works reliability and requirements for use of auxiliary and emergency spillways are discussed further in later paragraphs.

The three steps that have been discussed establish the level of safety for the dam, assuming the spillways and outlet works function in a manner that will not endanger the integrity of the dam. The design criteria for these structures will now be considered.

The service spillway control structure must have the capacity assumed in the flood routings and must safely convey the flood releases to the stream beyond the toe of the dam. These criteria involve careful crest design and gating provisions where applicable; adequate chute capacity with allowance for freeboard; and adequate energy dissipation for the conditions that exist in the stream and at the downstream toe of the dam.

Where an auxiliary spillway is used in combination with the service spillway it is current practice in the Bureau to set the free overflow control level, or the reservoir level where auxiliary spillway gates are to be put into operation, at the maximum water surface elevation for a 100-year frequency flood or some other specific frequency flood routed through the service spillway. The auxiliary spillway thus will only have infrequent use and should be designed to minimum standards provided the following requirements are met:

1. The auxiliary spillway will discharge into a water course sufficiently separated from the dam abutment and discharging into the main stream a sufficient distance downstream from the toe of the dam that maximum releases will not endanger the dam integrity.

2. The control weir and spillway channel are separated from the dam and service spillway a sufficient distance or by adequate rock formation to insure that maximum releases will be contained in the auxiliary spillway channel and not endanger the service spillway.

3. The auxiliary channel is in adequate rock or is adequately protected by grass lining or other protective surfacing to withstand maximum flows to the degree that the reservoir control does not degrade to a level below the top of conservation storage and further to the extent that restoration of the channel to original grade and condition is feasible.

4. A permanent grade sill or overflow crest is provided to maintain a positive control level.

Where the volume of the inflow design flood is temporarily stored in the surcharge space of the reservoir, the outlet works must be highly reliable and of adequate capacity to evacuate a 100-year frequency flood antecedent to or subsequent to the inflow design flood. The time interval between peaks is established by hydrometeorological determination. The outlet works must also be operated at full capacity to evacuate the surcharge capacity as rapidly as possible.

Where an emergency spillway is used for additional safety, the control is usually set at the maximum water surface for the inflow design flood routed through the reservoir. The emergency type spillway is rarely used in Bureau designs. It would be appropriate where storage is provided for the entire flood volume and marginal evacuation facilities are provided or where a small service spillway can become clogged. We consider the auxiliary type, which by our definition is intended to function on a specific frequency basis, more desirable and may be more economic where flood volumes are relatively large with respect to reservoir storage capability.

Normally our policy is to locate spillways on the rock formation where such exists at reasonable depths. In some instances we have decided that the dam embankment may provide better foundation conditions than the natural ground at the site. In recent years we have located free overflow spillways for small and intermediate dam

heights on the embankment. These spillways have rather small capacity and are combined with high capacity flood outlet works. Two examples are Meeks Cabin Dam which is in operation, and Currant Creek Dam on which construction will begin this year. Meeks Cabin Dam is 74 feet high above streambed and has a storage capacity of 33,090 acre-feet. The flood release capacities are 1,070 cfs for the outlet works and 6,250 cfs for the spillway. Currant Creek Dam will be 132 feet high above streambed and has a storage capacity of 15,670 acre-feet. The flood release capacities are 5,540 cfs for the outlet works and 850 cfs for the spillway.

Of somewhat less importance than the foregoing considerations are the design details of the control structure; conveyance chute, conduit or tunnel; energy dissipator; and upstream and downstream channels. These features become of greater significance with increases in dam height. Individual spillway features should be designed to give the level of performance and safety that can be justified by the conditions that exist for a particular dam. Factors that should be considered include frequency of operation and type of flood used in establishing the dam crest elevation.

This means that the inflow design flood - maximum probable, less than maximum probable, or definite frequency - used in sizing surcharge space and spillway capacity should also be the design flood for the individual features. As in our practice for establishing dam freeboard, however, the maximum probable flood must be accounted for. For example, if the less than maximum probable flood is used for

sizing surcharge storage capacity and spillway capacity, and normal freeboard is provided to establish the crest of the dam, encroachment on freeboard is permitted up to a minimum established value for the maximum probable flood routed through the reservoir. All of the features of the spillway should be designed to normal standards for the less than maximum probable flood and to reduced but safe standards for the maximum probable flood. Likewise, performance standards should be higher for a spillway where reservoir operation studies indicate there will be frequent spills, than if the studies indicate the spillway will be rarely used.

The control structure may be a simple weir or may be of more complex design with automatic gates and precisely designed flow surfaces. Ordinarily, large discharge capacities and high heads will require much more refinement in the hydraulic design than small discharge and low heads. The designer has many types of spillway control structures to choose from.

Spillways are frequently classified according to the type of control (free overflow or gated) and according to type of conveyance (chute, tunnel, or conduit). In addition to these general classifications, the control structure may be one of the following:

1. Ogee crest - straight or curved
2. Broad crest or flat crest
3. Side channel or double side channel
4. Drop inlet, often called shaft or morninglory
5. Siphon

6. Straight drop inlet

Types of energy dissipators are as numerous and include the following:

1. Hydraulic jump stilling basin
2. Impact type dissipator
3. Deflector bucket
4. Submerged deflector bucket
5. Plunge pool
6. Baffled chute

The most appropriate combination depends upon topographic features of the site, the relationship of storage capacity to flood volume, geologic conditions, degree of protection desired, type of dam, height of dam, and many other factors peculiar to a particular site. The experienced designer has many highly developed options to choose from in providing a desired level of protection for a dam. In the Bureau of Reclamation we prefer to select protective measures which can be designed to reasonably assure a degree of risk which can be evaluated in terms of progressive structural damage without endangering human life or causing excessive property loss.

My discussion of spillway designs has intentionally included spillways for large dams and large reservoirs in order to avoid limitations as to applicability. For small dams of 40-foot hydraulic height and less than 1,000 acre-feet of normal storage capacity, the chances of a catastrophic failure will ordinarily be much less than for a large dam. However, I have observed some very questionable residential zoning immediately downstream from dams of this size.

Small dams can be designed to withstand a small amount of overtopping without rapid loss of water in storage. For example, compacted cohesive embankments provide considerable resistance to erosion. Where accurate predictions of release rates can be made, use of specific frequency floods (50 to 100 years) may frequently be justified for establishing surcharge storage space and spillway capacity. We would use a maximum probable flood for evaluation of overtopping risks.

Spillways for small dams usually are of simple design but should have adequate provisions for safe release of the routed inflow design flood to the stream below the dam. Two feet of freeboard for the dam may be adequate if the reservoir storage capacity is small. We consider an uncontrolled overflow crest superior to a gated flood outlet, but where a gated flood outlet is used, provisions can ordinarily be made for free overflow in the gate shaft at very little added expense. This overflow weir is usually located at the top of a divider wall in the gate shaft and at the top of active storage to prevent storage in the surcharge space when the gates are closed. The need for energy dissipation should always be investigated in order that the downstream toe of the dam or the end of the spillway is not subjected to damaging erosion conditions.

The design criteria that I have discussed apply to new construction. It is desirable that existing dams meet these criteria but a different set of economic factors may exist for an old dam than for a dam that is being planned. For example, the ability of the dam owner to finance expensive modifications may be severely limited or the

ratio of benefits to costs may not justify continued operation of the project. Faced with such conditions, the reasonable approach is to evaluate the probable results of continued unsafe operation.

The Bureau of Reclamation has established systematic evaluation procedures that are used for dams that do not meet acceptable standards for safe operation. Deficiencies are usually discovered by routing an updated inflow design flood through the reservoir. In areas of high seismic risk a seismic study is made in addition to the flood routing study.

Before corrective measures are recommended, freeboard allowances are reviewed taking into account the known condition of the dam and other factors that have become known during the operating life of the dam. Operating procedures are examined to determine if an acceptable level of safety can be produced by a limited reservation of storage space during periods of high flood risk. If it is finally determined that overtopping will occur for the inflow design flood, the failure mode and rate are estimated and time-stage relationship is computed for the flow through the breached dam. Downstream loss of life and property damage is then estimated. Finally a decision on the best course of action may be made on the basis of the following alternatives:

1. Modification of dam and spillway to a satisfactory level of safety and performance. (This alternative includes replacement of the dam and construction of additional upstream storage.)

2. Continue to operate the project with reduced storage or other operational adjustments which will assure safety.

3. Continue to operate the project without modification but with knowledge of effects of overtopping. Where human life is endangered or excessive property loss would result from overtopping the dam, this alternative is not considered to be acceptable.

4. Dewater reservoir, demolish dam, and restore site to acceptable environmental standards.

My discussion of spillway design criteria has been focused on those elements that are critical with respect to dam safety. Of greatest importance is the selection of the appropriate inflow design flood for the level of safety indicated by such factors as value of stored water and risks to property and human life in the flood plain below the dam. This desired level of safety can only be assured by adequate provisions for temporarily storing and/or releasing flood water. Flood releases must be conveyed over, under, or around the dam to the stream below in such a manner that the dam will not be damaged. To economically accomplish this the designer has at his disposal many highly developed types of spillways and flood outlets that can be used separately or in combination. For large and small dams alike the spillway should be considered equal in importance to the dam. No dam should be built without adequate provisions for safely handling a major flood.

RECLAMATION

Managing Water in the West

Design Standards No. 14

Appurtenant Structures for Dams (Spillway and Outlet Works) Design Standards

Chapter 1: Introduction
Draft: Phase 3 (Public and Stakeholders Review)

U.S. Department of Interior
Bureau of Reclamation

March 2010

MISSION STATEMENTS

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Design Standards Signature Sheet

Design Standards No. 14

Appurtenant Structures for Dams (Spillways and Outlet Works) Design Standards

DS-14(1)-2: Draft: Phase 3 (Public and Stakeholders Review)
March 2010

Chapter 1: Introduction

Approved:

Deputy Commissioner, Operations	**Date**

Chapter Signature Sheet
Bureau of Reclamation
Technical Service Center

Design Standards No. 14

Appurtenant Structures for Dams (Spillways and Outlet Works) Design Standards

Chapter 1: Introduction

DS-14(1)-2:[1] Draft: Phase 3 (Public and Stakeholders Review)
March 2010

Design Standard 14 is a new document. Chapter 1 of this Design Standard was developed to provide:

- An overview of how the Bureau of Reclamation analyzes/designs spillways and outlet works appurtenant for dams and/or dikes.
- An outline of the following chapters for this design standard that provides details for spillway and outlet works analyses/designs.
- A list of key technical references used for each major task involved with spillway and outlet works analyses/designs.

[1] DS-14(1)-2 refers to Design Standard No. 14, chapter 1, revision 2.

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Contents

Page

Chapter 1: Introduction 1-1
1.1 Purpose 1-1
1.2 Application of Design Standards 1-1
1.3 Deviations and Proposed Revisions 1-1
1.4 Scope 1-2
1.5 Definitions 1-2
1.5.1 Spillway 1-2
1.5.1.1 Service Spillway 1-3
1.5.1.2 Auxiliary Spillway 1-3
1.5.1.3 Emergency Spillway 1-3
1.5.2 Outlet Works 1-3
1.6 Configurations 1-4
1.6.1 Spillway 1-4
1.6.2 Outlet Works 1-5
1.7 Design Procedures/Considerations 1-7
1.7.1 Spillway Design/Analysis 1-7
1.7.1.1 Location, Type and Size. 1-7
1.7.1.2 Hydraulic Analysis/Design 1-8
1.7.1.3 Foundation Analysis/Design 1-10
1.7.1.4 Structural Analysis/Design 1-11
1.7.1.5 Mechanical/Electrical Design 1-13
1.7.1.6 Risk Analysis (only for significant and high hazard dams/dikes) 1-15
1.7.2 Outlet Works Design/Analysis 1-17
1.7.2.1 Location, Type and Size. 1-17
1.7.2.2 Hydraulic Analysis/Design 1-19
1.7.2.3 Foundation Analysis/Design 1-21
1.7.2.4 Structural Analysis/Design 1-22
1.7.2.5 Mechanical/Electrical Design. 1-23
1.7.2.6 Risk Analysis (only for significant and high hazard dams/dikes) 1-25

References 1-39

Figures

Page

1.5.1.1-1 Example: Service spillway (gated) – Jackson Lake Dam, Wyoming..... 1-28
1.5.1.1-2 Example: Service spillway (ogee crest) – Crystal Dam, Colorado..... 1-28
1.5.1.2-3 Example: Auxiliary spillway (gated) in foreground, service spillway (gated) in background – Stewart Mountain Dam, Arizona..... 1-29
1.5.1.2-4 Example: Auxiliary spillway (grade control sill) – Heart Butte Dam, North Dakota 1-29
1.5.1.3-5 Example: Emergency spillway (fuseplug) in foreground, auxiliary spillway (ogee crest) in background – New Waddell Dam, Arizona 1-30
1.5.1.3-6 Example: Emergency spillway – San Justo Dam, California . 1-30
1.5.2-7 Example: Combination tunnel outlet work and power penstock, highlighting a number of features such as the upstream intake structure and the downstream control structures. Theodore Roosevelt Dam, Arizona..... 1-31
1.6.1-8 Common features of spillway 1-32
1.6.1-9 Ogee crest (uncontrolled) spillway – Bumping Lake Dam, Washington 1-33
1.6.1-10 Double side-channel (bathtub) crest (uncontrolled) spillway – Fontenelle Dam, Wyoming 1-33
1.6.1.-11 Side-channel crest (uncontrolled) spillway – Big Sandy Dam, Wyoming 1-34
1.6.1-12 Morning glory (glory hole) crest (uncontrolled) spillway – Whiskeytown Dam, California..... 1-34
1.6.1-13 Labyrinth crest (uncontrolled) spillway – Ute Dam, New Mexico..... 1-35
1.6.1-14 Radial gated (controlled) spillway – Stewart Mountain Dam, Arizona..... 1-35
1.6.1-15 Crest (Obermeyer type) gated (controlled) spillway – Friant Dam, California..... 1-35
1.6.1-16 Drum gated (controlled) spillway – Hoover Dam, Arizona-Nevada 1-35
1.6.1-17 Fuseplug crest (controlled) spillway (reservoir rim) – Horseshoe Dam, Arizona..... 1-36
1.6.1-18 Fusegate (controlled) spillway – Terminus Dam, California... 1-36
1.6.2-19 Common features of outlet works..... 1-37
1.6.2-20 Preferred outlet works configuration: hydraulic control at upstream intake with free flow conditions downstream of the regulating gate/valve 1-38

Figures

Page

1.6.2-21 Preferred outlet works configuration: hydraulic control at downstream control structure, with guard/emergency gate/valve at/near centerline of dam/dike, and downstream pressurized pipe (between dam/dike centerline and control structure inside larger access conduit) 1-38

1.6.2-22 Acceptable outlet works configuration: hydraulic control at/near centerline of dam/dike with free flow conditions downstream of the regulating gate/valve 1-38

1.6.2-23 Least acceptable outlet works configuration: hydraulic control at downstream control structure (i.e., pressurized flow conditions upstream of the regulating gate/valve along most of the outlet works) 1-38

Chapter 1

Introduction

1.1 Purpose

The design standards present clear and concise technical requirements and processes to enable design professionals to prepare design documents and reports necessary to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public. Compliance with these design standards assists in the development and improvement of Bureau of Reclamation (Reclamation) facilities in a way that protects the public's health, safety, and welfare; recognizes all stakeholder needs; and achieves the lasting value and functionality necessary for Reclamation facilities. The responsible designer(s) accomplishes this through processes that enable compliance with these design standards and all other applicable technical codes, as well as incorporation of the stakeholder's vision and values, that are then reflected in the construction project.

1.2 Application of Design Standards

All Reclamation design work, whether performed by the Technical Service Center (TSC), the regional offices, or an architectural/engineering (A&E) firm, will conform to the design standards.

Reclamation's use of its design standards requires designers to also integrate sound engineering judgment with applicable national standards, site-specific technical considerations, and project-specific considerations to ensure suitable designs and protect public safety.

The design standards are not intended to provide cookbook solutions to complex engineering problems. Strict adherence to a handbook procedure is not a substitute for sound engineering judgment. The designer should be aware of and use state-of-the-art procedures. Designers are responsible for using the most current edition of referenced codes and standards and to be aware that Reclamation design standards may include exceptions to requirements of these codes and standards.

1.3 Deviations and Proposed Revisions

Design activities must be performed in accordance with established Reclamation design criteria, Reclamation engineering, architectural, or technical standards, and approved national design standards. Exceptions to this requirement will be

pursued in accordance with provisions of Reclamation Manual Policy, *Performing Designs and Construction Activities*, FAC P03.

Reclamation designers should inform the TSC, via the Web site notification procedure, of any recommended updates or changes for the design standards to meet current design practices.

1.4 Scope

Design Standard No. 14 provides technical guidance concerning Reclamation's procedures/considerations for analyzing/designing two key appurtenant structures associated with dams and/or dikes. These appurtenant structures are spillways and outlet works. Chapter 1 provides an overview for the analysis/design of spillways and outlet works, while the following chapters provide detailed procedures/considerations that should be followed by Reclamation staff and others involved with analyzing/designing modifications to and/or new spillways and outlet works. It should be stressed that this design standard will not duplicate other existing technical references but, wherever possible, it will reference existing procedures/considerations that should be used for the analysis/design of spillways and outlet works.

1.5 Definitions

The following definitions are provided to clarify the terminology used in Design Standard No. 14. These definitions are consistent with other technical references used by Reclamation.

1.5.1 Spillway

A spillway is a hydraulic structure that passes normal (operational) and/or flood flows in a manner that protects the structural integrity of the dam and/or dikes (reservoir impoundment structures). Spillways are hydraulically sized to safely pass the Inflow Design Flood (IDF).[2] The IDF will be equal to, or less than, the Probable Maximum Flood (PMF).[3] For more details and guidance about floods, refer to Chapter 2, "Hydrologic Considerations."

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[2] For significant and high hazard dams and/or dikes and their appurtenant structures, selection of the IDF will be based on a quantitative risk analysis. The IDF will be less than, or equal to, the PMF.

[3] The largest flood that may reasonably be expected to occur at a given maximum runoff condition resulting from the most severe combination of meteorological and hydrologic conditions that are considered reasonably possible for the drainage basin under study.

There are three types or classifications of spillways typically employed by Reclamation, which are based on frequency of use. They are explained in more detail below.

1.5.1.1 Service Spillway

A service spillway provides continuous or frequent regulated (controlled) or unregulated (uncontrolled) releases from a reservoir without significant damage to the dam, dike, or appurtenant structures due to releases up to and including the design discharge. Service spillways are illustrated in figures 1.5.1.1-1 and 1.5.1.1-2.

1.5.1.2 Auxiliary Spillway

An auxiliary spillway is infrequently used and may be a secondary spillway which is operated sparingly. During operation there could be some degree of structural damage or erosion to the auxiliary spillway due to releases up to and including the design discharge. Auxiliary spillways are illustrated in figures 1.5.1.2-3 and 1.5.1.2-4.

1.5.1.3 Emergency Spillway

An emergency spillway is designed to provide additional protection against overtopping of a dam and/or dike and is intended for use under extreme conditions such as misoperation or malfunction of the service spillway or other emergency conditions or during very large, remote floods (such as the PMF). As with auxiliary spillways, some degree of structural damage and/or erosion would be expected due to releases up to and including the design discharge. Emergency spillways are illustrated in figures 1.5.1.3-5 and 1.5.1.3-6.

1.5.2 Outlet Works

Outlet works consist of a combination of features (i.e., intake structure, conveyance features such as conduits, control structure, etc.) and operating equipment (electrical and mechanical) required for the safe operation and control of water released from a reservoir to meet downstream needs. The outlet works serves various purposes such as regulating streamflow and water quality; releasing floodwater; power generation; emergency evacuation; and providing irrigation, municipal, and/or industrial water. Features of outlet works are illustrated in figure 1.5.2-7.

1.6 Configurations

There are some common/typical and unique configurations (features) associated with spillways and outlet works. These features are further discussed in the following sections.

1.6.1 Spillway

Generally speaking, features common to most spillways are illustrated in figure 1.6.1-8 and include:

- Approach channel and safety/debris/log boom.
- Control structure, such as crest structure or grade sill, and gates, bulkheads, stoplogs, along with associated operating equipment.
- Conveyance features, such as chute floor and walls and/or conduit(s)/tunnel(s).
- Terminal structure, such as hydraulic jump stilling basin, flip bucket, plunge pool, etc.
- Downstream channel.

Another consideration of the spillway configuration relates to the type of hydraulic control. With some exceptions,[4] the two types are uncontrolled or free-flow spillways and controlled or gated spillways. The hydraulic control is usually based on whether the spillway crest structure has gates or not. Finally, with some exceptions, the spillway is typically referred to by the type of crest structure, such as:

- For uncontrolled spillways:
    - Ogee crest (figure 1.6.1-9) and grade control sill spillway (figures 1.5.1.2-4 and 1.5.1.3-6).
    - Bathtub (or double side-channel) (figure 1.6.1-10) and side-channel ogee crest spillway (figure 1.6.1-11).
    - Morning glory (or glory hole) spillway (figure1.6.1-12).
    - Labyrinth weir spillway (figure 1.6.1-13).

[4] There are exceptions, such as the morning glory spillway, that could experience hydraulic control shifts with increasing hydraulic head: crest or free flow control to throat or orifice control to pipe or pressure control.

- For controlled spillways:
    - Gated spillway (figures 1.6.1-14 through 1.6.1-16).
    - Fuseplug spillway (figure 1.6.1-17).
    - Fusegate spillway (figure 1.6.1-18).

Consideration should also be given to the location of the spillway, which should not be on or through an embankment dam/dike unless there is justification to deviate. The preferred locations would be on the dam abutments or through the reservoir rim.

For more detailed guidance, refer to Chapter 3, "General Spillway Design Considerations."

1.6.2 Outlet Works

Features common to most outlet works are illustrated in figure 1.6.2-19 and include:

- Intake structures, trashracks, gates/valves, and bulkheads (if appropriate).
- Conveyance features, such as conduit(s)/tunnel(s).
- Control structure, such as gate chamber, gates/valves, access shaft/adit/conduit, along with operating equipment.
- Terminal structure, such as hydraulic jump stilling basin, impact structure, plunge pool, etc.
- Downstream channel.

Considerations that should be used, unless there is justification to deviate, include:

- Two gates or valves in series should be installed and operated in Reclamation outlet works. The downstream gate or valve provides regulating capabilities, while the upstream gate or valve provides emergency closure capabilities under unbalanced head (flow) conditions, or routine closure capabilities under balanced head (nonflow) conditions.
- Although common throughout the water resource engineering industry, constructing an outlet works through/beneath an embankment dam and/or dike should be viewed as a second choice (i.e., avoid/limit contact/interface between the conduit and embankment). A preferred

alternative to minimize internal erosion potential is to construct a tunnel outlet works through the dam and/or dike abutment or through the reservoir rim.

Another aspect of outlet works configuration relates to the location of hydraulic control (i.e., the location of the regulating gate or valve). The four configurations [1][5] typically used by Reclamation are illustrated in figures 1.6.2-20 through 1.6.2-23 and include the following:[6,7]

- **Preferred configuration.** Hydraulic control at intake with free flow conditions downstream of the regulating gate or valve. Most often used for low head applications in embankment dams where pressurized flow is not required at the downstream end of the outlet works. When the outlet works is not being operated, there is access for inspection and maintenance through the entire length of the conduit (figure 1.6.2-20).

- **Preferred configuration.** Hydraulic control at downstream control structure, with guard/emergency gate/valve at/near centerline of dam/dike, and downstream pressurized pipe (between dam/dike centerline and control structure) inside larger access conduit. Applicable for power generation or pressurized downstream flow. During both operation and nonoperation of the outlet works, there is excellent access for inspection and maintenance through downstream conduit (figure 1.6.2-21).

- **Acceptable configuration.** Hydraulic control at/near the dam/dike centerline with free flow conditions downstream of the regulating gate or valve. When the outlet works is not being operated, there is access for inspection and maintenance through the downstream conduit (figure 1.6.2-22).

- **Least acceptable configuration.** Hydraulic control at the downstream end of the outlet works, which may be near the downstream toe or face of the dam/dike (i.e., pressure flow conditions upstream of the regulating gate or valve along most of the outlet works). Commonly used in concrete

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[5] Numbers in brackets indicate a reference listed at the end of this chapter.

[6] Outlet works configurations are listed in order of increasing potential of pressurizing surrounding dam, dike, and/or foundation materials, which could lead to an incident or failure.

[7] Although the figures associated with following outlet works configurations illustrate an outlet works through/beneath an embankment dam/dike (which should be a second choice to a tunnel through the dam/dike abutment or reservoir rim), the focus is on the hydraulic control configuration, which would be applicable regardless of the location of the outlet works.

dams. Accessibility for inspection and maintenance is limited (underwater inspection) without bulkheading upstream intake structure or draining the reservoir (figure 1.6.2-23).

For more detailed guidance, refer to Chapter 4, "General Outlet Works and Diversion Design Considerations."

1.7 Design Procedures/Considerations

Details of analysis/design procedures/considerations will be provided in the following chapters of Design Standard No. 14. An overview is provided in the following text of this chapter. Analysis and/or design (including both new and modification) for a spillway and/or outlet works will typically follow a number of key procedural guidelines including:

- Design data collection guidelines [2].
- Feasibility design guidelines [3].
- Final design process [4].
- Cost estimating [5].
- Safety of Dams project management guidelines [6].

1.7.1 Spillway Design/Analysis

Tasks for analyzing/designing spillways are summarized in the following sections.

1.7.1.1 Location, Type, and Size

Selection of the location, type, and size of a spillway will be dependent on the evaluation of a number of factors including:

- Site conditions (geology/topography).
- Dam and/or dike type.
- Hydrologic considerations.
- Hydraulic considerations.
- Seismic considerations.

- Construction/constructability considerations.
- Project objectives.
- Robustness of design.
- Risks associated with Plausible Potential Failure Modes (PFMs), which must be tolerably below Reclamation's public protection guidelines [7].
- Operation and maintenance considerations.
- Economics.

These factors are further discussed in the following chapters of this design standard and some of the following sections. Two factors that should be highlighted, which are somewhat unique to Reclamation facilities, are the robustness of design and the risks associated with PFMs. Because many of the spillways are associated with significant and high hazard dams/dikes, it is important that any new or modified appurtenant structure designs protect the public to levels consistent with Reclamation's public protection guidelines [7] (i.e., maintain and/or reduce risks to acceptable levels). This could mean some redundant features/equipment and designing to stricter requirements than commonly called for by professional codes, standards, and/or guidelines.

For more detailed guidance associated with locating, along with selecting, a type and size of a spillway, refer to Chapter 3, "General Spillway Design Considerations."

1.7.1.2 Hydraulic Analysis/Design

The hydraulic analysis/design will be concurrent with the previous task of locating and selecting a type and size of spillway. The following chapters of this design standard explain the steps of hydraulic analysis/design, including:

- Develop/verify discharge curves.
- Prepare initial flood routings of frequency floods up to the PMF to verify the appropriateness of the spillway type and size, and to select the IDF.
- Refine spillway control (crest) structure layout and associated discharge curves based on results from previous steps.

- Prepare final flood routings to estimate maximum reservoir water surfaces (RWS) and discharge ranges for various floods and operational conditions (part of the final flood routings will include a freeboard assessment, which is sometimes referred to as a robustness study).

- Prepare initial water surface profiles to lay out the spillway conveyance features and terminal structure size and type.

- Refine spillway conveyance features and terminal structure based on results from previous step.

- Prepare final water surface profiles to finalize size and type of the spillway conveyance features and terminal structure.

For more detailed guidance associated with hydrologic and hydraulic analysis/design for a spillway, refer to Chapter 2, "Hydrologic Considerations," and Chapter 5, "Hydraulic Considerations."

Technical references associated with the hydraulic analysis/design of spillways include:

- *Design of Small Dams*, third edition [8].

- Engineering Monograph (EM) No. 9 – *Discharge Coefficients for Irregular Overfall Spillways* [9].

- EM No. 25 – *Hydraulic Design of Stilling Basins and Energy Dissipators* [10].

- EM No. 42 – *Cavitation in Chutes and Spillways* [11].

- Reclamation – Engineering and Research Center (REC-ERC)-73-5 – *Hydraulic Model Studies of Chute Offsets, Air Slots, and Deflectors for High-Velocity Jets* [12].

- REC-ERC-78-8 – *Low Froude Number Stilling Basin Design* [13].

- REC-ERC-85-7 – *Hydraulic Model Studies of Fuseplug Embankments* [14].

- REC-ERC-88-3 – *Overtopping Flow on Low Embankment Dams – Summary Report of Model Test* [15].

- Dam Safety Office (DSO)-07-07 – *Uplift and Crack Flow Resulting from High Velocity Discharges Over Offset Joints* [16].

- Assistant Commissioner – Engineering and Research (ACER) Technical Memorandum (TM) No. 10 – *Guidelines for Using Fuseplug Embankments in Auxiliary Spillways* [17].

- *Hydraulic and Excavation Tables*, 11th edition [18].

- *Computing Degradation and Local Scour* [19].

- *Guide for Computing Water Surface Profiles* [20].

- *Design of Labyrinth Spillways* [50].

- *Research State-of-the-Art and Needs for Hydraulic Design of Stepped Spillways* [51].

- *Plastic Pipe Used In Embankment Dams: Best Practices for Design, Construction, Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [52].

- *Outlet Works Energy Dissipators: Best Practices for Design, Construction, Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [53].

1.7.1.3 Foundation Analysis/Design

The foundation analysis/design will start when a tentative type and size of spillway is located and selected, and it parallels the structural analysis/design efforts. The following chapters of this design standard explain the steps of foundation analysis/design, including:

- Define site-specific foundation material and strength properties.

- Assist in identifying suitable sites based on site-specific foundation needs for a spillway.

- Define and analyze ground water conditions.

- Develop appropriate foundation designs which include:

    - Drainage features such as underdrains, slope drainage, and filters (NOTE: Identify and evaluate drainage features/systems that will be accessible for inspection using closed circuit television (CCTV) equipment).
    - Adequate support for the spillway, which could range from excavating to competent rock to driving piles or placing piers in a soil foundation.

- Accommodating loading conditions on the spillway that could include static (hydrostatic, rock/soil, ice and frost heave), hydrologic (floods), and seismic (hydrodynamic, dynamic soil/rock) loads.

For more detailed guidance associated with foundation analysis/design for a spillway, refer to Chapter 6, "Foundation Considerations."

Technical references associated with the foundation analysis/design of spillways include:

- *Design of Small Dams*, third edition [8].
- EM No. 13 – *Estimating Foundation Settlement by One-Dimensional Consolidation Tests* [21].
- REC-ERC-74-10 – *Rock Mechanics Properties of Typical Foundation Rock Types* [22].
- REC-ERC-82-17 – *Frost Action in Soil Foundations and Control of Surface Structure Heaving* [23].
- ACER TM No. 9 – *Guidelines for Controlling Seepage Along Conduits Through Embankments* [24].
- *Drainage for Dams and Associated Structures* [25].
- *Guidelines, Foundation and Geotechnical Studies for Existing Concrete Dams* [41].

1.7.1.4 Structural Analysis/Design

The structural analysis/design of the spillway typically follows the hydraulic analysis/design. The following chapters of this design standard explain the steps of structural analysis/design, including:

- Identify which features are considered "critical"[8] and "noncritical."[9] [26]

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[8] A critical feature is one in which its failure or damage could lead to failure and/or damage of the dam/dike or other appurtenant structures, which, in turn, may lead to the uncontrolled release of part or the entire reservoir or leave the appurtenant structure inoperable, preventing releases needed to protect the dam/dike.

[9] A noncritical structure is one in which failure or damage would not lead to failure of and/or damage to the dam/dike, nor would it inhibit operations of the structure to protect the dam/dike.

- Identify and define loading conditions for both critical and noncritical features, which will typically fall into four categories, including:

  - Construction.
  - Operational (normal or static).
  - Flood (hydrologic).
  - Earthquake (seismic).[10]

- Identify, evaluate, and select material types and associated properties. Materials could include concrete (reinforced, conventional mass, roller compacted, and precast), steel, and plastic such as high-density polyethylene (HDPE).

- Apply appropriate structural analysis/design methods, which are based on the crawl, walk, and run philosophy (i.e., using the simplest approach that is technically adequate to prepare the analysis/design). These include:

  - **For analyses.** Pseudo-static, linear elastic finite element modeling (FEM), using response spectra, and nonlinear elastic FEM, using time histories, are employed to analyze a structure.
  - **For designs.** Design methods are typically based on Reclamation technical references, such as *Design of Small Dams* [8], and industry codes are used, such as the American Concrete Institute (ACI) manuals.

For more detailed guidance associated with structural analysis/design for a spillway, refer to Chapter 7, "Structural Considerations."

Technical references associated with the structural analysis/design of spillways include:

- *Design of Small Dams*, third edition [8].

- EM No. 14 – *Beggs Deformeter-Stress Analysis of Single-Barrel Conduits* [27].

- EM No. 14 Supplement – *Beggs Deformeter-Analysis of Additional Shapes* [28].

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[10] For significant and high hazard dams and/or dikes and their appurtenant structures which have been identified as "critical," selection of the seismic design load will be based on a quantitative risk analysis. For noncritical structures, a minimum earthquake loading will be used which approximates a design basis earthquake (DBE) or about a 500-year event.

- EM No. 27 – *Moments and Reactions for Rectangular Plates* [29].
- EM No. 34 – *Control of Cracking in Mass Concrete Structures* [30].
- *Concrete Manual*, eighth edition [31].
- Current ACI 318 and ACI 350 building codes.
- EM 1110-2-2104 – *Strength Design for Reinforced Concrete Hydraulic Structures, U.S. Army Corps of Engineers* [32].
- ACI SP-3 – *Reinforced Concrete Design Handbook* (Working Stress Method), third edition [33].
- *Design Criteria for Retaining Walls* [34].
- *Roller-Compacted Concrete: Design and Construction Considerations for Hydraulic Structures* [35].

1.7.1.5 Mechanical/Electrical Design

The mechanical/electrical design takes place concurrently with the structural analysis/design. The following chapters of this design standard explain the steps of mechanical/electrical design, including:

- Select, size, and design gates/valves, if applicable.
- Select, size, and design trashracks, stoplogs, and/or bulkhead gates, if applicable.
- Select, size, and design gate/valve operators and generators, if applicable.
- Select, size, and design heating, ventilation, and cooling (HVAC) systems, if applicable.
- Select, size, and design hoists and/or cranes, if applicable.
- Design lighting and electrical control systems for gate/valve operations, HVAC systems, and supervisory control and data acquisition (SCADA) systems.
- Address life safety considerations as note in the National Fire Protection Association (NFPA) 100, *Life Safety Code Handbook*, 2009 edition.

For more detailed guidance associated with mechanical/electrical design for a spillway, refer to Chapter 7, “Mechanical/Electrical Considerations.”

Selection of type and size of a spillway gate and operating system will be dependent on the evaluation of a number of factors including:

- Site conditions.
- Spillway configuration.
- Access.
- Available power.
- Hydraulic considerations.
- Seismic considerations.
- Construction/constructability considerations.
- Ice loading.
- Navigation needs.
- Sediment and debris loading.
- Operation and maintenance considerations.
- Economics.

These factors are further discussed in the following chapters of this design standard and some of the following sections. Because many of the spillways are associated with significant and high hazard dams/dikes, and failure of gates/valves may result in uncontrolled release of large flows, some redundant features/equipment might be required. Therefore, it may be advisable to design to stricter requirements than commonly called for by professional codes, standards, and/or guidelines.

Technical references associated with the mechanical/electrical design of spillways include:

- American Institute of Steel Construction (AISC) *Manual of Steel Construction*, thirteenth edition - refer to American National Standards Institute (ANSI)/AISC 360-05, Specifications for Structural Steel Buildings).

- American Welding Society (AWS) AWS D1.1/D1.1M Structural Welding Code – Steel; AWS D1.6 Structural Welding Code - Stainless Steel.

1.7.1.6 Risk Analysis (Only for Significant and High Hazard Dams/Dikes)

Probabilistic (in the form of a quantitative risk analysis), rather than deterministic, considerations will be part of any analysis/design for significant and high hazard dams and/or dikes, along with associated appurtenant structures (such as spillways) or critical components of associated appurtenant structures. The steps will be integrated with the previous design/analysis and include:

- Identify and define credible PFMs for the existing, modified, and/or new spillway. Although each spillway may have some unique PFMs, common PFMs include:

    - Flood-induced overtopping of dam and/or dike.
    - Flood-induced spillway operations which exceed the original/maximum design discharge, leading to overtopping of the chute wall and/or terminal structure walls, pressurizing the conduit and/or tunnel, or sweepout of the terminal structure, and leading to erosional headcutting of the spillway foundation or erosion of the dam and/or dam foundation.
    - Flood-induced spillway operations which result in cavitation damage of the chute and/or conduit, leading to erosion of the foundation.

    - Flood-induced operations which result in stagnation pressure (hydraulic jacking) and/or structural collapse of the chute and/or terminal structure, leading to erosion of the foundation.
    - Seismic-induced structural collapse of the spillway crest structure or features (such as piers, walls, and/or gates).

- Based on Reclamation's public protection guidelines [7], estimate the sum of the baseline (existing) annual probability of failure (APF) for all credible PFMs, and annualized loss of life (ALL) for all credible PFMs for a given loading condition, associated with existing and/or new dams, dikes, and all appurtenant structures such as spillways and outlet works.

- If an existing spillway is to be modified, estimate the sum of the modified APF for all credible PMFs and the sum of ALL for all credible PMFs associated with a given loading condition.

- For the modified or new spillway, if estimated sum of APF for all credible PFMs, and the sum of ALL for all credible PFMs associated with a given loading condition are "tolerably"[11] below Reclamation guidelines (1E-4 or a 1 in 10,000 chance during a given year for APF; and 1E-3 or a 1 in 1,000 chance during a given year for ALL), designs may be acceptable; however, if not tolerably below Reclamation guidelines, additional design considerations/features will be necessary to lower the estimated APF and ALL for the modified or new spillway.

- To address the uncertainties associated with using quantitative risk analysis to select an IDF, a robustness study is done to evaluate plausible operational and hydrologic/hydraulic scenarios that could increase the maximum RWS above the IDF-induced RWS. Typical scenarios that are evaluated include:

  - Misoperation.
  - Change in hydrology.
  - Debris blockage.
  - Change in downstream consequences.
  - Wind-generated waves.

For more detailed guidance associated with risk analysis/design for a spillway, refer to Chapter 2, "Hydrologic Considerations" and Chapter 3, "General Spillway Design Considerations".

Technical references associated with the risk analysis/design of spillways include:

- *Guidelines for Achieving Public Protection in Dam Safety Decisionmaking* [7].

- REC-ERC-88-3 – *Overtopping Flow on Low Embankment Dams – Summary Report of Model Test* [15].

- DSO-07-07 – *Uplift and Crack Flow Resulting from High Velocity Discharges Over Offset Joints* [16].

- DSO-99-06 – *A Procedure for Estimating Loss of Life Caused by Dam Failure* [36].

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[11] Tolerably below Reclamation guidelines will be unique to each condition/situation and will be mutually agreed to by the designer of record and Reclamation's Dam Safety Office, along with concurrence by Reclamation management. Consideration will include the level of uncertainty associated with estimates, and future conditions that could increase the estimates (such as changes in downstream consequences).

- Final Technical Report No. 99DG81029 – *Considerations for Estimating Structural Response Probabilities in Dam Safety Risk Analysis* [37].

- Appendix D – *Toolbox for Handling Loads by Upstream Dams and Incorporating Consequences for Failure of Downstream Dams* [38].

- DSO-98-004 – *Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment* [39].

- *Interim Guidelines for Addressing the Risk of Extreme Hydrologic Events* [40].

- *Risk Analysis Facilitator's Notebook* [42].

- *Dam Safety Risk Analysis Best Practices Training Manual* [43].

1.7.2 Outlet Works Design/Analysis

Tasks for analyzing/designing outlet works are summarized in the following sections.

1.7.2.1 Location, Type and Size

Similar to the spillway, selection of the location, type, and size of an outlet works will be dependent on the evaluation of a number of factors including:

- Site conditions (geology/topography).

- Dam and/or dike type.

- Hydrologic considerations.

- Hydraulic considerations.

- Seismic considerations.

- Construction/constructability considerations.

- Project objectives.

- Robustness of design.

- Risks associated with credible PFMs, which must be tolerably below Reclamation's public protection guidelines [7].

- Operation and maintenance considerations.

- Economics.

These factors are further discussed in the following chapters of this design standard and some of the following sections. Also, the two factors that were highlighted for the spillway are also highlighted for the outlet works, which are the robustness of design and the risks associated with PFMs. It is worth restating that because many of the outlet works are associated with significant and high hazard dams/dikes, it is important that any new or modified appurtenant structure designs protect the public to levels consistent with Reclamation's public protection guidelines [7]. This could mean some redundant features/equipment and designing to stricter requirements than commonly called for by professional codes, standards, and/or guidelines. Additionally, it should be restated that, as a starting point, an outlet works should include:

- Two gates or valves in series (one guard or emergency valve and one regulating gate/valve) that can be operated under unbalanced head conditions.

- Hydraulic control at or upstream of the projected centerline of the dam/dike with either a pressurized pipe inside a larger access conduit and/or tunnel or a free flow conduit and/or tunnel downstream of hydraulic control, particularly when associated with embankment dams/dikes.

- Stoplogs or bulkhead and slots provided near the intake structure.

- For embankment and/or rockfill dams/dikes, isolate the outlet works from the dam/dike (i.e., consider tunnel through the dam/dike abutments or reservoir rim, rather than a cut-and-cover outlet works through or beneath the dam/dike).

There will be cases where deviation from the above considerations will occur. However, there should be strong justification to not incorporate these considerations.

For more detailed guidance associated with locating, along with selecting, a type and size of an outlet works, refer to Chapter 4, "General Outlet Works and Diversion Design Considerations."

1.7.2.2 Hydraulic Analysis/Design

Similar to the spillway, the hydraulic analysis/design will be concurrent with the previous task of locating and selecting a type and size of outlet works. The following chapters of the design standard explain the following steps of hydraulic analysis/design:

- Develop/verify discharge curves.
- Evaluate intake structure and conveyance features (such as conduit) sizes, along with gate/valve types/sizes based on:
  - **Diversion.** If outlet works will be used for diversion during construction, initial flood routings of diversion floods (sometimes referred to as construction floods) should be done to size conveyance features and temporary cofferdams.
  - **Normal operations.** Evaluate conveyance features size and gate/valve type/size for passing operational flows.
  - **Emergency evacuation.** Evaluate ability of outlet works to lower the reservoir in a timely fashion pursuant to guidelines noted in *Criteria and Guidelines for Evacuating Storage Reservoirs and Sizing Low-Level Outlet Works* [45].
  - **Floods.** Although not typical, some outlet works are used in passing floods. If this is the case, flood routing steps similar to those noted for the spillway should be employed.
- Refine discharge curves based on results from previous steps and finalize intake structure and conveyance features sizes, along with gate/valve types/sizes.
- Depending on type/configuration of the outlet works, initial water surface profiles to layout conveyance features (such as chute/conduit) and terminal structure size and type.
- Final water surface profiles to finalize size and type of outlet works conveyance features and terminal structure.

For more detailed guidance associated with hydraulic analysis/design for an outlet works, refer to Chapter 2, “Hydrologic Considerations,” and Chapter 5, “Hydraulic Considerations.”

Technical references associated with the hydraulic analysis/design of outlet works include:

- *Conduits through Embankment Dams: Best Practices for Design, Construction, Problems Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [1].
- *Design of Small Dams*, third edition [8].
- EM No. 25 – *Hydraulic Design of Stilling Basins and Energy Dissipators* [10].
- REC-ERC-73-5 – *Hydraulic Model Studies of Chute Offsets, Air Slots, and Deflectors for High-Velocity Jets* [12].
- REC-ERC-78-8 – *Low Froude Number Stilling Basin Design* [13].
- DSO-07-07 – *Uplift and Crack Flow Resulting from High Velocity Discharges Over Offset Joints* [16].
- *Hydraulic and Excavation Tables*, 11th edition [18].
- *Computing Degradation and Local Scour* [19].
- *Guide for Computing Water Surface Profiles* [20].
- ACER TM No. 3 – *Criteria and Guidelines for Evacuating Storage Reservoirs and Sizing Low-Level Outlet Works* [44].
- EM No. 7 – *Friction Factors for Large Conduits Flowing Full* [45].
- REC-ERC-78-24 – *Hydraulic Design of Stilling Basin for Pipe or Channel Outlets* [46].
- EM No. 41 – *Air-Water Flow in Hydraulic Structures* [47].
- ACER TM No. 4 – *Criteria for Bulkheading Outlet Works Intakes for Storage Dams* [48].
- *Plastic Pipe Used In Embankment Dams: Best Practices for Design, Construction, Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [52].

- *Outlet Works Energy Dissipators: Best Practices for Design, Construction, Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [53].

1.7.2.3 Foundation Analysis/Design

The foundation analysis/design will start when locating and selecting a type and size of outlet works and parallels the structural analysis/design efforts. The following chapters of this design standard explain the steps of foundation analysis/design, including:

- Define site-specific foundation material and strength of properties.

- Assist in identifying suitable sites based on site-specific foundation needs for an outlet works.

- Define and analyze ground water conditions.

- Develop appropriate foundation designs which include:

  - Drainage features, which in many cases are the key foundation design feature. (NOTE: Identify and evaluate drainage features/systems that will be accessible for inspection using CCTV equipment).
  - Adequate support for the outlet, which could range from excavating to competent rock to driving piles, placing piers in a soil foundation, or supporting excavated tunnels with rock bolts, shotcrete, etc.

- Accommodating loading conditions loads on the outlet works that could include static (hydrostatic, rock/soil, ice, and frost heave), hydrologic (floods), and seismic (hydrodynamic, dynamic soil/rock) loads, etc.

For more detailed guidance associated with foundation analysis/design for an outlet works, refer to Chapter 6, “Foundation Considerations.”

Technical references associated with the foundation analysis/design of outlet works include:

- *Conduits through Embankment Dams: Best Practices for Design, Construction, Problems Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [1].

- *Design of Small Dams*, third edition [8].

- EM No. 13 – *Estimating Foundation Settlement by One-Dimensional Consolidation Tests* [21].

- REC-ERC-74-10 – *Rock Mechanics Properties of Typical Foundation Rock Types* [22].

- REC-ERC-82-17 – *Frost Action in Soil Foundations and Control of Surface Structure Heaving* [23].

- ACER TM No. 9 – *Guidelines for Controlling Seepage Along Conduits Through Embankments* [24].

- *Drainage for Dams and Associated Structures* [25].

1.7.2.4 Structural Analysis/Design

Similar to the spillway, the structural analysis/design of the outlet works typically follows the hydraulic analysis/design. The following chapters of this design standard explain the steps of structural analysis/design, including:

- Identify which features are considered "critical"[7] and "noncritical"[8] [26].

- Identify and define loading conditions for both critical and noncritical features, which will typically fall into four categories, including:

  - Construction.
  - Operational (normal or static).
  - Flood (hydrologic).
  - Earthquake (seismic).

- Identify, evaluate, and select material types and associated properties. Materials could include concrete (reinforced, conventional mass, roller compacted, and precast), steel, and plastic such as HDPE.

- Apply appropriate structural analysis/design methods which are based on the crawl, walk, and run philosophy (i.e., using the simplest approach that is technically adequate to prepare the analysis/design). These include:

  - **For analyses.** Pseudo-static, linear elastic FEM using response spectra and nonlinear elastic FEM using time histories.

  - **For designs.** Design methods are typically based on Reclamation technical references such as *Design of Small Dams* [8], and industry codes are used such as the ACI manuals.

For more detailed guidance associated with structural analysis/design for an outlet works, refer to Chapter 7, "Structural Considerations."

Technical references associated with the structural analysis/design of outlet works include:

- *Conduits through Embankment Dams: Best Practices for Design, Construction, Problems Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [1].
- *Design of Small Dams*, third edition [8].
- EM No. 14 – *Beggs Deformeter-Stress Analysis of Single-Barrel Conduits* [27].
- EM No. 14 Supplement – *Beggs Deformeter-Analysis of Additional Shapes* [28].
- EM No. 27 – *Moments and Reactions for Rectangular Plates* [29].
- EM No. 34 – *Control of Cracking in Mass Concrete Structures* [30].
- *Concrete Manual*, eighth edition [31].
- Current ACI 318 and ACI 350 building codes.
- EM 1110-2-2104 – *Strength Design for Reinforced-Concrete Design of Hydraulic Structures* [32].
- ACI SP-3 – *Reinforced Concrete Design Handbook (Working Stress Method),* third edition [33].
- *Design Criteria for Retaining Walls* [34].
- *Roller-Compacted Concrete: Design and Construction Considerations for Hydraulic Structures* [35].

1.7.2.5 Mechanical/Electrical Design.

The mechanical/electrical design takes place concurrently with the structural analysis/design. The following chapters of this design standard explain the steps of mechanical/electrical design, including:

- Select, size, and design gates/valves.
- Select, size, and design bulkhead gates, if applicable.
- Select, size, and design trashracks.

- Select, size, and design steel pipe.
- Select, size, and design gate/valve operators and generators.
- Select, size and design HVAC systems.
- Select, size, and design hoists and/or cranes, if applicable.
- Design lighting and electrical control systems for gate/valve operations, HVAC systems, and SCADA systems.
- Address life safety considerations as noted in NFPA 100, *Life Safety Code Handbook*, 2009 edition.

For more detailed guidance associated with mechanical/electrical design for an outlet works, refer to Chapter 7, "Mechanical/Electrical Considerations."

Selection of type and size of an outlet gate or valve and operating system will be dependent on the evaluation of a number of factors including:

- Site conditions.
- Outlet works configuration.
- Access.
- Available power.
- Hydraulic considerations.
- Seismic considerations.
- Construction/constructability considerations.
- Ice loading.
- Sedimentation and debris loading.
- Operation and maintenance considerations.
- Economics.

These factors are further discussed in the following chapters of this design standard and in some of the following sections. Because reliable operation of outlet works may be critical in maintaining the safety of dams, some redundant

features/equipment might be required, and it may be advisable to design to stricter requirements than commonly called for by professional codes, standards, and/or guidelines. Of special note, gate and/or valve controls should be located outside of flooded area, should a failure occur, to ensure that emergency/guard gates and/or valves can be operated.

Technical references associated with the mechanical/electrical design of outlet works include:

- AISC *Manual of Steel Construction*, thirteenth edition (refer to ANSC/AISC 360-05, Specifications for Structural Steel Buildings).

- AWS D1.1/D1.1M Structural Welding Code – Steel; AWS D1.6 Structural Welding Code - Stainless Steel.

- *Hydraulic Downpull Forces on Large Gates*, Research Report No. 4 [49].

1.7.2.6 Risk Analysis (Only for Significant and High Hazard Dams/Dikes)

Similar to the spillway, probabilistic (in the form of a quantitative risk analysis), rather than deterministic, considerations will be part of any analysis/design for significant and high hazard dams and/or dikes, along with appurtenant structures such as outlet works. The steps will be integrated with the previous design/analysis and include:

- Identify and define credible PFMs for the existing, modified, and/or new outlet works. Although each outlet works may have some unique credible PFMs, common PFMs have included:

    - Flood-induced overtopping of dam and/or dike (if outlet works are used to help pass floods).

    - Flood-induced outlet works operations which exceed the original/maximum design discharge, leading to overtopping of the chute wall and/or terminal structure walls, or sweepout of the terminal structure, and leading to erosional headcutting of the outlet works foundation or erosion of the dam and/or dam foundation, overstressing the conduits and/or tunnels, introducing pressurized seepage through cracks/joints in the conduit and/or tunnels into surrounding embankment or foundation materials.

    - Operational- and/or flood-induced cavitation damage typically downstream of gates/valves in the chute and/or conduit, leading to erosion of the foundation.

  - Stagnation pressure (hydraulic jacking) and/or structural collapse of the chute and/or terminal structure, leading to erosion of the foundation.

- Based on Reclamation's public protection guidelines [7], estimate the sum of the baseline (existing) APF for all credible PFMs and ALL for all credible PFM for a given loading condition, associated with existing and/or new dams, dikes, and appurtenant structures such as spillways and outlet works.

- If an existing spillway is to be modified, estimate the sum of the modified APF for all credible PMFs and the sum of ALL for all credible PMF associated with a given loading condition.

- For the modified or new spillway, if estimated sum of APF for all credible PFMs, and the sum of ALL for all credible PFMs associated with a given loading condition are "tolerably" below Reclamation guidelines (1E-4 or a 1 in 10,000 chance during a given year for APF; and 1E-3 or a 1 in 1,000 chance during a given year for ALL), designs may be acceptable; however, if not tolerably below Reclamation guidelines, additional design considerations/features will be necessary to lower the estimated APF and ALL for the modified or new outlet works.

For more detailed guidance associated with risk analysis/design for an outlet works, refer to Chapter 2, "Hydrologic Considerations" and Chapter 4, "General Outlet Works and Diversion Design Considerations."

Technical references associated with the risk analysis/design of spillways include:

- *Guidelines for Achieving Public Protection in Dam Safety Decisionmaking* [7].

- REC-ERC-88-3 – *Overtopping Flow on Low Embankment Dams – Summary Report of Model Test* [15].

- DSO-07-07 – *Uplift and Crack Flow Resulting from High Velocity Discharges Over Offset Joints* [16].

- DSO-99-06 – *A Procedure for Estimating Loss of Life Caused by Dam Failure* [36].

- Final Technical Report No. 99DG81029 – *Considerations for Estimating Structural Response Probabilities in Dam Safety Risk Analysis* [37].

- Appendix D – *Toolbox for Handling Loads by Upstream Dams and Incorporating Consequences for Failure of Downstream Dams* [38].

- DSO-98-004 – *Prediction of Embankment Dam Breach Parameters: A Literature Review and Needs Assessment* [39].

- *Interim Guidelines for Addressing the Risk of Extreme Hydrologic Events* [40].

- *Risk Analysis Facilitator's Notebook* [42].

- *Dam Safety Risk Analysis Best Practices Training Manual* [43].

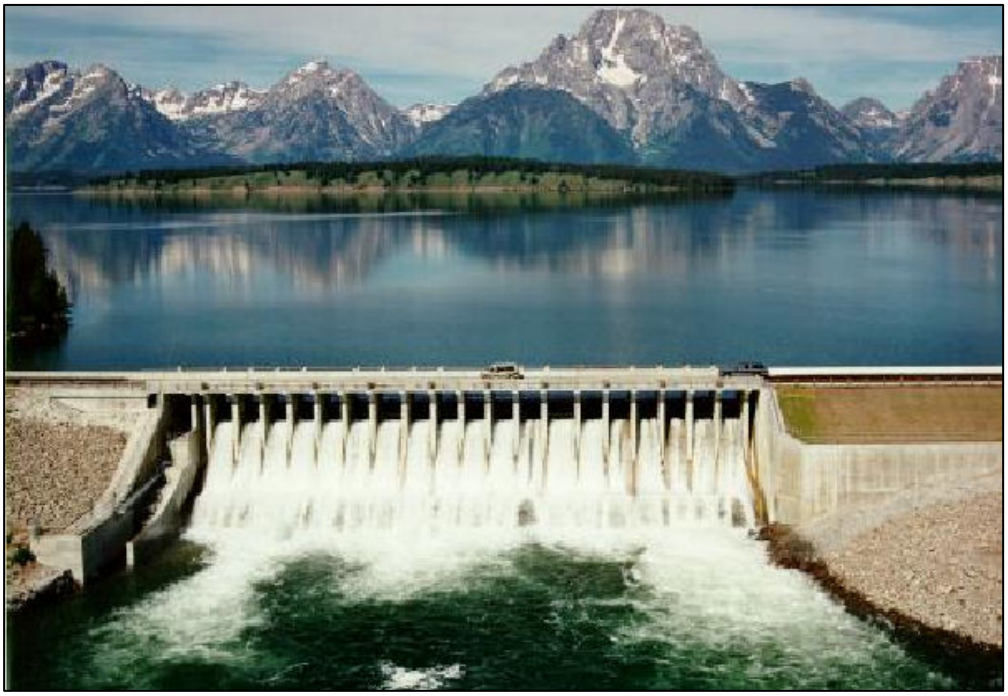

Figure 1.5.1.1-1. Example: Service spillway (gated), Jackson Lake Dam, Wyoming.

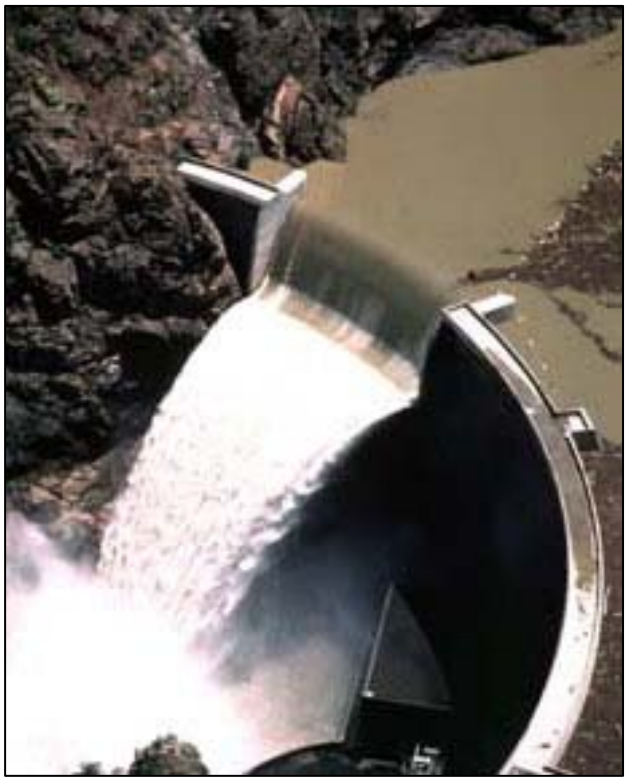

Figure 1.5.1.1-2. Example: Service spillway (ogee crest), Crystal Dam, Colorado.

Figure 1.5.1.2-3. Example: Auxiliary spillway (gated) in foreground and service spillway (gated) in background, Stewart Mountain Dam, Arizona.

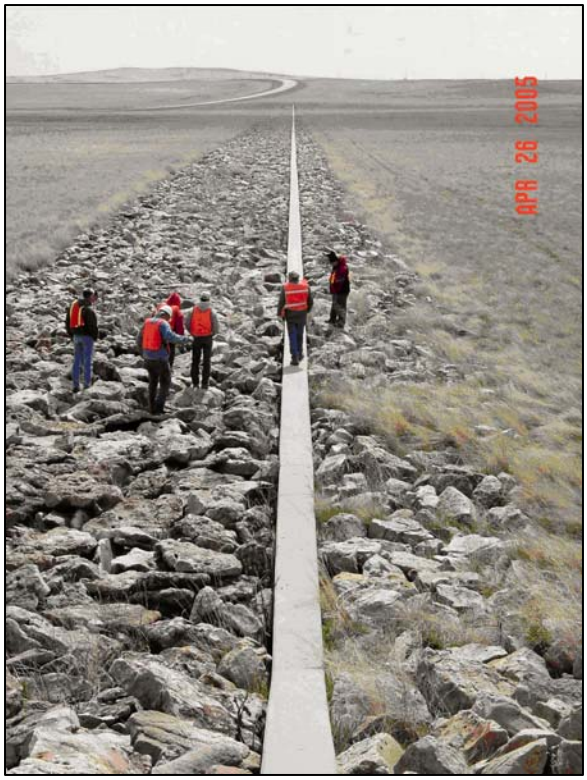

Figure 1.5.1.2-4. Example: Auxiliary spillway (grade control sill), Heart Butte Dam, North Dakota.

Figures 1.5.1.3-5. Example: Emergency spillway (fuseplug) in foreground and auxiliary spillway (ogee crest) in background, New Waddell Dam, Arizona.

Figure 1.5.1.3-6. Example: Emergency spillway (grade control sill), San Justo Dam, California.

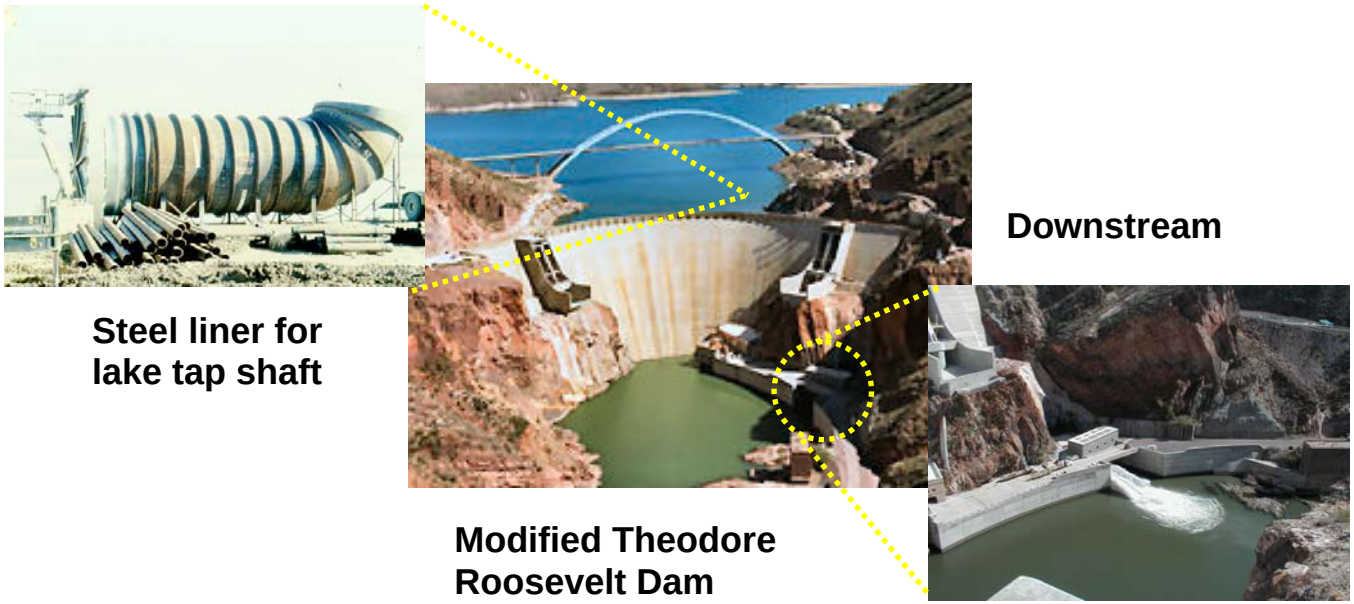

Figure 1.5.2-7. Example: Combination Tunnel outlet work and power penstock, highlighting a number of features such as the upstream intake structure and the downstream structures. Theodore Roosevelt Dam, Arizona.

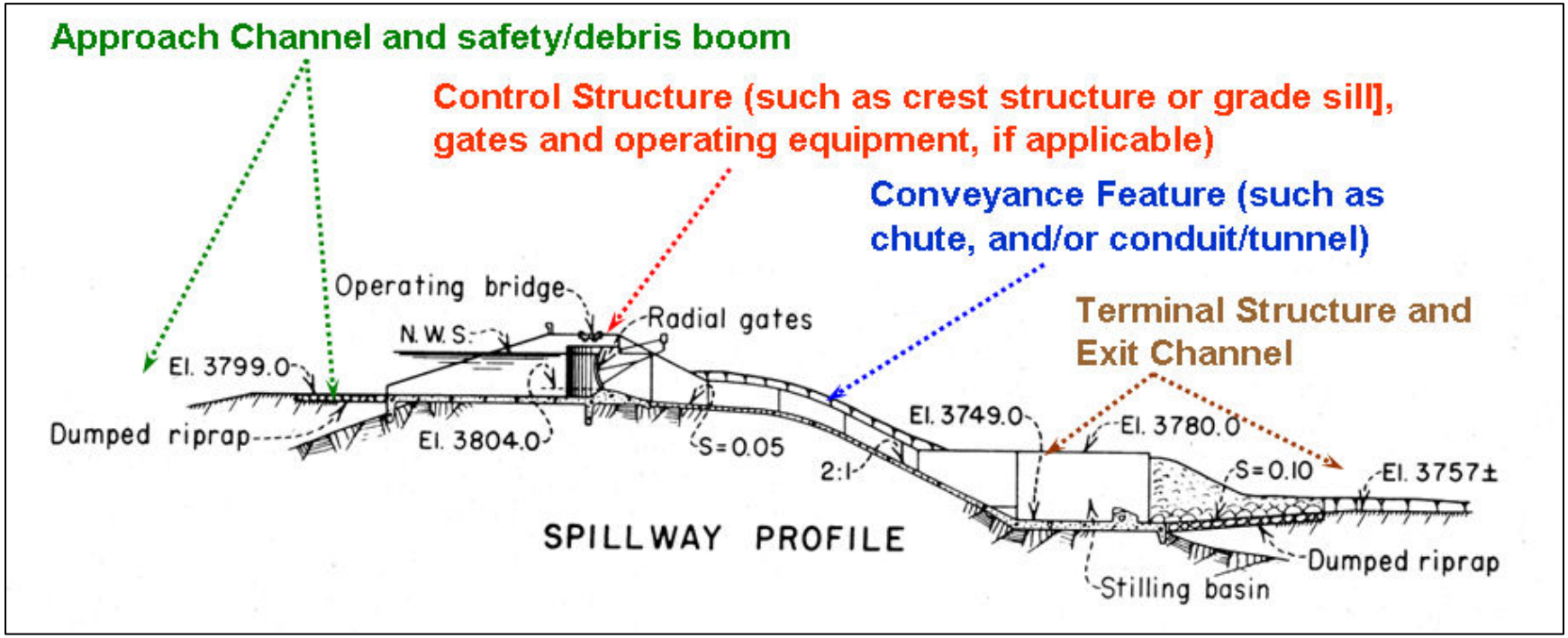

Figure 1.5.2-8. Common features of spillways.

Figure 1.6.1-9. Ogee crest (uncontrolled) spillway, Bumping Lake Dam, Washington.

Figure 1.6.1-10. Double side-channel (bathtub) crest (uncontrolled) spillway, Fontenelle Dam, Wyoming.

Figure 1.6.1-11. Side-channel crest (uncontrolled) spillway, Big Sandy Dam, Wyoming.

Figure 1.6.1-12. Morning glory (glory hole) crest (uncontrolled) spillway, Whiskeytown Dam, California.

Figure 1.6.2-13. Labyrinth crest (uncontrolled) spillway, Ute Dam, New Mexico.

Figure 1.6.1-14. Radial gated (controlled) spillway, Bradbury Dam, California.

Figure 1.6.1-15. Crest (Obermeyer type) gated (controlled) spillway, Friant Dam, California.

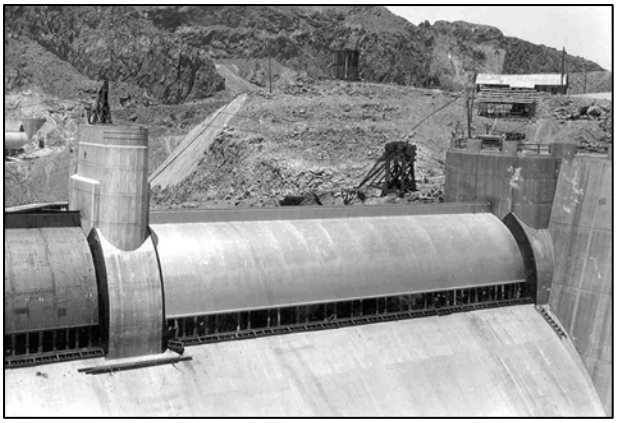

Figure 1.6.1-16. Drum gated (controlled) spillway, Hoover Dam, Arizona-Nevada.

Figure 1.6.1-17. Fuseplug crest (controlled) spillway (reservoir rim), Horseshoe Dam, Arizona.

Figures 1.6.1-18. Fusegate crest (controlled) spillway, Terminus Dam, California (courtesy of the U.S. Army Corps of Engineers, Sacramento District, Rick Poeppelman).

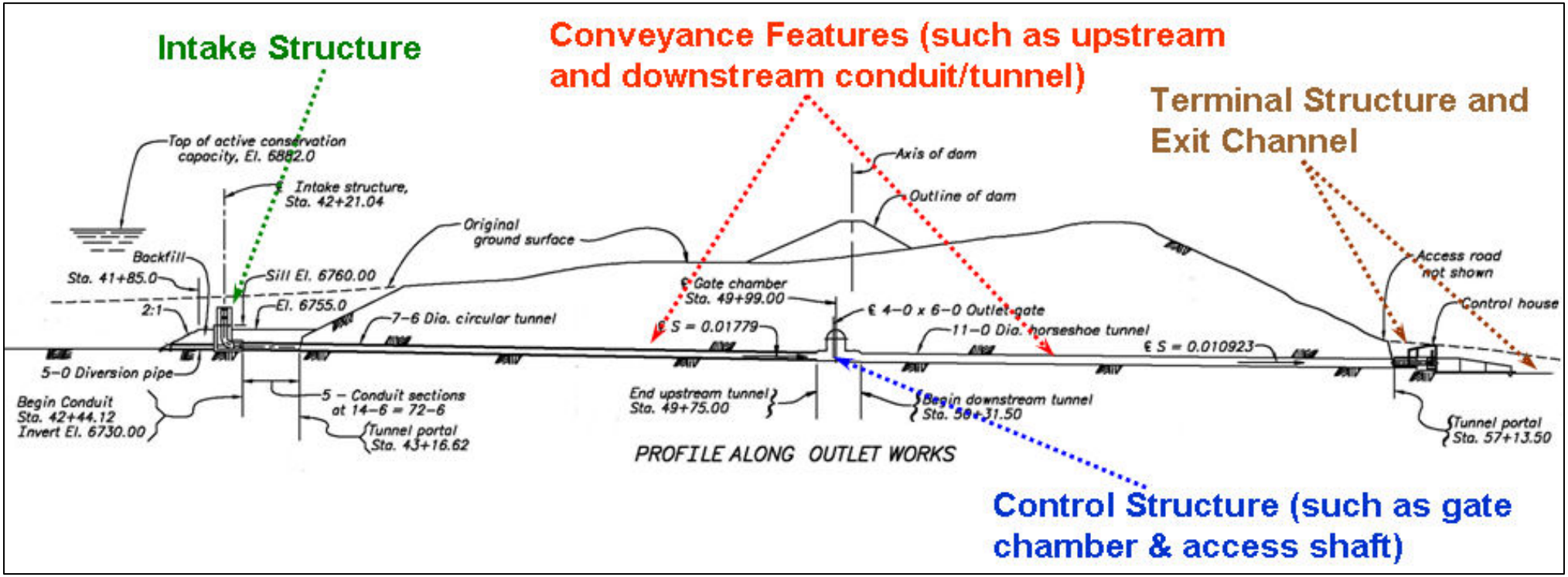

Figure 1.6.2-19. Common features of outlet works.

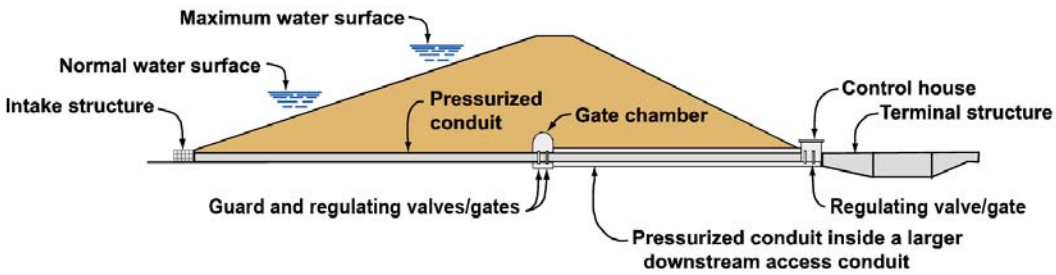

Figure 1.6.2-20. Preferred outlet works configuration for low-head embankment dams: hydraulic control at upstream intake with free flow conditions downstream of the regulating gate/valve [1].

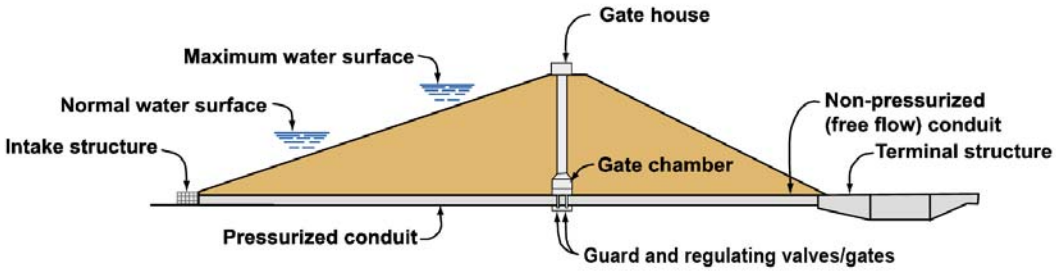

Figure 1.6.2-21. Preferred outlet works configuration for high-head embankment dams: hydraulic control at downstream control structure, with guard/emergency gate/valve at/near centerline of dam/dike, and downstream pressurized pipe (between dam/dike centerline and control structure inside larger access conduit) [1].

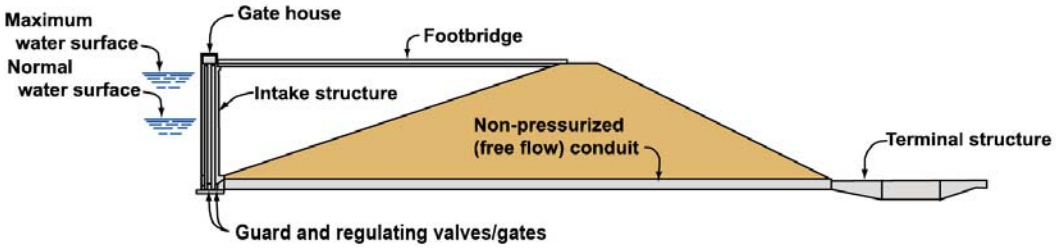

Figure 1.6.2-22. Acceptable outlet works configuration for embankment dams: hydraulic control at/near centerline of dam/dike, with free flow conditions downstream of the regulating gate/valve [1].

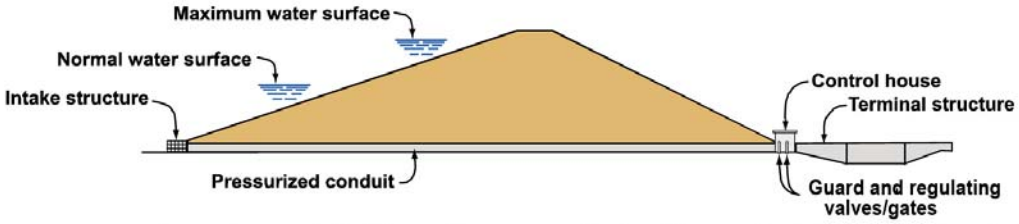

Figure 1.6.2-23. Least acceptable outlet works configuration for embankment dams: hydraulic control at downstream control structure (i.e., pressurized flow conditions upstream of the regulating gate/valve along most of the outlet works) [1].

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TSC Security Review of Proposed Public Disclosure of Technical Information

Type: Design Standard

Design Standard: Design Standards No. 14 – Appurtenant Structures for Dams (Spillway and Outlet Works) Design Standards

Chapters: 1. Introduction

Brief Description of Information: An overview of how the Bureau of Reclamation analyzes/designs spillways and outlet works appurtenant for dams and/or dikes. An outline of the following chapters for this design standard that provides details for spillway and outlet works analyses/designs. A list of key technical references used for each major task involved with spillway and outlet works analyses/designs.

Requesting/Sponsoring Organization: Director, Technical Service Center

Program Office: Deputy Commissioner, Operations

Is Information Official Use Only / SENSITIVE? (Y/N)
(to be completed by reviewer)

Is Information Official Use Only / RESTRICTED? (Y/N)
(to be completed by reviewer)

(Sensitive or Restricted information shall not be included in a design standard.)

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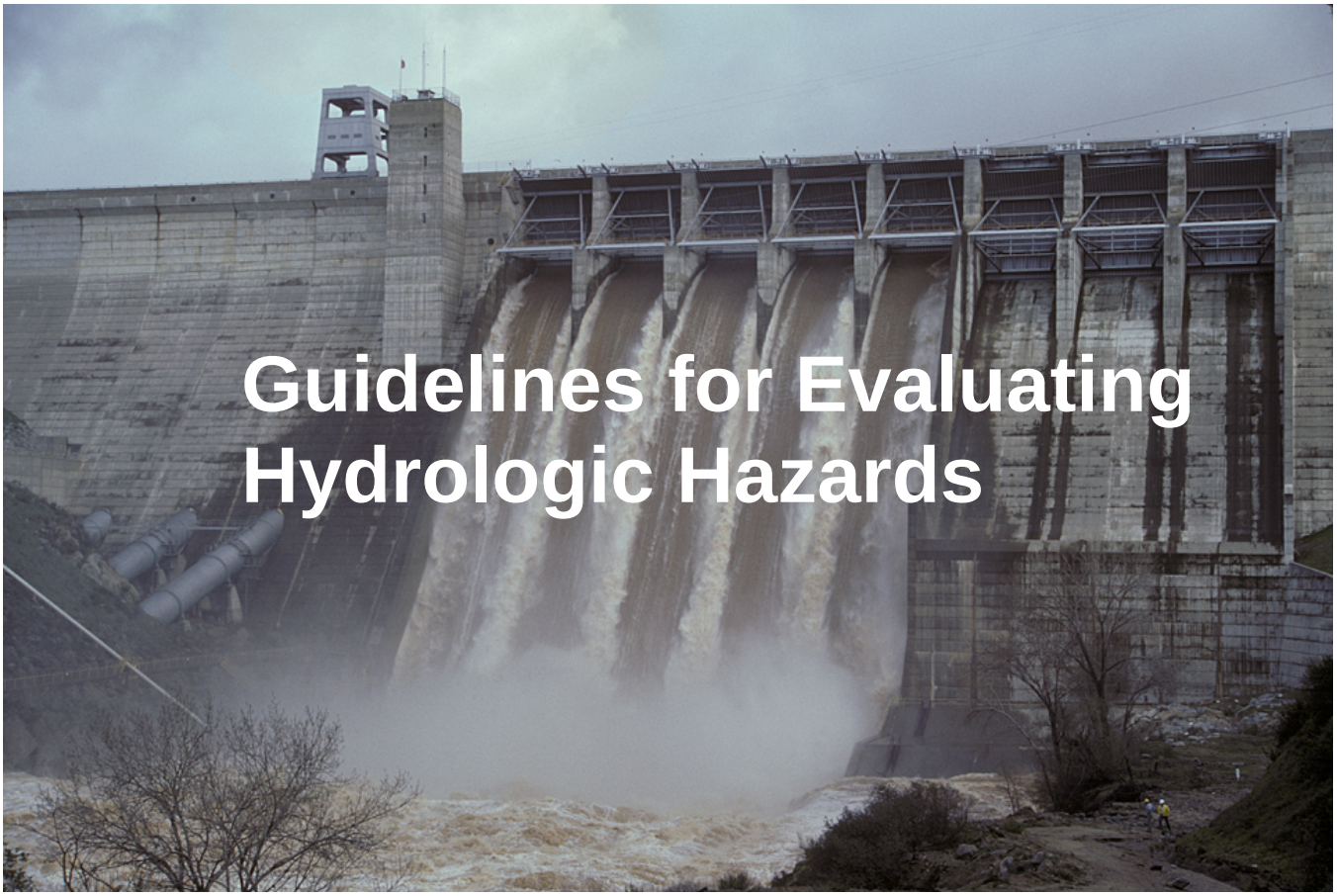
RECLAMATION

Managing Water in the West



Guidelines for Evaluating Hydrologic Hazards

U.S. Department of the Interior
Bureau of Reclamation

June 2006

MISSION STATEMENTS

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Guidelines for Evaluating Hydrologic Hazards

By
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John F. England, Jr.
Kenneth L. Bullard
David A. Raff

U.S. Department of the Interior
Bureau of Reclamation

June 2006

Contents

Page

Executive Summary **S-1**
1. Introduction **1**
2. Background **1**
2.1 General 1
2.2 Public Protection Guidelines 2
Dam Safety Program Processes 2
3. Process **3**
3.1 Data Sources 4
3.2 Flood Frequency Extrapolation 4
3.3 Flood Peak and Volume Relationships 7
4. Analysis Techniques **8**
4.1 Flood Frequency Analysis with Historical/Paleoflood Data 9
4.1.1 Historical and Paleoflood Data 9
4.1.2 Mixed-Population Graphical Approach 11
4.1.3 Expected Moments Algorithm 12
4.1.4 FLDFRQ3 13
4.2 Hydrograph Scaling and Volumes 15
4.3 GRADEX Method 17
4.4 Australian Rainfall-Runoff Method 25
4.4.1 Approach 26
4.4.2 Calibration 28
4.4.3 Strengths and Limitations 29
4.5 Stochastic Event-Based Precipitation Runoff Modeling with the SEFM 29
4.6 Stochastic Rainfall-Runoff Modeling With CASC2D 35
4.7 PMF Analysis Technique 36
5. Characterization of Hydrologic Hazards **45**
5.1 Integration of the PMF into Hydrologic Hazard Evaluations 46
5.2 Characterization of Hydrologic Risk for the CFR 46
5.3 Detailed Hydrologic Studies 50
6. Case Studies **52**
6.1 Los Banos Dam 52
6.1.1 Los Banos Hydrologic Hazard Curves Using Flood Frequency Analysis and Hydrograph Scaling 53
6.2 A.R. Bowman Dam 54
6.2.1 A.R. Bowman Hydrologic Hazard Curves Using Flood Frequency Analysis and Hydrograph Scaling 56
6.2.2 A.R. Bowman Hydrologic Hazard Estimates Based on a Stochastic Event Flood Model 57
6.2.3 A.R. Bowman Hydrologic Hazard Estimates Using Bayesian Statistical Estimation 60
6.2.4 Combined Hydrologic Hazard Estimates for Risk Analysis and Dam Safety Implications 62
6.3 Fresno Dam 64
6.3.1 Fresno Dam Hydrologic Hazard Curves Using Flood Frequency Analysis and Hydrograph Scaling 64
6.3.2 Fresno Dam Hydrologic Hazard Analysis Using the GRADEX Method 66
7. Summary **72**
8. Bibliography **74**

Executive Summary

The purpose of this document is to establish guidelines for generating hydrologic hazard information for use in evaluating hydrologic risk at dams. This information is intended to be used for risk analysis and prioritization of further work at Bureau of Reclamation (Reclamation) dams and other U.S. Department of the Interior facilities. Hydrologic hazard information consists of a flood frequency analysis and frequency flood hydrographs for a full range of Annual Exceedance Probabilities (AEP) necessary for decision making.

Reclamation has developed an approach toward developing hydrologic hazard curves for use in evaluating dam safety issues. The procedure relies on extracting information from existing studies to the fullest extent possible. The procedures and analysis techniques defined in these guidelines allow for the possibility, and even plausibility, that peak discharge and volume estimates may exceed the probable maximum flood (PMF). This is a function of the uncertainty and inconsistency among and between analysis techniques. Therefore, in these cases, the PMF is believed to represent the upper limit to hydrologic risk.

The procedure for developing hydrologic hazard curves considers the dam safety decision criteria, potential dam failure mode and dam characteristics, available hydrologic data, possible analysis techniques, resources available for analysis, and tolerable level of uncertainty. Dam safety decision criteria determine the probabilistic range of floods needed to address hydrologic issues. The potential dam failure mode and dam characteristics impact the type of hydrologic information needed to assess the problem. The specific elements selected to be incorporated in an analysis of hydrologic hazards should consider the tolerable level of uncertainty. To reduce the uncertainty in the estimates, additional data collection and use of more sophisticated solution techniques may be required.

Reclamation currently uses a combination of seven hydrologic methods to develop hydrologic hazard curves. These general techniques include:

- Flood frequency analysis with historical/paleoflood data
- Hydrograph scaling and volumes
- The GRADEX Method
- The Australian Rainfall-Runoff Method
- Stochastic event-based precipitation runoff modeling with stochastic event flood model
- Stochastic rainfall-runoff modeling with CASC2D
- The PMF

It is believed that increasing the level of effort and sophistication of analysis technique increases the level of confidence associated with the results.

The amount of effort expended on analyzing a hydrologic hazard depends on the nature of the problem and the potential cost of the solution. A staged approach toward evaluating a hydrologic safety issue is recommended. Initially, very little effort is expended to determine the

magnitude of the hydrologic hazard. Reclamation attempts to make use of all the available studies for the site of interest. Often, the PMF and initial flood frequency studies are the only hydrologic studies available before the start of a probabilistic investigation. When other hydrologic studies have been performed, available data will be used to decrease uncertainty in results as well as provide an overall assessment of hydrologic risk.

Dam safety evaluations usually begin by characterizing hydrologic risk for the Comprehensive Facility Review (CFR) process. If detailed studies have been conducted for the site of interest, they are summarized, consolidated, and presented to the risk assessment team. About two-thirds of Reclamation's dams can safely accommodate the PMF; when the PMF is selected as the inflow design flood, no additional work may be required unless other hydraulic issues need evaluation. Additional hydrologic work begins with a flood frequency analysis developed for peak flows and volumes and hydrograph scaling. It is believed that this type of information is sufficient to address hydrologic issues and make dam safety decisions at about 80 percent of the remaining dams. For the sites that still have potential safety problems, more sophisticated solution techniques than the initial flood frequency analysis and hydrograph scaling may be required.

When planning more detailed studies, the goal is to achieve a balance between the amount of hydrologic analysis needed to address the issues and the level of effort required to conduct the study. As the studies get more detailed, the results should become more precise and contain less uncertainty.

When multiple methods are used, alternative hazard curves are developed by weighting results from the individual analyses. A team of hydrologists evaluates the alternatives and selects the one most representative for the site for use in the risk assessment. Selection of the final hydrologic hazard curve depends on the experience of the hydrologists and the assumptions that went into each analysis.

Three case studies, Los Banos, Fresno, and A.R. Bowman Dams, are presented in these guidelines to illustrate the variety of methods available. These sites were chosen to demonstrate the use of flood frequency analysis and hydrograph scaling to characterize the flood hazard and more detailed followup studies, where available. The A.R. Bowman example shows how multiple studies were combined into a single flood hazard curve for use in risk assessment.

1. Introduction

The purpose of these guidelines is to develop hydrologic hazard curves and flood hydrographs for use in evaluating and prioritizing the need for dam safety modifications at Reclamation and other U.S. Department of the Interior (Interior) facilities. Hydrologic hazard curves are defined as graphs of peak flow and volume (for specified durations) versus Annual Exceedance Probability (AEP). The range of AEPs that is displayed on these graphs is to be sufficient to support the decision making needs of the organization. The intent of these guidelines is to provide a procedure for developing hydrologic hazard information that will allow decision makers to determine appropriate courses of action to assure the safety of the dam while minimizing study costs. Hydrologic hazard information is intended to support Reclamation's risk-based Dam Safety Program.

These guidelines are based on the Dam Safety Research Report DSO-04-08, *Hydrologic Hazard Curve Estimating Procedures* (Swain et al., 2004). The research project used information developed at the Logan, Utah Workshop held in 1999 to provide a framework for Reclamation to assess flood hazards. The workshop produced the report, *A Framework for Characterizing Extreme Floods for Dam Safety Risk Assessment* (Bureau of Reclamation, 1999). Hydrologic research has led to advances in flood estimation procedures that allow improvements to the framework. These guidelines describe current approaches used by Reclamation to determine flood loadings for its dams. New techniques for developing hydrologic hazard information can be added to these guidelines as they are developed by the hydrology community.

2. Background

2.1 General

Reclamation's Dam Safety Program mission is "To ensure that Reclamation dams do not present unacceptable risks to people, property, and the environment" (Bureau of Reclamation, 1993). As the owner of over 350 high- or significant-hazard storage dams in the western U.S., Reclamation is committed to providing the public and the environment with adequate protection from the risks that are inherent in collecting and storing large volumes of water. Traditional design and analysis methods have focused on selecting a level of protection based on spillway evaluation flood loadings, which were usually based on the probable maximum flood (PMF) (Bureau of Reclamation, 1999).

Since 1995, Reclamation has used a risk assessment process to determine an appropriate level of public protection by evaluating a full range of loading conditions and possible dam failure consequences. This is in contrast to the traditional approach of using upper bound events without regard to their likelihood of occurrence and without assessment of their incremental consequences. As a water resources management agency, Reclamation strives to provide decisionmakers with risk-based information founded upon current or emerging water resources management and public safety practices (Bureau of Reclamation, 1999).

Risk assessment methods provide techniques to organize and plan the data collection and technical studies necessary to evaluate dam safety issues at a site. The risk assessment process allows the risk assessment team to consider the possible adverse outcomes to a given loading condition and compute the risk associated with each possible outcome. The process involves identifying all of the possible loading conditions, dam responses, exposure conditions, and consequences. The overall risk from the dam is the accumulation of the risks associated with each of these factors (Bureau of Reclamation, 1999).

When evaluating hydrologic hazards, a systematic means of developing flood hazard relationships is needed for risk-based assessments to determine hydrologic adequacy for Reclamation dams. The nature of the potential failure mode and characteristics of the dam and reservoir dictate the type of hydrologic information needed. For some sites, only a peak-discharge frequency analysis may be required, while at other sites, flood volumes and hydrographs may be required. The goal of any hydrologic analysis is to provide the hydrologic information needed to make dam safety decisions at the least possible cost.

2.2 Public Protection Guidelines

Guidance for providing adequate and consistent levels of public protection in the evaluation and modification of existing dams and the design of new structures are described in the *Guidelines for Achieving Public Protection in Dam Safety Decisionmaking*, (Bureau of Reclamation, 2003a). The reader may refer to the guidelines for a complete description of the assessment measures used by Reclamation in making dam safety decisions.

Determining an appropriate level of public protection involves assessing the existing risks, determining the need for risk reduction, and, where needed, evaluating specific alternatives to reduce risk. Because the total needs for the agency's financial and human resources generally exceed the available resources, the Public Protection Guidelines were prepared to assist Reclamation staff in presenting public safety information to decisionmakers for prioritizing among projects and allocating limited resources.

Reclamation's Public Protection Guidelines consist of two assessment measures of risk that are considered in the decision process for a dam: (1) the probability of dam failure and (2) the life loss consequences resulting from unintentional reservoir release. The annual probability of failure guideline considers the accumulation of risks from Reclamation's total inventory of dams. The life loss guideline deals with agency public trust responsibilities.

Dam Safety Program Processes

Hydrologic hazard information is generally required during four stages of the dam safety program process. These four stages include the Comprehensive Facility Review (CFR), Issue Evaluation (IE), Corrective Action Study (CAS), and Final Design (FD). Most projects do not progress through each stage of the process because the process is intended to address dam safety deficiencies, and many projects either have no deficiencies or the safety issues can be resolved without a need for structural modifications. The remainder of this section of the guidelines will

briefly describe the four stages of the dam safety program process that require hydrologic hazard information. For more detailed information about the dam safety process, the reader should review the references cited.

The CFR provides a mechanism for early detection of developing and/or existing dam safety issues. The CFR is performed every 6 years and consists of a state-of-the-art review of the dam and its performance, previous studies/analyses (including hydrology), construction practices, downstream consequences, risk, and dam safety decisions (Bureau of Reclamation, 1998). The CFR is used to identify risks at individual dams and to prioritize further work. Once hydrologic hazard information is developed for the CFR, the Dam Safety Office determines whether or not additional hydrologic studies are required to make decisions during subsequent stages of the process.

The IE stage is used to confirm problems identified previously. Data collection and/or analysis activities are focused on addressing specific dam safety issues and updating risk estimates. At the conclusion of the IE, the decision makers determine whether or not actions are required to reduce risk at the dam (Bureau of Reclamation, 2003b).

A CAS formulates and evaluates risk reduction alternatives. Data is collected and analyzed to the extent necessary to develop the details of identified alternatives, to estimate project costs, and to provide sufficient information to allow decision makers to select and justify the proper course of action. The baseline risk analysis is updated to show the risk reduction potential of each of the developed alternatives (Bureau of Reclamation, 2003b).

During the FD stage, the conceptual design is transformed into the final design. Additional data collection and analysis are used to improve the design, reduce and refine project costs, and finalize design drawings and specifications (Bureau of Reclamation, 2003b).

3. Process

The elements selected for incorporation in an analysis of hydrologic hazards must consider the potential dam failure mode and dam characteristics, available hydrologic data, possible analysis techniques, resources available for analysis, and tolerable level of uncertainty. The potential dam failure mode and dam characteristics impact the type of hydrologic information needed to assess the problem. Some problems may require only a peak-discharge frequency curve, while others may need complete hydrographs. The available data, possible analysis techniques, resources available, and needs of the decision makers influence the selection of elements to be included in developing hydrologic hazard curves.

The process that follows provides a systematic approach for estimating hydrologic hazard curves that can be used for dam safety decisionmaking. It recognizes that additional studies do not always lead to better decisions. Therefore, the process relies on using existing data and previous analyses as much as possible to produce hydrologic information suitable for dam safety decisionmaking at the least possible cost.

3.1 Data Sources

Developing hydrologic hazard curves for risk assessment uses the length of record and type of data to determine the extrapolation limits for flood frequency analysis. Extrapolation beyond the data is often necessary to provide information needed for dam safety risk assessments. The sources of information used for flood hazard analyses include streamflow and precipitation records and paleoflood data.

Streamflow records consist of data collected at established gaging stations and indirect measurements of streamflow at other sites. Streamflow data can include estimates of peak discharge as well as average or mean discharge for various time periods. Most streamflow measurements on U.S. streams began after 1900, with only a few records dating back that far. Most often, streamflow records at a single site range in length from about 20 to 60 years. In some cases these records can be extended to about 150 years using historical information, which include human observations and recordings prior to the development of systematic streamflow measurement.

Precipitation and weather data used in hydrologic models can include rainfall, snowfall, snow water equivalent, temperature, solar radiation, and wind speed and direction. These data are available from various sources and vary greatly in record length and quality throughout the United States. Some of these types of data (i.e., snowfall, snow water equivalent, solar radiation, and wind) are limited to record lengths of less than about 30 years; rainfall and temperature data are available for some stations for up to 150 years, but in most cases are limited to less than 100 years.

Paleoflood hydrology is the study of past or ancient flood events which occurred before the time of human observation or direct measurement by modern hydrological procedures (Baker, 1987). Unlike historical data, paleoflood data do not involve direct human observation of the flood events. Instead, the paleoflood investigator studies geomorphic and stratigraphic records (various indicators) of past floods, as well as the evidence of past floods and streamflow derived from historical, archeological, dendrochronologic, or other sources. The advantage of paleoflood data is that it is often possible to develop records that are 10 to 100 times longer than conventional or historical records from other data sources in the western United States. Paleoflood data generally include records of the largest floods, or commonly, the limits on the stages of the largest floods over long time periods.

3.2 Flood Frequency Extrapolation

The type of data and the record length used in the analysis form the primary basis for establishing a range on credible extrapolation of flood estimates. The objective of flood frequency analysis and extrapolation is to provide reliable flood estimates for a full range of AEPs necessary for dam safety decisionmaking. In order to develop reliable flood estimates, flood frequency relationships should include an estimate of the uncertainty around the median values. The data used in the analysis provide the only basis for verification of the analysis or modeling results, and as such, extensions beyond the data cannot be verified. The greatest gains to be made in providing credible estimates of extreme floods can be achieved by combining regional data from multiple sources. Thus, analysis approaches that pool data and information

from regional precipitation, regional streamflow, and regional paleoflood sources should provide the highest assurance of credible characterization of low AEP floods. The information that follows was developed in a workshop sponsored by Reclamation and documented in Bureau of Reclamation, 1999.

For Reclamation dam safety risk assessments, flood estimates are needed for AEPs of 1 in 10,000 and possibly ranging down to as low as 1 in 100,000,000. Developing credible estimates at these low AEPs generally requires combining data from multiple sources and a regional approach. Table 3-1 lists the different types of data that can be used as a basis for flood frequency estimates and the typical and optimal ranges of credible extrapolation for AEP (Bureau of Reclamation, 1999). In general, the optimal ranges are based on the best combination(s) of data envisioned in the western U.S. in the foreseeable future. Typical ranges are based on the combination(s) of data that are commonly available and analyzed for most sites.

Table 3-1.—Data types and extrapolation ranges for flood frequency analysis (Bureau of Reclamation, 1999)

Type of data used for flood frequency analysis	Range of credible extrapolation for annual exceedance probability	
	Typical	Optimal
At-site streamflow data	1 in 100	1 in 200
Regional streamflow data	1 in 500	1 in 1,000
At-site streamflow and at-site paleoflood data	1 in 4,000	1 in 10,000
Regional precipitation data	1 in 2,000	1 in 10,000
Regional streamflow and regional paleoflood data	1 in 15,000	1 in 40,000
Combinations of regional data sets and extrapolation	1 in 40,000	1 in 100,000

Many factors can affect the equivalent independent record length for the optimal case. For example, gaged streamflow records in the western United States only rarely exceed 100 years, and extrapolation beyond twice the length of record, or to about 1 in 200 AEP, is generally not recommended (Interagency Advisory Committee on Water Data [IACWD], 1982). Likewise, for regional streamflow data the optimal range of credible extrapolation is established at up to 1 in 1,000 AEP by considering the number of stations in the region, lengths of record, and degree of independence of these data (Hosking and Wallis, 1997). For paleoflood data, only in the Holocene epoch (or the past 10,000 years) is climate judged to be sufficiently like that of the present climate for these types of records to have meaning in estimating extreme floods for dam safety risk assessment. This climatic constraint indicates that an optimal range for extrapolation from paleoflood data, when combined with at-site gaged data, for a single stream should be up to about 1 in 10,000 AEP. For regional precipitation data, a similar range is imposed because of the difficulty in collecting sufficient station-years of clearly independent precipitation records in the orographically complex regions of the western United States. Combined data sets of regional gaged and regional paleoflood data can be extended to smaller AEPs, perhaps to about 1 in

40,000, in regions with abundant paleoflood data. Analysis approaches that combine all types of data are judged to be capable of providing credible estimates for an AEP range up to about 1 in 100,000 under optimal conditions.

In many situations, credible extrapolation ranges may be less than optimal. Typical ranges would need to reflect the practical constraints on the equivalent independent record length that apply for a particular location. For example, many at-site streamflow record lengths are shorter than 100 years. If in a typical situation the record length is only 50 years, then the range of credible extrapolation might be up to an AEP of about 1 in 100. Similarly, many paleoflood records do not extend to 10,000 years, and extensive regional paleoflood data sets do not currently exist. Using a record length of about 4,000 years, a typical range of credible extrapolation might be up to an AEP of 1 in 15,000 based on regional streamflow and regional paleoflood data.

The information presented in table 3-1 is intended as a guide; each situation is different and should be assessed individually. The ranges of extrapolation should be determined by evaluating the lengths of records, number of stations in a hydrologically homogeneous region, degree of correlation between stations, and other data characteristics that may affect the accuracy of the data.

Ideally, one would like to construct the flood frequency distribution for all floods that could conceivably occur. However, the amount of data and flood experience for any site or region constrain the range of the floods to which AEPs can be assigned based solely on data. In general, the scientific range to which the flood frequency relationship can be credibly extended, based upon any characteristics of the data and the record length, will fall short of the PMF for a site. However, there is a need in dam safety risk assessment to determine the probability of occurrence of very large floods with very small AEPs. The lack of an ideal data set does not absolve the hydrologist from extending the flood frequency relationship to cover the full range of AEPs needed for risk assessment. Therefore, a systematic approach is provided for estimating hydrologic hazard curves that can be used for dam safety decisionmaking.

Floods can be categorized, according to the Australian Rainfall and Runoff: A Guide to Flood Estimation (Nathan and Weinmann, 2001), as large, rare, and extreme. These flood categories are shown in figure 3-1. Large floods generally encompass events for which direct observations and measurements are available. Rare floods represent events located in the region between direct observations and the credible range of extrapolation from the data. Extreme floods generally have very small AEPs, which are beyond the credible range of extrapolation but are still needed for dam safety risk assessments. Occasionally, Reclamation has an interest in floods with an AEP as low as 1 in 10^8.

Extreme floods border on the unknowable. Uncertainty is very large and unquantifiable. Since data cannot support flood estimates in this AEP range, hydrologists and engineers must rely on our knowledge and understanding of hydrologic processes to estimate extreme floods. Oftentimes, these floods may result from unforeseen and unusual combinations of hydrologic parameters generally not represented in the flood history at a particular location. One potential upper bound to the largest flood at a particular site of interest is the PMF.

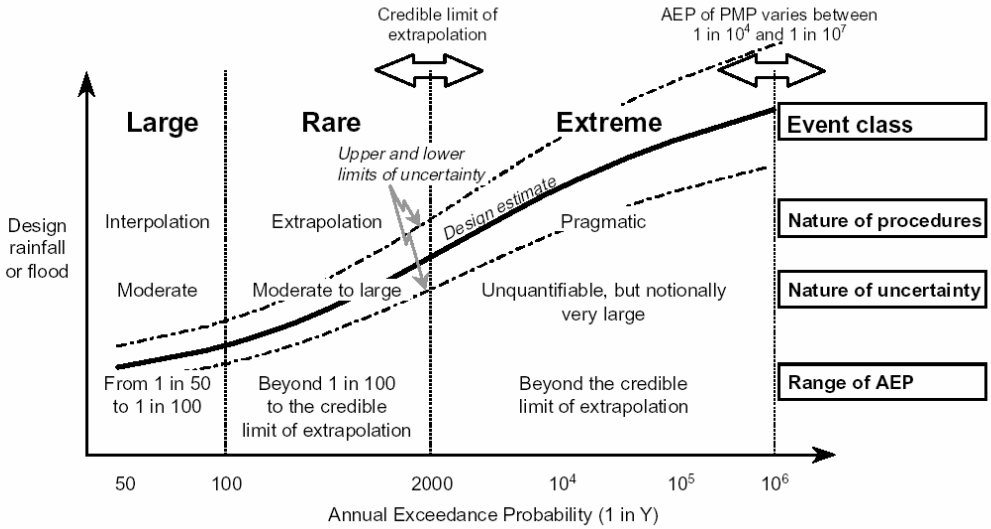

Figure 3-1.—Characteristics of notional floods (Nathan and Weinmann, 2001).

Reclamation uses the PMF as the upper limit of flood potential at a site for storm durations defined by the probable maximum precipitation (PMP). If peak flows or volumes calculated using probability or statistically based hydrology methods exceed those of the PMF, then the PMF is used in evaluating the hydrologic risk and as a theoretical and practical upper limit to statistical extrapolations. The PMF is defined as "the maximum runoff condition resulting from the most severe combination of hydrologic and meteorological conditions that are considered reasonably possible for the drainage basin under study" (Cudworth, 1989). If the PMF has been properly developed, it represents the upper limit to runoff that can physically occur at a particular site. Various storm types, sequences, and durations are taken together with the most severe hydrologic parameters in its development. Extrapolation of statistical analyses can become unbounded for flood distributions that exhibit positive skewness; therefore, Reclamation uses the PMF to limit extrapolation to flood discharges that are physically possible.

Oftentimes, reliable flood frequency estimates are needed for very low AEPs for use in dam safety decisionmaking. Considerable judgment may be required to extrapolate out to the low AEPs required for risk assessment. Even though the flood frequency estimates become less reliable as they are extended beyond the optimal ranges, a systematic way of doing this provides a useful decision making tool.

3.3 Flood Peak and Volume Relationships

Hydrologic hazard relationships display peak flow and flood volumes for various durations versus AEP. Figure 3-2 is a hypothetical example of the type of relationship needed to address hydrologic dam safety issues. Floods with AEPs as low as 1 in 10^8 are desired to encompass the full range of events needed for dam safety risk assessment. The next section of this report describes the available approaches for developing the flood peak and volume relationship. Some of the approaches will also produce flood hydrographs, which can be routed through the reservoir.

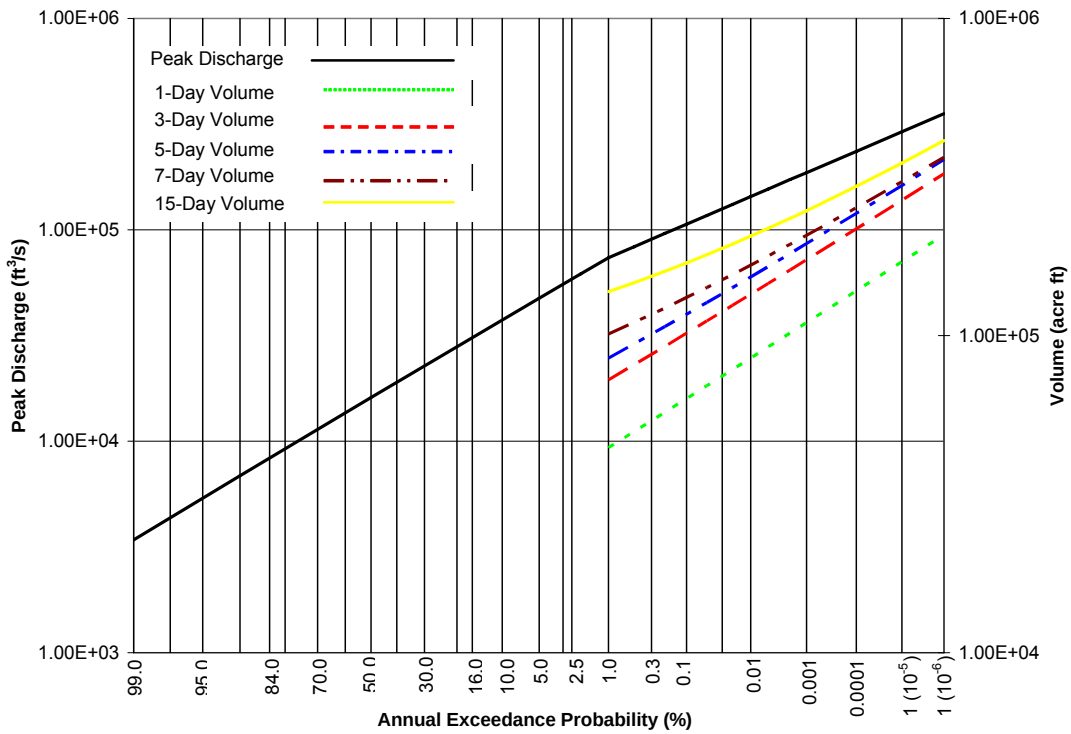

Figure 3-2.—Example hydrologic hazard curve.

4. Analysis Techniques

The main probabilistic and engineering hydrology methods that are currently being used, applied, and under investigation by the Flood Hydrology Group are summarized in this section of the report. There are seven general techniques:

- Flood frequency analysis with historical/paleoflood data
- Hydrograph scaling and volumes
- The GRADEX Method
- The Australian Rainfall-Runoff Method
- Stochastic event-based precipitation runoff modeling with the stochastic event flood model (SEFM)
- Stochastic rainfall-runoff modeling with CASC2D

- The PMF

Other models and approaches are briefly noted by reference in each section. General sources of models and approaches for estimating extreme floods are listed in Maidment (1993), Singh (1995), and Bureau of Reclamation (1999). Methods to calculate extreme floods and associated probabilities have recently been revised and published in the United Kingdom (Institute of Hydrology, 1999) and Australia (Nathan and Weinmann, 2001).

4.1 Flood Frequency Analysis with Historical/Paleoflood Data

There are three main techniques that Reclamation currently uses to develop a peak-flow frequency curve and integrate streamflow (gage) data, historical data, and paleoflood data. The first is a mixed-population graphical approach (England et al., 2001). The two other techniques are statistical models that use gage, historical, and paleoflood data. The Expected Moments Algorithm (EMA) (England, 1999) uses moments to estimate the parameters of a log-Pearson Type III (LP-III) distribution and is consistent with Bulletin 17B (IACWD, 1982). A Bayesian maximum likelihood approach is used by FLDFRQ3 (O'Connell, 1999) to estimate a peak-flow frequency curve with historical and paleoflood data and uncertainties. All three techniques have been used for estimating flood peaks at various Reclamation dams.

4.1.1 Historical and Paleoflood Data

Many different kinds of historical and paleoflood data can be used for flood frequency analysis. Historical flood data are typically extreme floods that have occurred and were described in some qualitative or quantitative fashion before establishing a stream gaging station. The typical information that is available for historical floods is the date of occurrence and the height of the water surface (Thomson et al., 1964). In many cases, people physically mark, on a relatively permanent surface, the approximate high-water mark of a flood (Thomson et al., 1964; Leese, 1973; Natural Environment Research Council, 1975; Sutcliffe, 1987; Fanok and Wohl, 1997).

Paleoflood hydrology is the study of past or ancient floods that occurred before the time of human observation or direct measurement by modern hydrologic procedures (Baker, 1987). The basic types of paleoflood indicators that are useful for flood frequency analysis are paleostage indicators and botanical evidence (Wohl and Enzel, 1995; Baker, 2000). Recent investigations, techniques, and analyses for collecting and using paleoflood data are discussed in House et al. (2002). Fluvial geomorphic evidence includes erosional and/or depositional features that are used to infer paleostages or non-inundation levels. The fluvial geomorphic evidence used in paleoflood and flood frequency studies that represents paleostage indicators includes: silt lines, scour lines, slackwater deposits, boulder and gravel bars, and modified geomorphic surfaces (Costa, 1978; Baker, 1987; Kochel and Ritter, 1987; Jarrett and Costa, 1988; Salas et al., 1994; Jarrett and England, 2002; Levish, 2002). Botanical evidence consists of vegetation that records evidence of a flood (or several floods) or indicates stability of a geomorphic surface for some time period. Botanical evidence of floods includes: corrosion scars, adventitious sprouts, tree age, and tree-ring anomalies (Hupp, 1987).

The historical and paleoflood data can generally be represented with four major data classes (Stedinger et al., 1988): floods of known magnitude, floods of unknown magnitude that are less than some level, floods of unknown magnitude that exceed some level, and floods with magnitudes described by a range. Historical and paleoflood data generally are described in terms of exceedance or non-exceedance of a discharge threshold (Q_o). To correctly interpret the data, one needs to understand the mechanisms by which historical and paleoflood records document the magnitudes of floods that either did, or did not, occur (Stedinger et al., 1993). In many situations, one knows the magnitude of each flood. Annual (gage) peak discharge estimates, historical floods, and paleofloods whose magnitudes are known are described by "floods of known magnitude" class. For example, the solid bars depicted in figure 4-1 describe known floods in the gage and historical period.

The most common situation for using historical and paleoflood data in flood frequency analysis is that a peak discharge Q is known to be smaller than some threshold Q_o, but the magnitude of Q is unknown. The shaded region in figure 4-1 represents these unknown floods that are below a threshold Q_o. The total record length (n) is the sum of the systematic (s) and historical/ paleoflood (h) record lengths (n=s+h). We define the number of observations that exceed the threshold in the systematic record (s) as e, which is equal to 1 in figure 4-1. The number of known observations in the historical period (h) is designated e' (equal to 3 in figure 4-1); it is also known that the values are greater than Q_o. We define k as a random variable equal to the number of observations greater than Q_o in the entire record n, where k = e + e'. The number of observed floods is denoted g, where g = s + k - e.

In some cases, one may know that no floods exceeded the discharge threshold, or k=0. Data in this case have been termed a "non-exceedance bound" (Levish et al., 1994; Levish, 2002), where one has knowledge that no flood has exceeded a designated threshold or geomorphic surface in some time period. An example of a non-exceedance bound is knowledge that a river has not inundated "Main Street," a bridge deck spanning a river, or a geomorphic surface in some time period. Knowledge that k = 0 is valuable information that can be used in flood frequency analysis.

Historical and paleoflood information may be described in terms of a flood that exceeded a threshold, with no upper bound. In some cases, one knows only that a flood was larger than some level and does not know the magnitude of the flood. One knows the number of floods that exceeded the discharge threshold. Stedinger and Cohn (1986) have termed this category as "binomial censoring," in which the exact magnitude of a value is unknown except that it exceeded a lower threshold (see also Russell, 1982). This situation is common for some types of botanical investigations where one can, at present, determine only the minimum stage for plant damage (Hupp, 1988).

There are many situations in which one does not know the exact magnitude of a flood, but that it lies within a range or interval. Interval censoring is used when the exact magnitude of a flood is unknown, but is known to be between some upper and lower amount (Stedinger et al., 1988; Cohn et al., 1997). This class can be used to describe floods with measurement uncertainty. In some cases, the upper threshold, Q_u, can vary for each observation, depending on the data source.

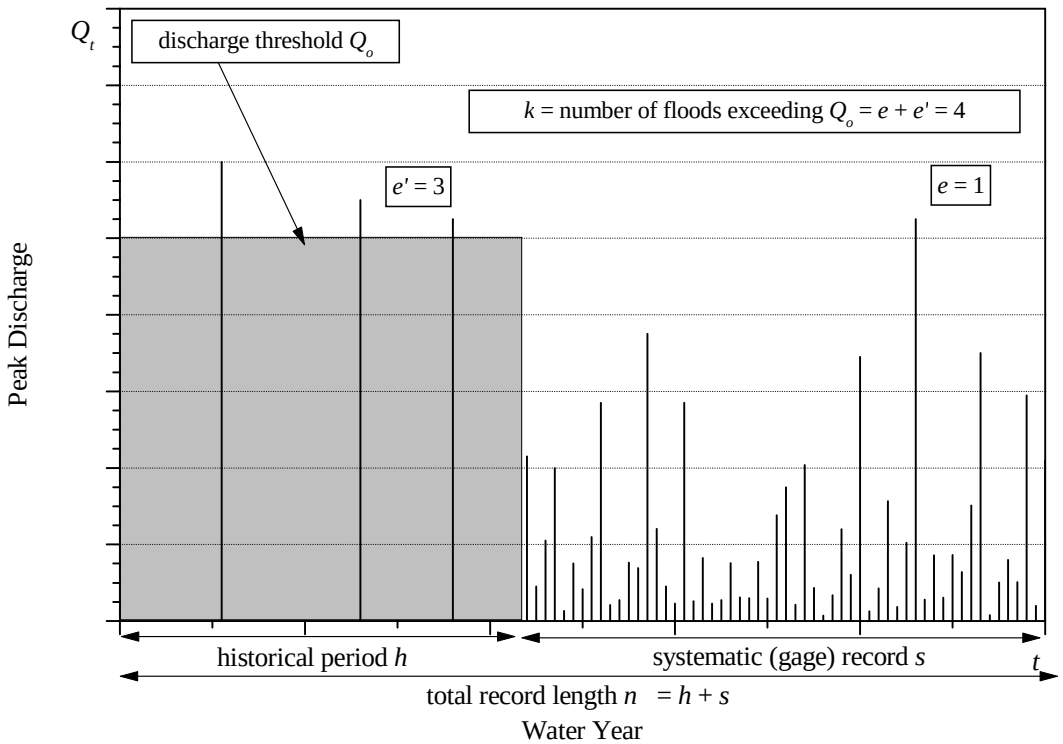

Figure 4-1.—Example of peak discharge time series with historical period and discharge threshold Q_o. The shaded area represents floods of unknown magnitude less than Q_o.

4.1.2 Mixed-Population Graphical Approach

A mixed-population graphical peak-discharge frequency approach has been developed by Reclamation (England et al., 2001). The graphical approach is an at-site frequency method and the frequency curve is constructed in two distinct parts: (1) standard hydrologic statistical methods are used to define a frequency curve for return periods less than and including the 100-year return period (e.g., IACWD, 1982; Ries and Crouse, 2002) and (2) graphical methods are used for estimates greater than the 100-year return period. Peak discharge estimates from gaging stations are used to define the first part of the curve and at-site paleoflood data are used to define the second part of the curve. The first part is estimated assuming an LP-III distribution. One of three at-site techniques and associated computer programs is typically used to estimate the parameters of the LP-III distribution, calculate quantiles, and estimate confidence intervals: (1) the Bulletin 17B Method (IACWD, 1982) and FREQY (Carson, 1989); (2) expected moments methods (Cohn et al., 1997) and EMA (England, 1999); or (3) Bayesian maximum likelihood and FLDFRQ3 (O'Connell, 1999). Historical information is included in the at-site frequency analysis when it is available. Historical data can be used to adjust a so-called "high outlier" using FREQY, EMA, or FLDFRQ3. Low outliers can be adjusted using IACWD (1982) methods. The second portion of the frequency curve is estimated assuming a 2-parameter log-Normal (LN-2) distribution. It is defined between the 100-year and the available paleoflood data return periods, and extrapolated beyond the paleoflood data using this LN-2 distribution. Two points are typically used to estimate this portion of the flood-frequency curve: (1) the LP-III

model 100-year peak discharge estimate and (2) the midpoint in time and discharge of the paleoflood data. Logarithms (base 10) of the peak flows and standard Normal variates of return periods are used to estimate the LN-2 parameters using least squares (England, 2000). The LN-2 distribution was found to reasonably represent daily standardized precipitation in the western United States (Lane, 1997).

The mixed-population graphical approach is used to estimate flood hazard curves. The approach has been developed so that one can estimate an extreme flood frequency curve at any location in the western United States with a minimal amount of effort using existing streamflow data and some site-specific paleoflood data. There are two main assumptions of this graphical approach for estimating extreme flood probabilities: the upper portion of the frequency curve is appropriately defined by the 100-year peak discharge and paleoflood data and the extrapolation of this portion of the curve using a LN-2 model is appropriate. An example peak-flow frequency curve using the graphical approach is shown in figure 4-2. The approach has been reviewed by Kuczera (2000). Kuczera pointed out the major weaknesses were the use of an envelope curve, lack of confidence intervals, and extrapolation. Kuczera recommended that regional growth curves be used to compliment the use of envelope curves.

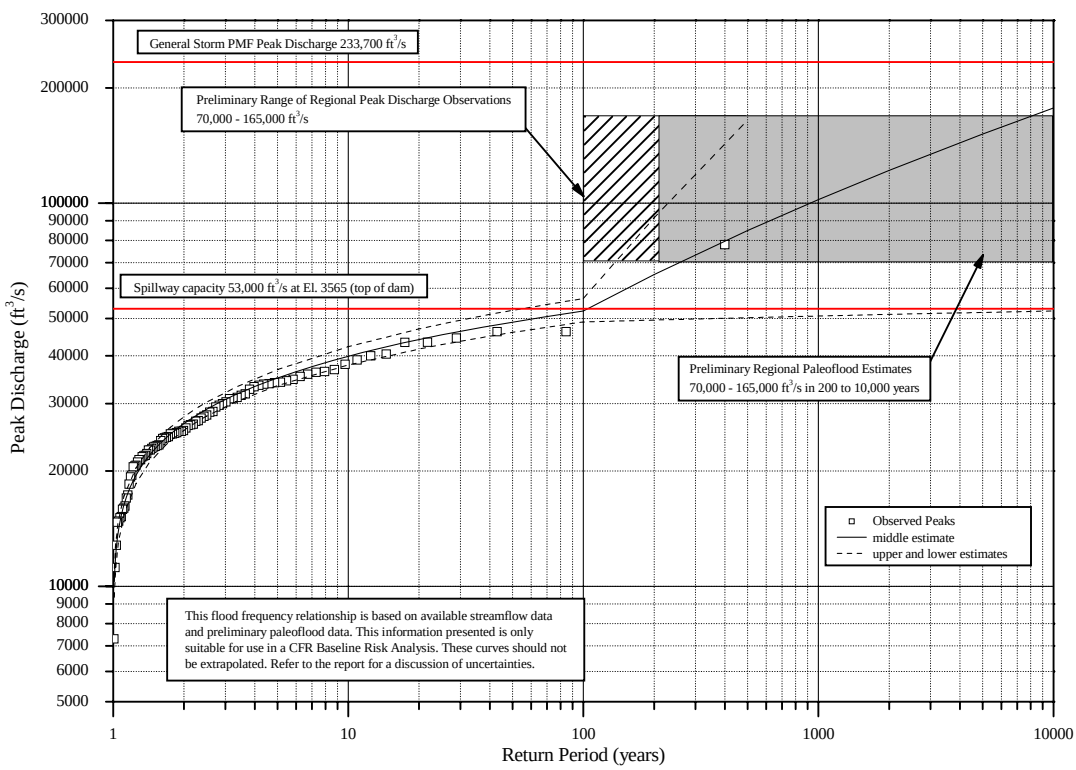

Figure 4-2.—Example application of mixed-population graphical flood frequency curve using peak discharges on the South Fork Flathead River near Hungry Horse, Montana.

4.1.3 Expected Moments Algorithm

The EMA (Lane, 1995; Lane and Cohn, 1996; Cohn et al., 1997, 2001) is a new moments-based parameter estimation procedure that was designed to incorporate many different types of systematic, historical, and paleoflood data into flood frequency analysis. EMA assumes the

LP-III distribution is the true distribution for floods. EMA was designed to handle the four different classes of historical and paleoflood data beyond the applicability of the Bulletin 17B historical weighting procedure (IAWCD, 1982). As noted by Cohn et al. (1997, 2001) and England (1998), EMA is philosophically consistent with, and is an improvement to, the Bulletin 17B method of moments procedure when one has historical or paleoflood information. EMA is specifically designed to use historical and paleoflood data, in addition to annual peak flows from gaging stations, in a manner similar to Maximum Likelihood Estimators (Lane and Cohn, 1996). It is a more logical and efficient way to use historical and paleoflood data than the current Bulletin 17B historical method, and it is a natural extension to the moments-based framework of Bulletin 17B.

The five basic steps of EMA are:

(1) Estimate an initial set of the three sample statistics $\left(\hat{\mu}, \hat{\sigma}^2, \hat{\gamma}\right)$, from the floods with known magnitudes. These floods are typically observations from the gaging station record and possibly some historical or paleofloods. At this step, floods with unknown magnitudes and magnitudes described by a range are not included.

(2) Based on the initial sample statistics from step (1), estimate a set of the LP-III distribution parameters $\left(\hat{\tau}, \hat{\alpha}, \hat{\beta}\right)$.

(3) From the set of LP-III parameters from step (2), estimate a new set of sample moments based on the complete data set: known-magnitude floods, floods less than some threshold(s), unknown magnitude floods that exceed some threshold(s), and floods described by a range.

(4) From this new set of moments, estimate a new set of LP-III parameters.

(5) Compare the parameters from step (4) to those computed from step (2). Repeat steps (3) and (4) until the parameter estimates converge. The main equations used by EMA are listed in Cohn et al. (1997), England (1999), and England et al. (2003).

EMA has been rigorously peer reviewed in the literature (Cohn et al., 1997, 2001; England et al., 2003a, 2003b) and provides a suitable flood frequency model. EMA has been applied at many sites for peak-flow frequency (England et al., 2003b). The National Research Council applied EMA for 3-day annual maximum mean floodflows on the American River (NRC, 1999). An example peak-flow frequency curve with EMA is shown in figure 4-3. There are several limitations with the current version of EMA: (1) the program assumes that the distribution is LP-III, (2) software has not been fully developed to implement the confidence interval technique of Cohn et al. (2001), and (3) low outlier and regional skew methods with EMA have been recently developed (Griffis et al., 2003), but not tested with actual data.

4.1.4 FLDFRQ3

FLDFRQ3 (O'Connell, 1999; O'Connell et al. 2002) uses a Bayesian maximum likelihood procedure to estimate parameters of various distributions. The Bayesian approach includes

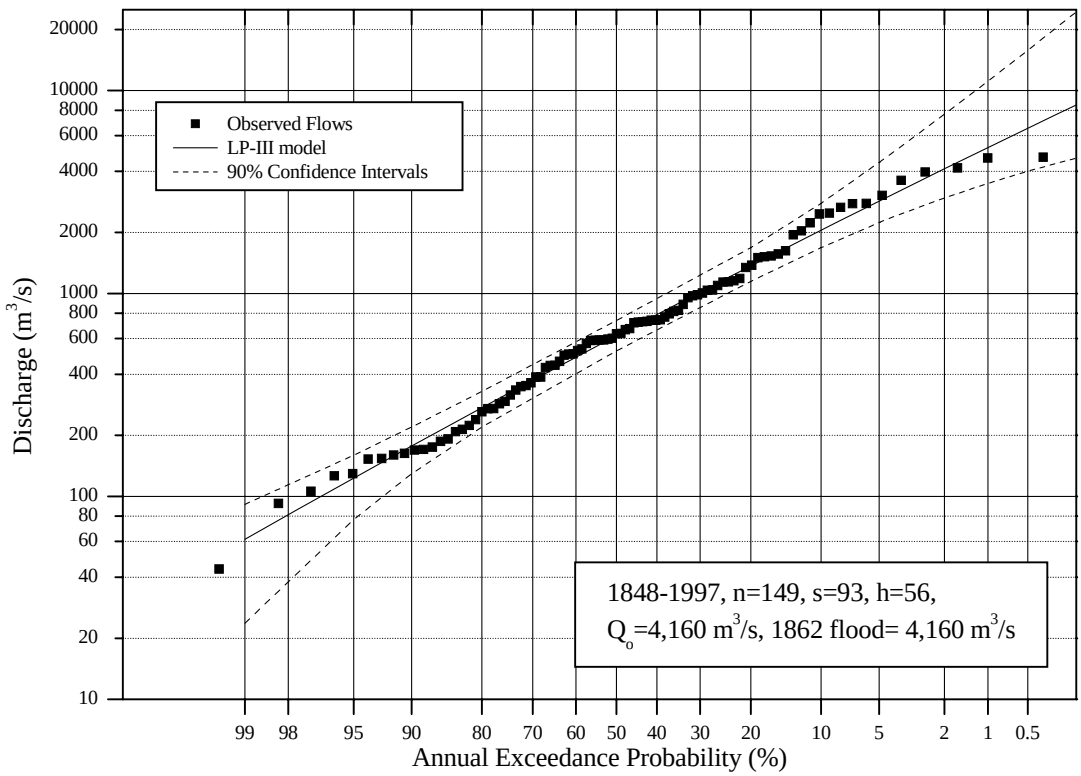

Figure 4-3.—Example application of EMA for American River annual maximum 3-day mean discharge frequency analysis.

measurement uncertainty in the parameter estimation procedure. This approach uses a "global" parameter integration grid in order to identify ranges of probability distributions that are consistent with the data (O'Connell, 1999). Two measurement error sources are included: peak discharge measurement errors and errors in paleohydrologic bound ages. Bayesian methods (Tarantola, 1987) and likelihood functions modified from Stedinger and Cohn (1986) are used to incorporate data and parameter uncertainties. Two options can be used to find the "global" maximum likelihood estimate in FLDFRQ3 (O'Connell, 1999): simulated annealing and the downhill simplex method. In FLDFRQ3, one is able to choose among five main three-parameter probability distributions to assume a peak discharge parent distribution. These distributions are the Generalized Extreme Value, Generalized Logistic, Generalized Normal, Generalized Pareto, and Pearson Type III (P-III) (Hosking and Wallis, 1997). These distributions include two logarithmic transform options for P-III models to include the LP-III. O'Connell (1999) provides details of the numerical approach used for estimating distribution parameters and uncertainty using grid integration.

There are generally three main steps in running FLDFRQ3 (O'Connell, 1999): input and data check, parameter estimation for a particular distribution, and generating parameter uncertainties for a particular model (e.g., LP-III) using grid integration. The data are grouped into two broad classes: data with normal uncertainties, such as peak discharge, and values in a range with potentially variable probability density and skew within the range, such as paleohydrologic bound discharges and ages and discrete paleofloods. After entering and checking data, the parameter estimates are obtained from the data and assumed model. The user then checks the appropriateness of the model and estimated parameters. There can be several steps here to

determine the "best models" (there can be more than one) that fit the data and the model parameters. Finally, the user estimates the parameter uncertainty given the chosen model and parameter combination. O'Connell et al. (2002) demonstrate how to combine results of several models and their parameter uncertainties using a likelihood criterion.

FLDFRQ3 has been rigorously peer reviewed in the literature (O'Connell et al., 2002) and contains suitable flood frequency models for all levels of analysis. It has been used at many sites for peak-flow frequency, such as Folsom Dam (Bureau of Reclamation, 2002), Seminoe and Glendo Dams (Levish et al., 2003), and Pathfinder Dam (England, 2003). An example peak-discharge frequency curve using FLDFRQ3 is shown in figure 4-4.

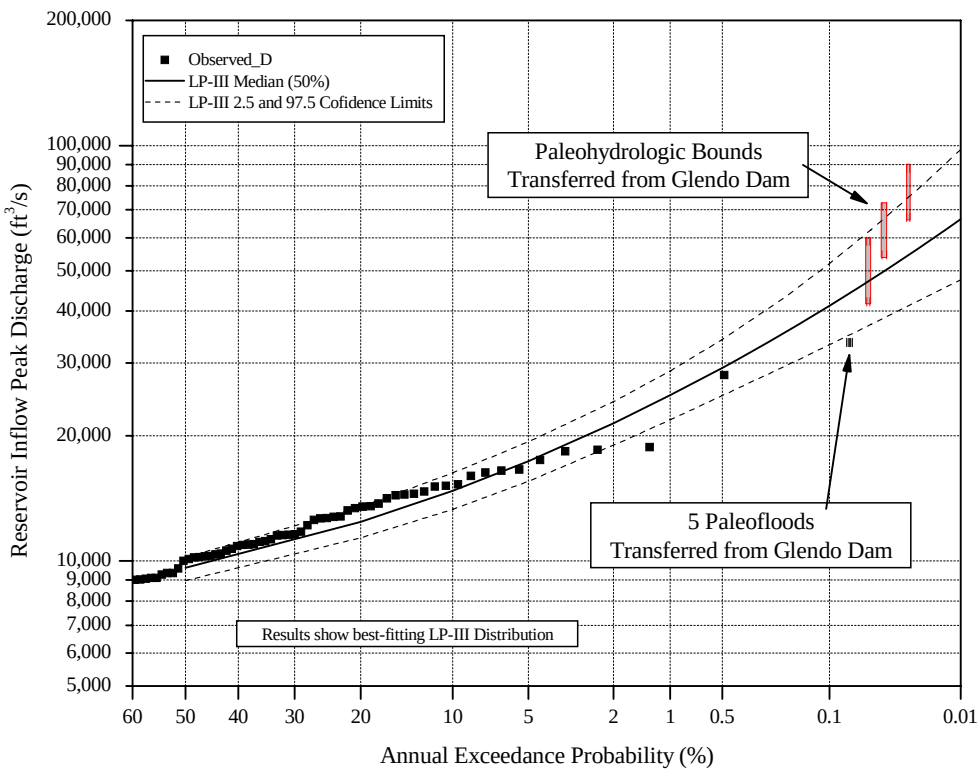

Figure 4-4—Annual peak-discharge frequency inflows to Pathfinder Dam, Wyoming, from best-fitting LP-III distribution using FLDFRQ3 (England, April 2003).

4.2 Hydrograph Scaling and Volumes

Practical tools have been developed for estimating probabilistic hydrographs that can be used in risk analyses for dam safety. These tools are presented in England (2003a) and are summarized below. The key feature of the approach is to use peak-discharge frequency curves that include paleoflood data as a basis to develop hydrographs and volume frequency curves. The methods are relatively flexible and can be tailored to different types of investigations. The methods need to be adjusted depending on the available data at the site and region of interest. For example, if a peak-discharge frequency curve developed using the graphical approach is available, one could use less detailed methods to develop hydrographs because the data might not warrant sophisticated techniques. In contrast, if detailed, high-quality peak discharge and paleoflood

data are available, one could use more refined methods such as the SEFM (MGS Engineering Consultants, Inc. [MGS], 2001) discussed below.

Probabilistic hydrographs can be constructed based on streamflow estimates from gaging stations, historical data, and paleoflood data. Four components are used: (1) a peak discharge-probability relationship, (2) an extreme storm duration probability relationship, (3) relationships between peak discharge and maximum mean daily flow volumes, and (4) observed hourly flow hydrographs that have regulation effects removed. The key idea is calibration or scaling of hydrographs to match peak discharge for a given probability. The approach relies completely on the specification of a peak-flow frequency curve that describes the probabilities of interest, based on paleoflood data.

There are four major assumptions for developing the hydrographs: (1) the probability of peak discharge represents a probability of the composite hydrograph, (2) unit hydrograph assumptions apply to the basin, (3) direct runoff volumes can be estimated from daily flow hydrographs, and (4) the recorded streamflow observations, historical information, and paleoflood data in the river basin of interest provide an adequate sample so one can extrapolate peak discharge probabilities, peak-volume relationships, and hydrographs for extreme floods. Maximum mean discharge ($\overline{Q}_d$) for n-day periods is related to peak discharge (Q_p) by a power function:

$$\log \overline{Q}_d = a + b \log Q_p \tag{1}$$

The assumed known variable is peak discharge (Q_p), with an associated exceedance probability estimate from the frequency curve. The quality of the regression relationship expressed in equation (1) depends principally on the data from the site of interest and the flow duration (n). Mixed-population flood data (e.g., from thunderstorms, snowmelt, or rain-on-snow) can lead to difficulties in obtaining statistically significant relationships. Good regression fits are typically found for shorter duration (1- to 7-day) flow volumes; the relationships become progressively worse for longer durations. The maximum n-day hydrograph ordinates are linearly scaled, based on the selected n-day volume.

An alternate approach to using streamflow data is to use hydrographs from rainfall-runoff models as a basis for scaling. In these cases, there are typically no flood hydrograph data at the site of interest. A design flood hydrograph, a PMF hydrograph, or other suitable hydrograph for the basin is obtained. The hydrograph can then be scaled in some linear fashion to match peak flows from a peak-flow frequency curve. The analyst needs to be careful to ensure that flood volumes do not exceed physical limits when applying this scaling procedure.

Probabilistic hydrographs, developed from scaling streamflow observations or from rainfall-runoff models, are combined with recommendations for initial reservoir levels for hydrograph routing. Reservoir routing issues and selection of varying initial levels are discussed in England (2003a). One can then determine a maximum reservoir level by routing the given hydrograph and initial reservoir level. Initial reservoir levels can sometimes have a large effect on maximum reservoir level estimates for extreme floods. Maximum reservoir elevation probability estimates depend on the inflow hydrograph peak, volume, shape, and probability estimate. The initial reservoir level can also be a major factor. The selection of an appropriate initial reservoir level is of considerable importance in determination of spillway adequacy (Nathan and Weinmann,

1999, p. 57). For estimating maximum reservoir levels for design floods such as the PMF, Reclamation uses a fixed initial reservoir level. This initial reservoir level is usually set at the top of active conservation or bottom of the flood control pool. This assumption has been criticized as being unduly conservative. Newton (1983, p. 914) notes that current practice for most agencies is to assume conservatively high initial pool levels for routing PMFs. Instead of using a fixed initial reservoir level for routing hydrographs, variable initial reservoir levels are needed for risk analysis. Initial reservoir levels and associated exceedance probabilities should be estimated from daily reservoir elevation estimates for the period of record at the site of interest.

4.3 GRADEX Method

Much of this description of the GRADEX Method is paraphrased from the Ph.D. dissertation, "Methodology for Estimating the Upper Tail of Flood-Peak Frequency Distributions Using Hydrometeorological Information," by Mauro Da Chunha Naghettini, completed in partial fulfillment of the requirements for the Ph.D. degree at the University of Colorado, Department of Civil, Environmental, and Architectural Engineering, 1994. Naghettini, Potter, and Illangasekare later described the same method in the Water Resources Research publication in 1996. Some additional comments related to Reclamation dam safety needs are inserted when appropriate.

In its 1988 report, the National Research Council Committee on Estimating the Probabilities of Extreme Floods identified principles for improving the estimation of floods with AEPs on the order of 10^{-3} or smaller. These principles are: "(1) 'substitution of space for time'; (2) introduction of more 'structure' into the models; and (3) focus on extremes or 'tails' as opposed to or even to the exclusion of central characteristics" (NRC, 1988). The methodology proposed in Naghettini's Ph.D. dissertation (1994) presents techniques for the estimation of extreme flood peaks and volumes that make strong use of these principles. The main objective is to develop a peak-flow frequency curve for the extremely rare probabilities. To do so, the method involves a peak to volume relationship and the derivation of a frequency curve of extreme flood volumes based on extreme regional rainfall statistics. The method is useful to current Reclamation dam safety needs in that it provides a means to produce frequency curves for rare flood volumes and also some apparatus to define peak flows for the extreme flood volumes. It can also be used to create hydrographs based on the flood volumes and peaks, if needed.

The method relies on extrapolating a conventionally estimated probability distribution of flood volumes. To strengthen this step, the GRADEX Method, originally developed by Guillot and Duband (1967), is incorporated. The GRADEX Method has been used extensively in France since about 1967 for various improvements and hydrologic safety investigations and spillway renovations at numerous hydroelectric dams and facilities. The French Committee on Large Dams has prepared the publication, *Small Dams* (undated), which outlines the very basic steps that can be used to perform such calculations in France. The main GRADEX Method is based on two assumptions:

(1) That, asymptotically, the upper tail of the flood volume distribution is exponential with the same scale parameter as that which describes the upper tail of the

distribution of rainfall volumes for the basin. Figure 4-5 graphically displays this assumption.

(2) That any increase in total precipitation during a severe rain event, falling on already saturated ground, will produce a corresponding increase in volume of the resulting flood.

The estimation of the rainfall scale parameter has been enhanced in this application from the original French methodology by incorporating the work of Smith (1989), who developed a regional model for estimating the upper tail of a frequency distribution based on extreme order statistics. Figure 4-5 depicts the GRADEX Method.

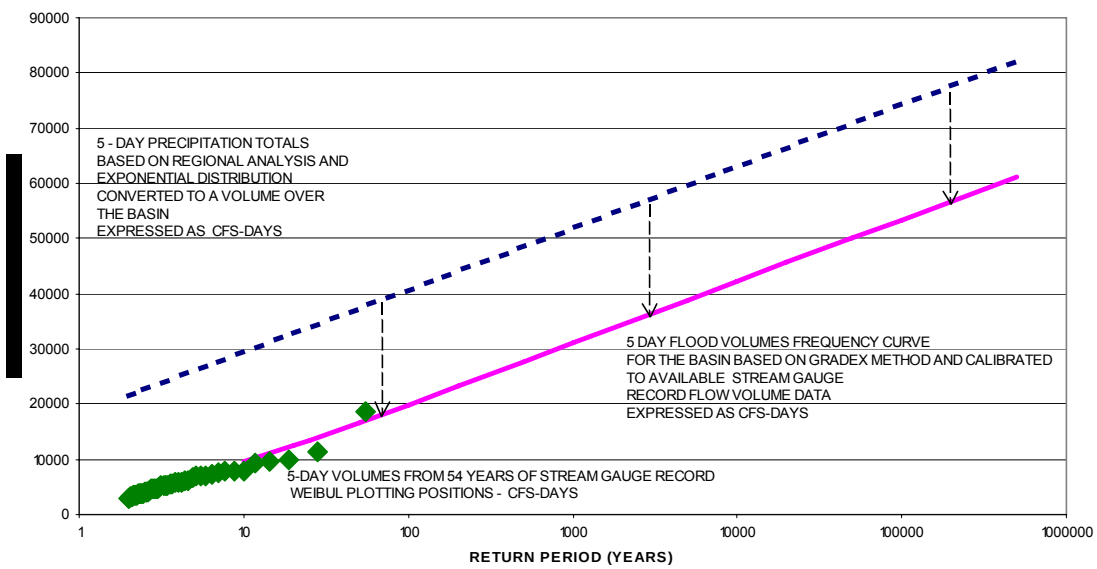

Figure 4-5.—GRADEX Method of volume frequency curve calculation.

The location where the extrapolated flood volume curve takes over from a more conventional analysis of stream gage volume data, such as using LP-III, is not fixed, but must be assumed. In the literature from France, this return period ranges from about 10 years, for very impermeable basins, to 50 years for very permeable basins.

The first assumption of the GRADEX Method given above refers to the upper tail of the rainfall volume distribution, which is assumed to be a generalized Pareto density function of the form:

$$g_p(p|s,K) = 1/a\ [1- ((Kp)/a)]^{1/K - 1} \quad \text{if } K \neq 0 \qquad (2)$$

This will reduce to a simple exponential density function of the form:

$$g_p(p|s,K) = (1/a)\exp(-p/a) \quad \text{if } K = 0 \qquad (3)$$

Where the positive constants K and *a* are the location and scale parameters, respectively. The scale parameter a is a function of various physical components of the available rain gage data

sets such as elevation and mean annual precipitation (MAP). If K > 0 then the distribution of rainfall for all sites has an upper bound; if K < 0 it is unbounded. If K = 0 the upper tail of the distribution is exponential with a scale parameter *a*. The parameter estimates are found by fitting a distribution that asymptotically exhibits an exponential upper tail (e.g., Exponential, Gumbel, Gamma, or log-Normal) to rainfall maxima. Combining the two GRADEX assumptions causes the upper tail of the flood volume distribution to also be exponential with the same scale parameter *a* (the GRADEX parameter) as the one estimated for the upper tail of the distribution of rainfall volumes, except for a necessary conversion to units of volume instead of precipitation depth.

The first step in the GRADEX Method involves selecting a critical duration. This begins with an examination of the time series of unregulated daily flows for a stream gage record deemed to be hydrologically similar to the basin being studied or a record of reservoir daily inflows. What is required is a series of independent flood events (hydrographs) that have occurred over the entire length of the unregulated streamflow record. These flood events should be rain-generated, as opposed to floods derived from snowmelt. Also, the rain-generated flood events should all be of the same storm type. For these reasons, the stream gage record analysis should be limited to a "season" when the rain floods of the same type are most likely to occur based on historic experience. Once the season is selected, the daily streamflows for each year within that season are examined. A threshold discharge, Q threshold, is set. The number of daily flows above this threshold value is observed. Multi-day events with several days of flow above the threshold are observed. The number of and the duration of each of these multi-day events are then calculated. It is desired to obtain a set of independent flood events with nearly the same number of events as the number of years in the length of record. If the number of events calculated is too large or small, then the threshold Q value is raised or lowered until approximately the number of events equals the number of years in the stream gage record. The average duration for the entire set of events is then calculated. This average duration, generally raised to the next highest number of days, will become the critical duration d used for the rest of the study. Once the critical duration *d* is determined, the average flow discharge of all of these events can be calculated. A second flow value, termed the reference discharge, is also determined such that 90 percent of the selected flood events will have average *d*-day flow values less than this discharge value. An approximate return period is also placed on this reference discharge value by the inverse of the Gringorten plotting position formula.

$$\text{Reference return period} = 1/\left((i - 0.44)/(N + 0.12)\right) \tag{4}$$

Where N is the total number of years of record and *i* is the rank of the selected reference discharge. This part of the analysis can require much hydrologic judgment. Often, a set of daily flows will show a pattern that is above the selected threshold Q for 1, 2, or 3 or more days, then drop just below the threshold for 1, 2, or 3 days, and then continue for a few more days above the threshold. Decisions have to be made as to whether this should all be considered one flood event or separated into two or more events. Rainfall records from the area may help with this decision, but, generally, it is left to the analyst to make the decision. Independence of the events is generally assumed if the time from the end of the first event to the beginning of the next event is longer than the critical duration that is calculated for this set of events.

In hydrology, this process is often referred to as a marked point process. Much literature is available dealing with statistical assumptions related to events derived by the marked point process. By its nature, the set of floods derived by this process may include several events in any one year, and no events for several years. This is what is desired because the data will be used to help calibrate the GRADEX-derived flood volume information from rainfall totals for the critical duration. It is often the case that several large rainfall events can occur in any one year. Cautions that are given for the selection of the critical duration are: (1) that selection of too short a duration might result in non-exponential, probably heavier-than-exponential, upper tails for the rainfall volumes and (2) adoption of too long a duration might result in poor peak-volume relationships. Since the goal of the application of the GRADEX Method to Reclamation dam safety investigations is to create a good volume relationship, it is advised to raise the computed critical duration value to the next higher full day.

In conventional applications of the GRADEX Method, the parameter *a* can be estimated by fitting an exponentially tailed distribution to seasonal or annual rainfall maxima. The simplest estimation procedure of the GRADEX parameter is to fit a Gumbel distribution to a series of annual maximum rainfall events for a duration *d* that is equal to the watershed critical duration, or some other measure based on time of concentration calculations. However, the most frequently used estimation procedure is to fit an exponentially tailed distribution to seasonal (sometimes monthly) rainfall maxima and then combine the seasonal (monthly) distributions to obtain the annual distribution. What can be shown is that the annual frequency curve, which would no longer be a strictly exponential curve, will tend to have the same shape or slope (GRADEX) as that of the month that produces the largest rainfall amounts, especially at the extreme upper end. A slightly more conservative approach is to use smaller durations of seasonal maxima rainfall totals for even smaller durations, even 24 or 48 hours.

Estimation of the GRADEX parameter of flood volumes requires that different units for expressing the rainfall be used. If the drainage area and critical duration *d* are expressed as mi^2 and days, respectively, and the GRADEX parameters are to be expressed in English units, then:

$$\text{Flood Volume GRADEX} = [(26.89 * DA)/d] * \text{Rainfall GRADEX} \qquad (5)$$

Similar expressions exist for computations done in metric units.

Usually, following the French examples, the extrapolation of the flood volume distribution according to the GRADEX parameter starts at the 10-year flood for small and relatively impervious basins, or at the 20-year flood for larger basins, or possibly the 50-year flood for watersheds showing very little topographical relief or high infiltration capacity.

The current application of the GRADEX Method applies a new methodology to estimate the slope of the rainfall durations for a critical duration d within a specified season. This new approach combines deterministic constraints with contemporary statistical techniques, extracting the maximum information from the available data. The regional rainfall frequency model described in this section is based on the premise that meteorological processes affecting large rainfall events may be different from those affecting smaller rainfall events. The model is an adaptation of a regional flood frequency model developed by Smith (1989) and is based on results from extreme value theory. In this model, the parameters *a* and *K* in the Pareto or

exponential distribution functions (equations 2 or 3) are determined based on a regional analysis of the largest *d*-day rainfall totals for several daily rainfall stations that are shown to be or believed to be homogeneous and to represent the meteorological conditions of the basin under study. The parameter *a* is further allowed to be a function of the basin mean annual precipitation and the basin mean elevation.

$$a = S_i = \exp(c + b_1 W_{i1} + b2 W_{i2}) \tag{6}$$

Where W_{i1} and W_{i2} are the natural logarithms of the set of rain gage elevations and mean annual precipitation values for each gage site i, respectively. The constants b_1, b_2, and c are determined as part of the parameter estimation process. This is an improvement over the French general cases where only one rain gage or set of regional information reduced to one point for any basin may be used. Further, no consideration of elevation or mean annual precipitation is given in the standard GRADEX analysis.

The mathematical process to estimate the parameters K, c, b_1, and b_2 from the data set of rainfall totals proceeds as a maximum likelihood parameter estimation process. A log-likelihood function is then formed.

$$L_n(K, b_1, b_2, c) = \sum \sum \ln[g_p(Zg / K, b_1, b_2, c)] + C \tag{7}$$

Zg is the set of random variables of *d*-day rainfall totals above the threshold precipitation value at each gage site. The double sum is for all of precipitation values above the threshold value at each site and then summed over all sites.

Partial derivatives of the log-likelihood function with respect to the four parameters to be estimated (K, b_1, b_2, and c) are derived. In taking the partial derivatives, the additional constant C in the log-likelihood function is eliminated. These partial derivative functions are then set equal to zero, and a series of four non-linear equations with four unknowns (if $K \neq 0$) or three non-linear equations with three unknowns (if K = 0 is assumed) are formed. As part of the parameter estimation process, statistical tests are performed to see if the three-parameter exponential distribution form is equally valid for the data set as is the four-parameter Pareto distribution. In almost all cases, this is true. The assumption that the extreme rainfall totals can follow an exponential distribution is validated, and the rest of the GRADEX Method follows. The three parameters are then used to form the single scale parameter a (equation 5) for a single parameter exponential distribution form. In cases where the statistical test does not prove the validity of the three-parameter exponential distribution form, the rainfall total data sets need to be further investigated as to homogeneity.

Software to solve the complex sets of non-linear equations was adopted from the MINPACK software package originally developed in 1980 at the Argonne National Laboratory. This software is now free and in the public domain. Only the most extreme rainfall totals for the critical durations *d* at each daily rainfall stations are used as data. Once the equations are solved, the scale parameter *a* is estimated. Readers who are interested in the complete theoretical and mathematical background are referred to Naghettini (1994, chapter 4). The remainder of this discussion deals with the hydrological and meteorological details of this method.

The method requires that all stations selected have a common period of record that is as long as possible. Daily rainfall totals for all official rainfall gage stations in the United States are available on compact disks from Hydrosphere Corp. (2002). Several stations near the basin being studied need to be selected and their periods of record noted. These rain gage records should represent climate and meteorological conditions similar to conditions in the basin being studied. Stations too far from the study area or too high or low in elevation should not be used. The same continuous period of record should be available for each rain gage selected. It is also advisable to avoid selecting too many stations in any one area, which would then overly weight the climate and rainfall records in that localized area compared to the rest of the surrounding areas for the basin being studied.

The method relies on data from the rainfall gage records that cover the same continuous period of time for each gage. If large gaps in the gage record are found (even though the beginning and ending dates may cover the continuous period needed), the record should be discarded. Recorded rainfall data is subject to many errors, omissions, and other anomalies. Within each rain gage record, missing days, days with accumulated rainfall from several previous days, and days with only a trace of precipitation or other notations are noted. Analyzing the daily rainfall totals involves summing the total rainfalls for the number of days previously defined as the critical duration d for this basin based on analysis of the appropriate stream gage records. The process is complicated by the need to eliminate all the days with missing data or with special notes, such as when the recorded value was already an accumulated value. Any multi-day total rainfall that includes such data is then set to zero and eliminated from further consideration. Trace values are set to zero for the day that they were reported and then they are allowed in the summation process. Any extremely large daily rainfall totals need to be further checked against official hardcopy records, and the correct daily values for these dates are inserted in the analysis if changes are needed. In the process, independence of the rainfall total events also needs to be ensured. The start dates of any two multi-day events must be more than the critical duration *d* apart.

The method requires selection of a number of multi-day total rain events at each gage equal to the number of common years of record for all selected gages. A threshold *d*-day total rainfall for each gage is selected such that exactly the same number of independent *d*-day rain totals is above this value as are in the continuous period of record covered by all the rain gages in the analysis. Further, a reference total precipitation value for each rain gage is also selected such that 90 percent of the previously selected events are below this reference precipitation value. The threshold and reference precipitation values are used later in the statistical analysis. Only the top 10 percent of the *d*-day rainfall totals are used in the regional analysis. This amounts to a form of top-end fitting for the precipitation totals.

To further facilitate the computations, the rainfall multi-day totals are reduced by subtraction of the reference precipitation amount for each rain gage. This step is necessary to eliminate very large numbers in the calculations that follow. This is a form of “indexing” and is common in many regional flood methodologies.

The upper order statistical method calculates the slope (or GRADEX parameter) of the best-fit decaying exponential distribution of the top 10 percent of the *d*-day total indexed precipitation amounts for each selected rain gage site. The selected station elevations and mean annual

precipitations help weight the slope parameter. Knowing the basin's mean elevation and MAP, a GRADEX parameter fit specifically to the drainage basin being studied can be calculated. The result is the slope of the decaying exponential distribution of the most extreme precipitation amounts that the selected precipitation data suggest can occur over the drainage basin. The distribution of *d*-day total index precipitation values can then be used with knowledge of the contributing drainage area for the basin to create associated *d*-day volumes as shown in equation 5, above. This distribution of *d*-day volumes now has a slope, but it must also be fit to the actual reservoir *d*-day inflow volumes at the lower return periods. This is done through a statistical procedure. The fitted curve will match the experienced stream gage *d*-day volumes near the computed reference Q value previously computed. The resulting curve can be extended to very high return periods based on the second basic assumption of the method, that all large flood volumes will occur from rain falling on already thoroughly saturated conditions in the contributing areas of the basin, and any increase in a *d*-day rainfall will result in a corresponding increase in *d*-day inflow volume to the reservoir.

Since the GRADEX parameter *a* is calculated using a maximum likelihood estimate, it is further possible to place a confidence bound on this parameter. Note that for one-parameter distributions such as the exponential, the natural logarithm ratio between the estimated likelihood function and a true likelihood function for the one parameter can be proportional to a chi-square distribution with one degree of freedom. This process is displayed on the Web page, <http://www.weibull.com.LifeDataWeb/likelihood_ratio_confidence *bounds*exp.htm>. By a trial and error process, the upper and lower confidence bounds associated with the *a* parameter estimate can be determined for some set confidence level.

Once the slope and location of the flood volume curve for the *d*-day durations have been established, the question of what is the probability that a particular volume of flooding will be equaled or exceeded in any year can be answered. The more common question is what is the volume of flooding that will be exceeded on average only once in a stated return period, T_c = number of years. To answer that question, the calculated exponential distribution and associated confidence bounds, need to be inversed. The inverse of the distribution has the form:

$$\hat{x}(T_c) = \hat{\beta}_1 + \hat{a}\ln\left(T_c + \hat{\beta}_2\right) \tag{8}$$

Where:

$\hat{x}(T_c)$ is the *d*-day flow value for any required return period, in ft^3/s-days, T_c is the return period (in years) for which a *d*-day flow estimate is required, $\hat{a}$ is the previously estimated GRADEX slope factors converted to volume units, and $\hat{\beta}_1$ and $\hat{\beta}_2$ are constants that can be estimated from a system of simultaneous equations that are formed knowing the mean of the sample of *d*-day flood discharges and the reference *d*-day discharge with an approximate return period. Both of these discharge values and the reference discharge return periods are previously computed from the daily inflow record for the location of the study. The $\hat{x}(T_c)$ value can then be further converted to more common volume units such as acre-feet for a specified number of days.

The original goal of the method, as presented in Naghettini (1994), was to produce a peak-flow frequency curve. In this procedure, a known set of peak flows associated with *d*-day volumes can be determined from the stream or reservoir inflows, assuming peak flows have been recorded. This set of paired data for the period of the streamflow record can be further extended by the use of various rainfall-runoff models. Calibrated rainfall-runoff models can be created for some of the largest events in the stream gage record if appropriate rainfall data are also available.

In the original presentation of the method, it is suggested that several large storms, all of the same type and from meteorologically similar areas, could be transposed into the basin. For each of these large storms, the calibrated rainfall-runoff models can then be rerun with the transposed storm precipitation data and a new peak flow and hydrograph can be generated. Additional sensitivity analysis runs can be made by varying certain parameters in the rainfall-runoff model that affect the peak, such as the lag time or other parameters related to unit hydrograph development. The peaks and *d*-day volumes from all of the additional transposed storms can then be added to the original set of peak and volume data. Regressions on this extended set of peaks and volumes can provide the necessary information to help determine a peak flow for a selected volume at some rare return period that has been calculated by the GRADEX Method. In both the French and American literature for the GRADEX Method, it is suggested that the regression between volume and peak data should not be linear. The French literature states that the ratio of peaks to a *d*-day volume will increase with increasing return periods. A regression procedure known as LOWESS (Locally Weighted Regression and Smoothing of Scatter Plots) (Cleveland, 1979) can be used to perform the non-linear curve fitting required for this procedure.

Reclamation's practice with the method has not involved multiple storm transpositions. For each application, some attempt has been made to create a calibrated rainfall-runoff model using HEC-HMS (U.S. Army Corps of Engineers, 2002). The largest one or two floods from the stream gage record and the best available rainfall data are used to create the calibrated runoff model. The model is calibrated to match as nearly as possible the peak and the entire volume of flooding, which may be longer than the *d*-day critical duration determined earlier. Once the calibrated rainfall-runoff model is completed, the historic rainfall information, with both temporal and spatial distribution, is increased by a constant ratio at each time period. The resulting peak and *d*-day volume of the hydrograph is recorded. Additional runs are made and the lag times are reduced by 10 or 20 percent to account for the fact that the historic flood may not have resulted from such intense rainfall as the desired higher return period floods might produce. With more intense rainfalls, it may be that the basin lag times should be reduced to allow for quicker formation of the flood peaks. The extended peak and *d*-day volume set is then fit with the LOWESS procedure. Using this non-linear regression, peak flows associated with the various volumes for different return periods by the GRADEX Method can be estimated. This method will also allow for production of the entire hydrograph with exactly the required *d*-day volume estimated by the GRADEX Method. Examples of the results of this computation can be seen in the Fresno Dam example at the end of this report.

In the publication, *Small Dams*, (French Committee on Large Dams, undated), an empirical equation is given that will produce an entire hydrograph with a specified peak, time to peak, and the desired time step. One of the parameters in that equation can be varied by trial and error until the desired volume of the hydrograph over any period of time, such as *d*-days, is achieved. This

represents another strictly empirical method to derive a hydrograph once a peak and volume for the desired return period are known.

Some other concerns have become apparent in the application of the GRADEX Method to some dams in the Reclamation inventory. The first concern is with the possible additional volume of flooding that may result from snowmelt that may not be explicitly considered in the GRADEX Method. The available literature indicates that a separate snowmelt volume analysis should be undertaken. For each year of stream gage record, the maximum snowmelt volume for some time period larger than *d*-days should be estimated. A separate LP-III (or any other distribution) analysis of the snowmelt volumes can be constructed and extrapolated to rare return periods. This frequency curve of snowmelt flood volumes can be used with a combined probability analysis of the rain flood GRADEX *d*-day volumes. The resulting frequency curve will display the probability of getting a flood volume composed of both snowmelt and rain flood volumes. What becomes apparent for the large return periods is that the GRADEX rain flood curve will dominate the combined probability volumes. The combined probability curve is almost identical to the GRADEX curve at the large return periods. For a large return period, the probability of getting a flood with X acre-feet composed of Y acre-feet of snowmelt, plus Z acre-feet of rain generated flood volume (X = Y + Z), is nearly identical to getting the rain flood alone with X acre-feet of volume. An example of this type of combined probability analysis is given in the Fresno Dam example later in this report. It is recommended that in all future applications of the GRADEX Method to Reclamation dams some attempt be made to create a separate snowmelt flood volume frequency curve and a combined probability analysis with the GRADEX rain-flood curve be made to ensure that the rain floods dominate at the rare return periods.

A second concern is with drainage area size. The GRADEX Method is based on assumed basin average rainfall. Because of this, there is a clear question as to its applicability to large basins. The original French literature limits the size of the drainage basins where the GRADEX Method can be applied to about 10^4 square kilometers, or about 3,800 square miles. It is noted that few storms with greater aerial coverage exist in the rain gage data. The application of the method to such larger drainage sizes would not produce defendable results. To approach this problem, the suggestion is that the larger basin be broken into smaller parts along logical lines, such as at major tributary confluences, such that each part is no larger than 3,000 square miles. The GRADEX Method could be applied to each separate part, and a combined probability analysis could then be performed with the resulting curves for each part. The resulting frequency curve would show the volume of flooding that could occur resulting from contributions from each separate part of the basin. This approach has not yet been tried for any Reclamation dams.

For the full application of the method for a detailed hydrologic study, additional effort should be made to determine a homogeneous set of rain gages for use in the GRADEX Method. Naghettini (1994) provides some useful suggestions and examples along these lines. Due to time and money constraints, this has not been done in any Reclamation studies to date.

4.4 Australian Rainfall-Runoff Method

The Australian Institution of Engineers developed and published an approach for estimating large to extreme floods in 1999 and revised the method in 2001 (Nathan and Weinmann, 2001). The focus of this work is on estimating floods with very low probabilities of occurrence. The

floods developed using this technique usually have AEPs ranging between 1 in 50 and 1 in 10 million. Uncertainties involved in estimating floods increase with increasing sizes of floods. The following discussion describes Reclamation's experience with estimating rare and extreme floods using the Australian approach.

Three categories of floods are considered – large, rare, and extreme. Large floods typically have probabilities of occurrence ranging from 1 in 50 to 1 in 100. Rare floods include floods with AEPs extending from 1 in 100 to the credible limit of extrapolation, generally around 1 in 2,000. Extreme floods involve estimating floods for the AEPs beyond the limit of credible extrapolation. For risk analysis purposes, rare and extreme floods are of most interest.

Rare floods include events between the largest observed flood and the credible limit of extrapolation. The creditable limit of extrapolation depends on the type and amount of data used for flood frequency analysis. Generally, regional flood and precipitation data, and the inclusion of paleoflood data, allow extrapolation out to around 1 in 2,000 or 1 in 5,000. It is important to note that floods in this category contain considerable uncertainty because estimates are outside the range of observations.

Extreme floods extend beyond the credible limit of extrapolation from the data to AEPs out to 1 in 10 million. Estimating these floods requires prescriptive measures, which do not allow the hydrologist to quantify the uncertainty of the estimates even though it is known to be very large. Extreme floods determined by these methods are intended to be consistent and as reasonable as possible given the state of current knowledge.

4.4.1 Approach

The procedures involved in the Australian Rainfall-Runoff Method are based on flood frequency analysis and rainfall-runoff modeling. Any of the flood frequency analysis techniques previously discussed in previous sections of this report are applicable to the Australian Rainfall-Runoff Method. The unique concept in this approach is the use of "AEP-neutral" parameters in the rainfall-runoff modeling process. This involves selecting model parameters such that the AEP of the 1 in Y rainfall amount produces a flood with a 1 in Y AEP.

Reclamation has used an event-based deterministic rainfall-runoff model to convert a 1 in Y AEP design rainfall into a 1 in Y AEP flood. A single set of hydrometeorological parameters and watershed characteristics are used to produce a flood event. No soil moisture or surface storage recovery is provided. Therefore, the deterministic model always produces the same output.

The major inputs to the deterministic rainfall-runoff model are: (1) precipitation (rainfall and snowfall), (2) losses (infiltration/interception), (3) physical watershed characteristics for runoff and routing simulations (drainage areas, watershed and channel slopes, lag times, antecedent moisture, etc.), (4) precipitation-runoff transformation function, and (5) runoff conveyance and routing mechanisms. Model output includes runoff hydrographs at user-specified locations, maximum peak discharges, and total runoff volumes.

Deterministic event-based precipitation-runoff modeling applies design rainfall distributions and volumes to watersheds for which runoff response is characterized by unit hydrographs and

generalized loss-rate functions. Calculations proceed from upstream to downstream in the watershed. Subbasin hydrographs are routed and combined at the points of interest.
A design storm (rainfall and basin snow cover) is the primary model input. Typically, a time series of basin-average rainfall for a preselected duration and frequency is input to the model. A 1-hour to 72-hour duration storm event is typically simulated. Appropriate duration storm events should be derived from local and regional rainfall records. In the process of developing extreme floods, the Australian Rainfall-Runoff Method assigns an AEP to the PMP, and is solely a function of drainage area size. Reclamation assigns an AEP to the PMP on an as-needed basis and does not endorse the drainage area relationship used by the Australians. Reclamation considers the proximity to moisture sources, areal coverage of the storm, and other factors in assigning an AEP to the PMP.

Excess precipitation is estimated by subtracting losses typically due to infiltration and interception. A variety of infiltration models are available and range from constant uniform loss rates to approximate theory-based functions (Green and Ampt, Philips equation). Antecedent storm assumptions can have a severe impact on basin infiltration estimates. Since the deterministic rainfall-runoff model is based on a single event, soil moisture storage and recovery during and between storms is not considered.

The amount of watershed information required is a function of the type of precipitation-runoff model used. Two classes of models are currently used—lumped and distributed parameter models. Lumped parameter models consider the system as being spatially averaged. In contrast, a distributed system considers hydrologic processes at various points in space and defines model variables as functions of the space dimensions. Some lumped parameter models that are widely in use are HEC-1 (HEC, 1990), FHAR (Bureau of Reclamation, 1990), and RORB (Laurenson and Mein, 1995). Some distributed models that can handle single events include DR3M (Dawdy et. al, 1978), PRMS (Leavesley et. al, 1983), HEC-HMS, and WMS. Additional surface water models are discussed in DeVries and Hromadka (1993).

Transformation of rainfall excess to a direct runoff hydrograph is completed via a convolution integral using (1) unit hydrograph techniques or (2) kinematic wave routing for overland flow. The unit hydrograph has been used extensively for flood runoff estimation. Kinematic wave and distributed modeling approaches may be more appropriate for modeling non-linear systems. A good discussion about rainfall-runoff processes and floods, including practical issues comparing design floods and actual storms, is presented in Pilgrim and Cordery (1993). Beard (1990) presents a methodology for simulating floods of a given probability from hypothetical design storms derived from point rainfall.

Streamflow routing may be classified as either lumped/hydrologic (linear reservoirs, level-pool, Muskingum, etc.) or distributed/hydraulic (diffusive wave, kinematic wave, etc.). In lumped flow routing, streamflows are computed as a function of time at one location; however, in distributed flow routing, streamflows are computed as a function of time at several locations along the stream. Most precipitation-runoff models have adequate routing mechanisms. A detailed discussion of routing options is presented in Chow et al. (1988).

For risk-based dam safety studies, it is necessary to adopt an AEP-neutral approach, where the objective is to derive a 1 in Y AEP flood with an AEP equivalent to its 1 in Y rainfall. The

factors that influence the transfer between rainfall and runoff can be characterized by probability distributions. Thus, ideally, the design hydrograph should be determined by considering the joint probabilities of all the input factors. Stochastic methods are ideally suited to the AEP-neutral objective because they accommodate the observed variability of the inputs while still preserving the interdependencies between parameters. However, for the least important parameters, it may be appropriate to adopt a single representative value instead of the full distribution. Since the relationship between rainfall and runoff is non-linear, it is important to note that adoption of a single representative value for the major inputs will introduce bias into the rainfall-runoff transformation. Therefore, more important model inputs may require use of a joint probability approach.

The simplest approach to deriving AEP-neutral inputs is to use the correlation relationship between the two variables. For example, if it is necessary to derive a temporal relationship to use with the design rainfall magnitude, an appropriate relationship may be derived from the correlation between the largest observed storms and their temporal characteristics during the largest storms on record. When applying relationships based on a limited historical sample to large flood events, the inputs should be conditioned by physical reasoning. For instance, large snowmelt events may require large snowpacks and high temperatures, but the meteorological conditions required to sustain an extreme rainfall event may preclude the joint occurrence of extreme wind speeds. The concurrent wind speeds used in the transformation of snow into runoff must be bounded by a reasonable upper limit.

The selection loss parameters are required inputs common to all event-based rainfall-runoff models. With loss rates, there is evidence to suggest that loss rates are independent of flood magnitude for design floods up to 1 in 100 AEP, though, for more extreme events, it is possible that the loss rates depend on both the AEP and the duration of the design rainfall. When considering snowmelt design floods, it may be necessary to vary loss rates with snowpack extent (Nathan and Bowles, 1997).

The most appropriate approach required to achieve AEP-neutrality depends on the complexity of the system being modelled, the nature of the available data, and the requirements of the flood model. In many cases, it may be expedient to adopt model input parameters derived using regional data, and it will be necessary to supplement empirical evidence by physical reasoning. Calibration of the design flood estimates to flood frequency quantiles will help reduce the uncertainty in extreme flood estimates.

4.4.2 Calibration

Calibration of a flood event model for application to design flood estimation is traditionally restricted to the selection of model parameters to achieve a fit between observed and estimated hydrographs. Attention is focused on collecting streamflow and rainfall data corresponding to the largest events on record. Considerable effort is required to ensure that the temporal and spatial distribution of the rainfall data is representative of the actual event. The ability of a model to reproduce historic events certainly gives some confidence to the validity of subsequent flood estimates. However, the available historic information for floods is usually much smaller than the extreme floods of interest. In most watersheds, the AEPs of the calibration floods are likely to range between 1 in 10 and 1 in 25. While it would be expected that floods of this

magnitude would activate some floodplain storage, the non-linear nature of the out-of-bank flood response is such that the streamflow routing characteristics of larger events may be considerably different. Therefore, while calibration of the model provides valuable information on the flood routing parameters for small floods, caution is needed when using the model to estimate extreme floods of much larger magnitude.

Calibration of rainfall-runoff model results to flood frequency quantiles can provide important information on flood response characteristics for extreme flood events. With this approach, rainfall data are prepared for a specified AEP and then used with a given set of model parameters and input assumptions to derive a flood hydrograph. The peak (or volume) of the flood hydrograph can then be compared to the corresponding quantile obtained from flood frequency analyses. The model inputs associated with the greatest uncertainty can be varied within appropriate limits to ensure agreement between the selected flood quantile. It is recommended that model calibration be undertaken for a range of exceedance probabilities to ensure a consistent variation of parameters with flood magnitude. The approach is suited to ungaged watersheds using regional flood frequency methods as well as sites with limited information. The approach is considered to be particularly useful when combined with flood frequency information that uses paleoflood data.

4.4.3 Strengths and Limitations

Event-based deterministic rainfall-runoff modeling has a long track record in the engineering and hydrologic community and is a proven technique for generating design hydrographs. Reclamation uses deterministic precipitation-runoff modeling. From a technical standpoint, the approach is flexible and requires less effort than most of the more complex approaches. Model choice should be a function of the hydrometeorological and physical data available. For example, a distributed model could be applied in cases where detailed information was available; however, a simplified lumped-parameter model would be appropriate in less data-rich locations. Output from either model would be similar.

Limitations to this method arise from the need to calibrate model results to known historical flood events or to flood frequency analyses so that the AEP of the storm is the same as the resulting flood. Calibration can be difficult if the model is sensitive to many input parameters. A lack of good meteorological data can prove troublesome in developing design storms with appropriate temporal and special characteristics. Rainfall-runoff modeling in data-sparse locations may require a high level of regional data gathering and analyses to obtain the necessary hydrometeorological inputs.

4.5 Stochastic Event-Based Precipitation Runoff Modeling with the SEFM

The SEFM has been developed by MGS Engineering Consultants, Inc., in conjunction with Reclamation personnel. The SEFM was developed for analysis of extreme floods resulting from 72-hour general storms and to provide magnitude-frequency estimates for flood peak discharge, runoff volume, and maximum reservoir levels for use in hydrologic risk assessments at dams

(Schaefer and Barker, 2002). The SEFM is fully described in MGS Engineering Consultants, Inc. (March 2001) and summarized in Schaefer and Barker (2002).

The basic concept of the SEFM is to employ a deterministic flood computation model and treat the input parameters as variables instead of fixed values. Monte Carlo sampling procedures are used to allow the hydrometeorological input parameters to vary in accordance with those observed in nature, while preserving the natural dependencies that exist between some climatic and hydrologic parameters. Example outputs from the SEFM are shown in figures 4-6, 4-7, and 4-8 (MGS, March 2001).

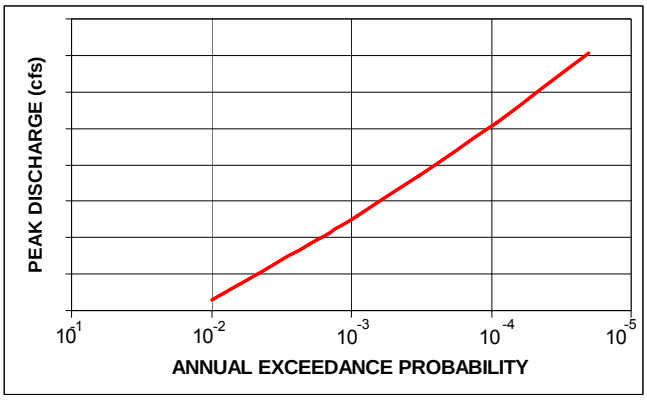

Figure 4-6.—Example of magnitude-frequency curve for peak discharge.

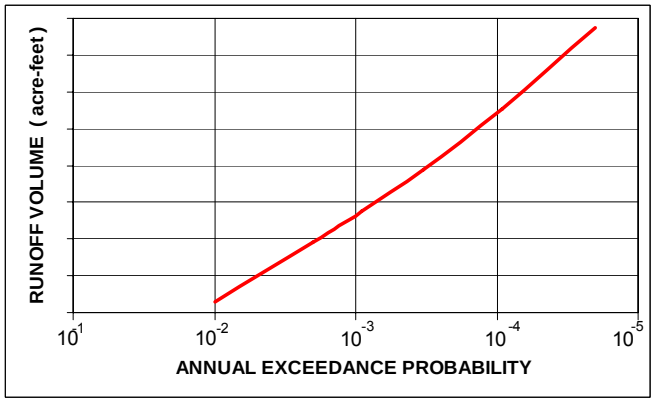

Figure 4-7.—Example of magnitude-frequency curve for runoff volume.

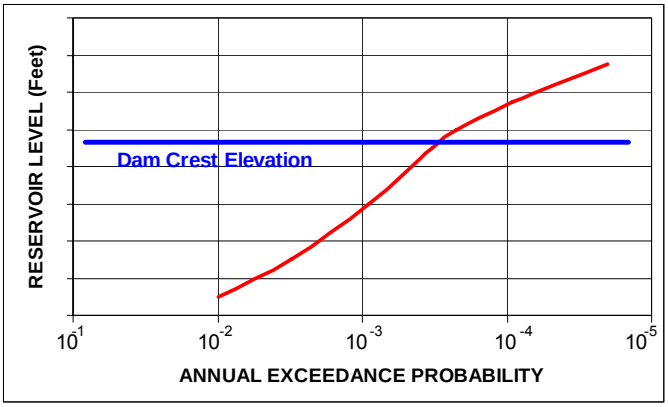

Figure 4-8.—Example of magnitude-frequency curve for maximum reservoir level.

A general flowchart for stochastic modeling with the SEFM is shown in figure 4-9. Thousands of computer simulations are conducted where each simulation contains a set of input parameters that were selected based on the historical record and, collectively, the simulations preserve the dependencies between parameters. The simulated floods constitute elements of an annual maxima flood series that can be analyzed by standard flood-frequency methods. The resultant flood magnitude-frequency estimates reflect the likelihood of occurrence of the various combinations of hydrometeorological factors that affect flood magnitude. The use of the stochastic approach allows the development of separate magnitude-frequency curves for flood peak discharge, flood runoff volume, and maximum reservoir level. Frequency information about maximum reservoir levels is particularly important for use in hydrologic risk assessments because it accounts for flood peak discharge, runoff volume, hydrograph shape, initial reservoir level, and reservoir operations. The precipitation estimates used in the SEFM are typically developed using regional precipitation frequency analysis with L-Moments (Hosking and Wallis, 1997). This approach uses a space-time substitution principle and assumes that there is sufficient precipitation data in a region to combine for subsequent model extrapolation to the probabilities of interest.

The general storm SEFM is composed of seven software components: data entry, input data pre-processor, multiple sample parameter test workbook, HEC-1 template file, stochastic inputs generator, HEC-1 rainfall-runoff flood computation model, and an output data post-processor. The flowchart shown below (figure 4-10) depicts the sequence of actions required for conducting the computer simulations using the software components. Each of these components is described in the SEFM Technical Support Manual (MGS, March 2001). The software (MGS, April 2001) helps the user input data, run the model, and visualize results.

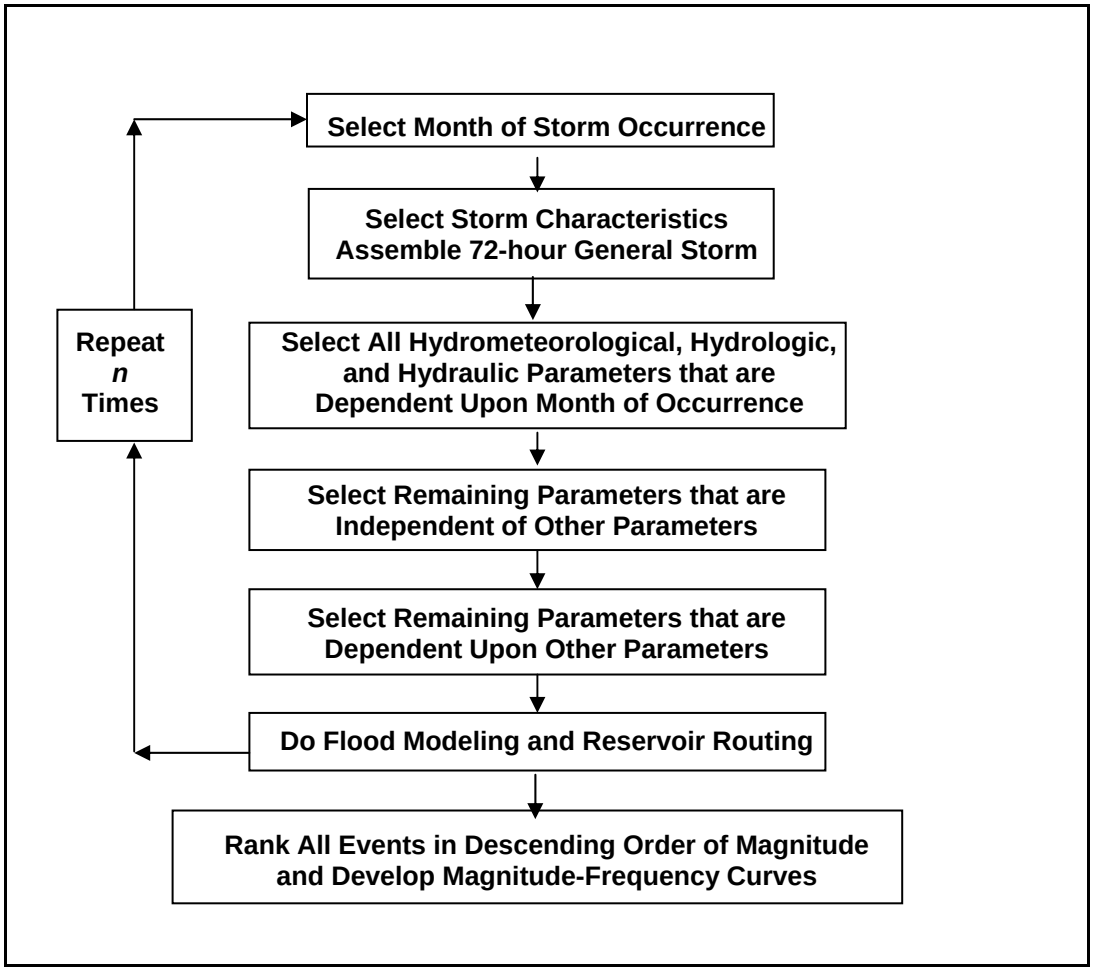

Figure 4-9.—General flowchart for the SEFM simulations (after MGS, 2001).

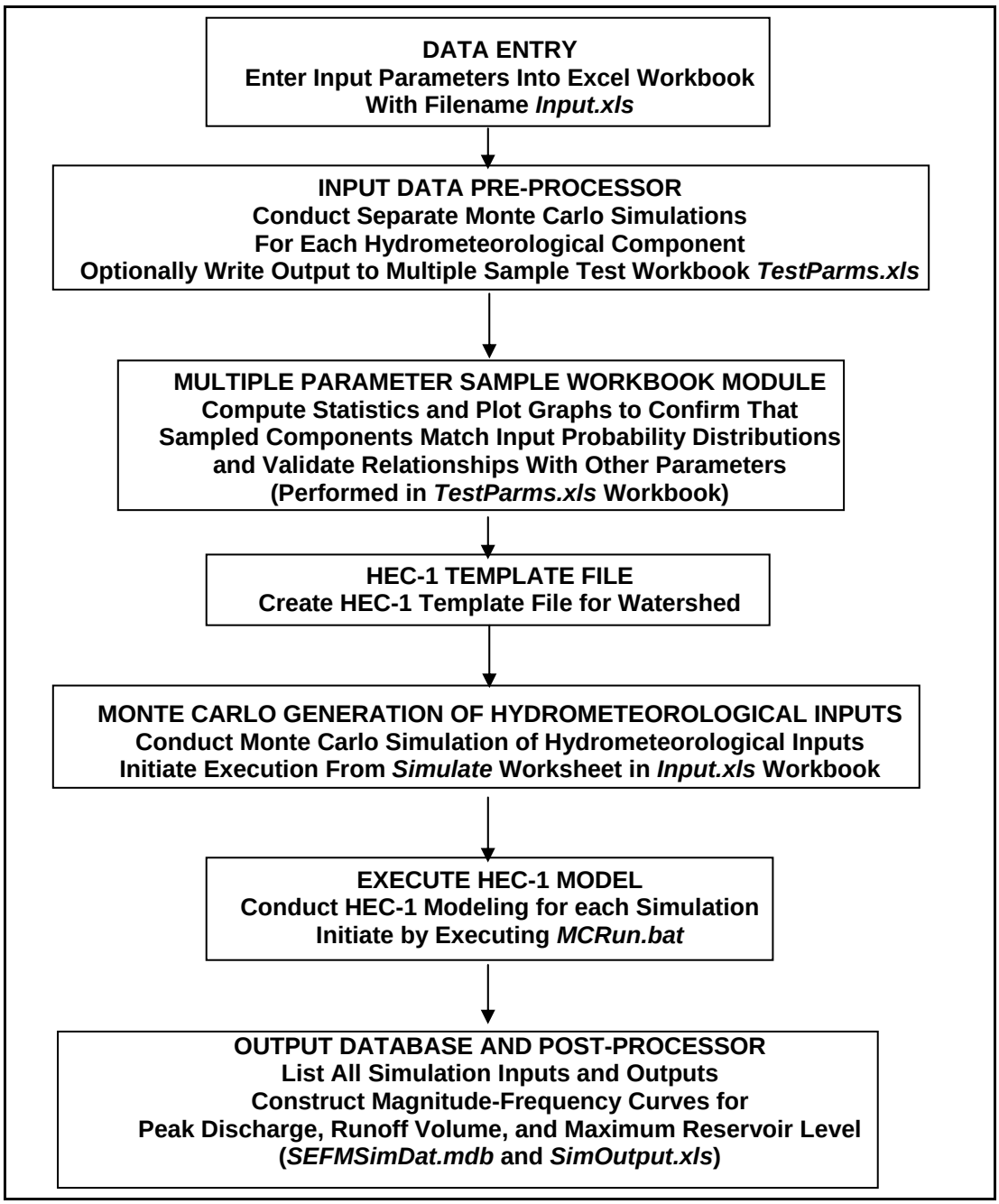

Figure 4-10.—General flowchart for the sequence of actions required for conducting the computer simulations using the SEFM.

The input processor to the SEFM is an Excel workbook. The workbook includes different spreadsheet screens for input (figure 4-11), for model execution, and for output (figure 4-12). The improvements are contained in software version 1.8 of the SEFM.

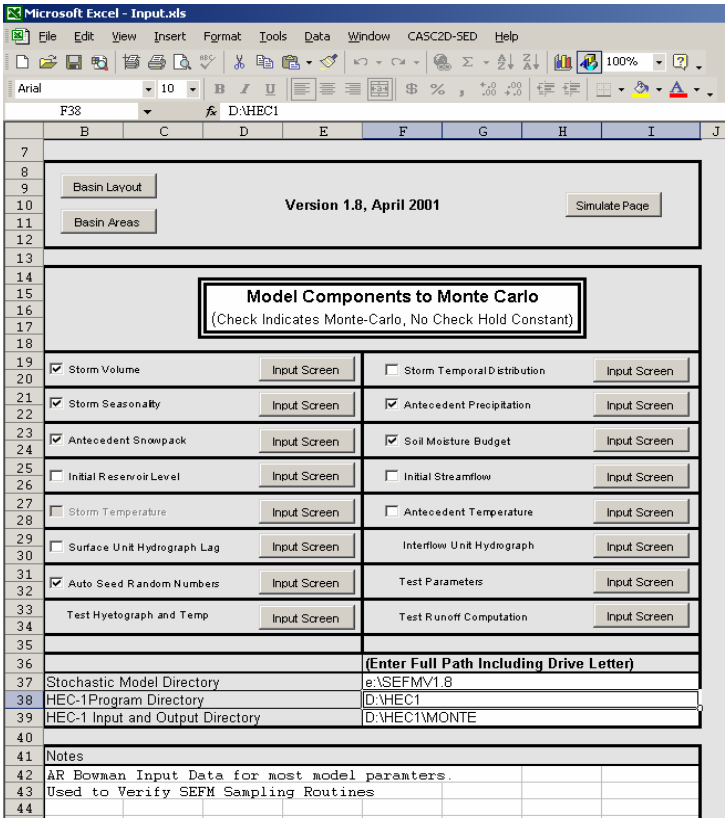

Figure 4-11.—The SEFM input screen.

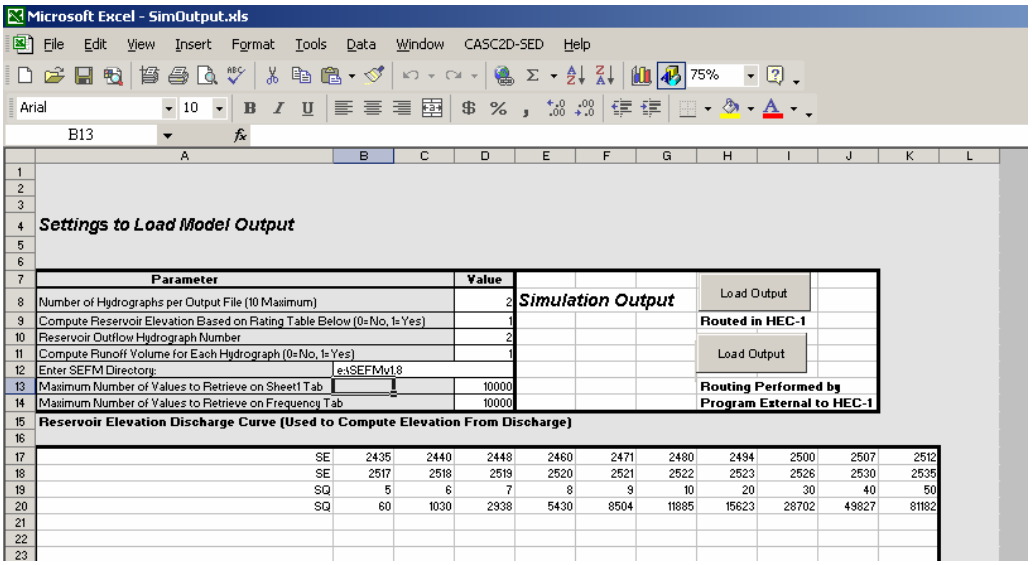

Figure 4-12.—The SEFM output simulation options.

The basic concept behind the SEFM was first used to explore PMF variability and extreme flood probability issues at Bumping Lake Dam for Reclamation (Barker et al., 1996; 1997). The prototype of the SEFM was developed for application to the A.R. Bowman watershed (Schaefer and Barker, 1997; 1998). The SEFM was subsequently generalized for use by Reclamation on other projects and applied at Keechelus and Cle Elum Dams (Bullard and Schaefer, 1999). A sensitivity analysis has been performed for A.R. Bowman Dam (MGS, January 2001) to determine the dominant features that affect model results. A brief review of the model (Singh, 1999) indicated that the model is a sound package for extreme flood modeling, and there could be some eventual improvements.

There are some current limitations to the SEFM and the applicability to certain Reclamation sites. The SEFM is currently configured for simulation of 72-hour general storms. There is no computational limit to the size of the watershed to which it can be applied. However, implicit in the development of the model is the condition that some hydrometeorological parameters are highly correlated spatially. As the watershed size increases, the requirement for high spatial correlation of multi-month precipitation and snowpack becomes more difficult to satisfy. This consideration suggests that the stochastic model is applicable to watersheds up to a nominal size of about 500 mi^2. For larger watersheds, the spatial variability of some hydrometeorological parameters may warrant that site-specific modules be developed to address the site-specific characteristics of the watershed under study (Schaefer and Barker, 2002, p. 732). Currently, the model does not handle thunderstorm events. Additional routines would need to be added to simulate any storms with durations that differ from 72 hours. These limitations may be relaxed as improvements are made to the model.

4.6 Stochastic Rainfall-Runoff Modeling With CASC2D

CASC2D is a fully unsteady, physically based, distributed-parameter, raster (square-grid), two-dimensional, infiltration-excess (Hortonian) hydrologic model for simulating the runoff response of a watershed subject to an input rainfall field for a particular storm event (Julien and Saghafian, 1991; Julien et al., 1995; Ogden and Julien, 2002). Major components of the model include rainfall interception, infiltration, surface and channel runoff routing using the diffusive wave method, soil erosion, and sediment transport. CASC2D is appropriate for simulating extreme floods and physically based extrapolations of frequency relationships combined with a derived distribution approach. The main differences between CASC2D and the SEFM are that CASC2D is a fully distributed model and uses hydraulic principles for runoff generation and routing precipitation excess. The SEFM is essentially a lumped model and uses the unit hydrograph as the basis for runoff and routing precipitation excess. Other differences are the infiltration models and routing mechanisms for river channels. CASC2D is also a somewhat experimental model that has not been used in extreme flood applications for dam safety, or for many applications outside academic research.

The idea and basis to use CASC2D for extreme flood modeling and prediction is centered on two concepts: (1) a derived distribution approach (e.g., Eagleson, 1972) can be used to estimate the extreme flood peak and volume probability distributions and (2) physically-based methods for flood runoff and routing provide a suitable and improved physical basis for the extrapolations of derived flood probability distributions. Ramirez (2000) summarizes the theory behind the

derived distribution approach. In the disciplines of science and engineering, relationships that predict the value of a dependent variable in terms of one or many basic (independent) variables are commonly developed. Physical systems are naturally complex. The functional form of the relationship between independent and dependent quantities or the values of the independent variables (or both) is not usually known with certainty. Techniques based on probabilistic assumptions can be used to account for this uncertainty. When the uncertainty derives from uncertainty in the independent variables, but not from uncertainty in the functional dependence, a derived distribution approach leads to the probability density function of the dependent variable. In this case, the functional form relating independent and dependent variables is assumed known with certainty. In such instances, it is possible to derive the probability density function of the dependent variable(s) from that of the independent variable(s) (Ang and Tang, 1975).

Several research applications use the derived distribution approach to estimate flood frequency curves; these show much promise. The pioneering study for flood frequency is Eagleson (1972). Bras (1990) discusses some of the potential applications of derived distributions in hydrology.

4.7 PMF Analysis Technique

The PMF is defined by Reclamation as “the maximum runoff condition resulting from the most severe combination of hydrologic and meteorologic conditions that are considered reasonably possible for the drainage basin under study” (Cudworth, 1989). Other agencies have developed somewhat different definitions, but all consider the PMF to be a “maximum runoff condition” that is “reasonably possible.”

In Reclamation practice, the basic model to convert PMP to runoff is the unit hydrograph. It is recognized that many other techniques, including sophisticated computerized models, are available for making this conversion. The unit hydrograph concept represents the modeling of the rainfall-runoff process as a linear system. The fact that the rainfall-runoff process is actually non-linear is one of the acknowledged shortcomings of the concept. However, if properly applied, the concept provides entirely satisfactory results for developing flood hydrographs resulting from extreme rainfall events.

During the period from the mid-1940s to the late 1990s Reclamation engineers designed and built most of the large storage dams currently in Reclamation’s inventory. During this period, Reclamation published three editions of the *Design of Small Dams*. That publication has served as a textbook and as a technical guide for numerous States and many foreign countries. Many States, consultants, and other agencies still use the methods and PMF philosophies expressed in the recent editions of those publications. The basic PMF methodologies described in those publications have served as the basis for thousands of dams in the U.S. and other countries designed by both Reclamation and non-Reclamation engineers. These publications are still in widespread use. Reclamation engineers often employed somewhat different techniques and nomenclature for deriving design floods, but the intent was nearly always to design for the PMF determined by the then current techniques. Some exceptions were made for small dams and diversion structures where it was believed that failure of the structure caused by overtopping would not produce any loss of life or major economic damages.

The steps involved in deriving the PMF hydrograph for a single basin have been described in many sources. The following steps are modified from the list given by the National Research Council (NRC, 1988). This list briefly summarizes the PMF calculation process currently followed by Reclamation and many other dam building agencies:

1. Divide the drainage area into subbasins, if necessary, and determine the appropriate drainage areas.

2. Derive a runoff model (unit hydrographs for Reclamation studies).

3. Determine the PMP using criteria contained in NOAA Hydrometeorological Report (HMR) series.

4. Arrange the PMP increments into a logical storm rainfall pattern.

5. Estimate for each time interval the losses from rainfall, due to such actions as surface detention and infiltration within the watershed.

6. Deduct the losses from rainfall to estimate rainfall excess values for each time interval.

7. Apply rainfall excess values to the runoff model for each subbasin.

8. Add to the storm runoff hydrograph allowances for stream base flow, runoff from prior storms, etc., to obtain the synthesized flood hydrograph for each subbasin.

9. Route the flood from each subbasin to a point of interest.

10. Compare the computed PMF peak and volume to the applicable envelope curve of peak and volume flows, if available.

11. Route the resulting inflow hydrograph through the reservoir storage, outlets, and spillways to obtain estimates of maximum storage, elevations and discharges, and durations at the dam.

Many factors influence the ultimate magnitude of the PMF hydrograph, but the intensity and duration of the rainfall are the most important. Considerable analysis and discussion of the derivation and application of PMP estimates has taken place in the past. The original definition of PMP dates back to the late 1930s. In 1981, Reclamation, the National Weather Service, and the U.S. Corps of Engineers adopted a mutually acceptable, uniform definition of the widely used term PMP. The PMP, as defined by these three agencies at that time, is “theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year.” PMP must always be termed as an estimate because there is no direct means of computing and evaluating the accuracy of the results. Since the mid-1980s, Reclamation has considered that the series of

HMRs prepared and updated by the National Weather Service provide the best estimates of PMP potential within the limits of each report. Figure 4-13 displays the current coverage of the continental United States by this series of reports.

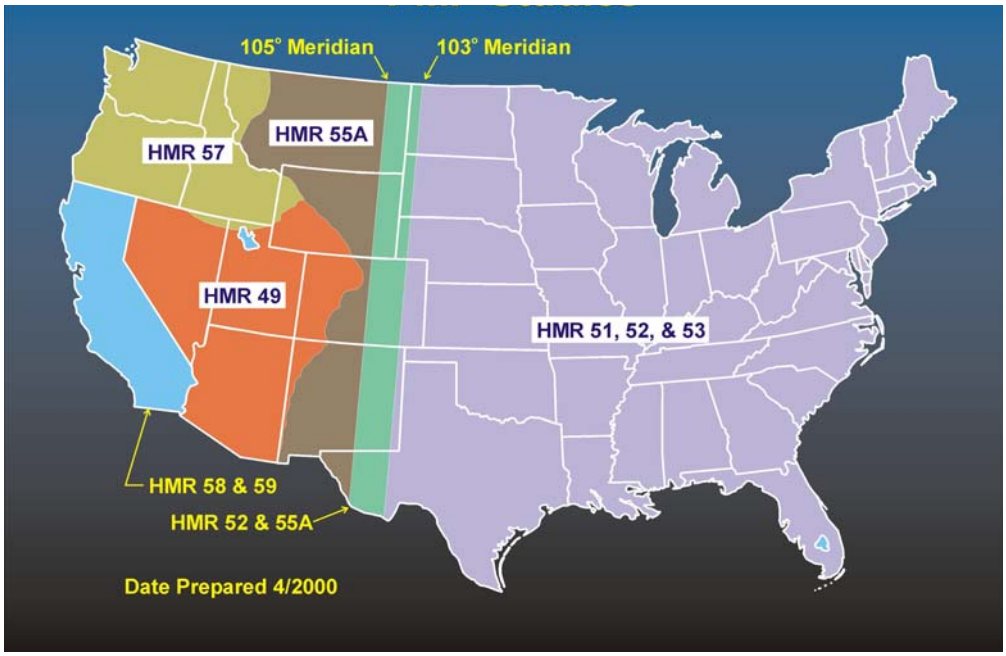

Figure 4-13.—Regions covered by generalized PMP studies.

Other aspects of the PMP to PMF conversion that are unique to Reclamation are discussed in the following paragraphs.

After determining the total PMP depths for specific time intervals from the appropriate HMR, a smooth depth-duration curve is created. This curve is then read at time intervals equal to the desired computation interval for the PMF hydrograph. Each individual precipitation total for each time increment is subtracted from the preceding time increment total PMP. The result is the incremental values of PMP to be used in the computation of the PMF hydrograph. Figure 4-14 displays a typical depth-duration curve from which incremental PMP can be determined.

The temporal rainfall pattern most commonly used with Reclamation PMF studies places the maximum increment of rainfall at the 2/3 point of the storm and arranges the remaining increments of precipitation in descending order about this point. This distribution is applied throughout the United States and results from an examination of individual drainage area regionalized storm criteria combined with various hydrological tests. This arrangement, when combined with the unit hydrograph procedures, will produce a maximum runoff condition for the basin being studied that is still reasonable based on meteorological experience. Figure 4-15 displays this temporal distribution.

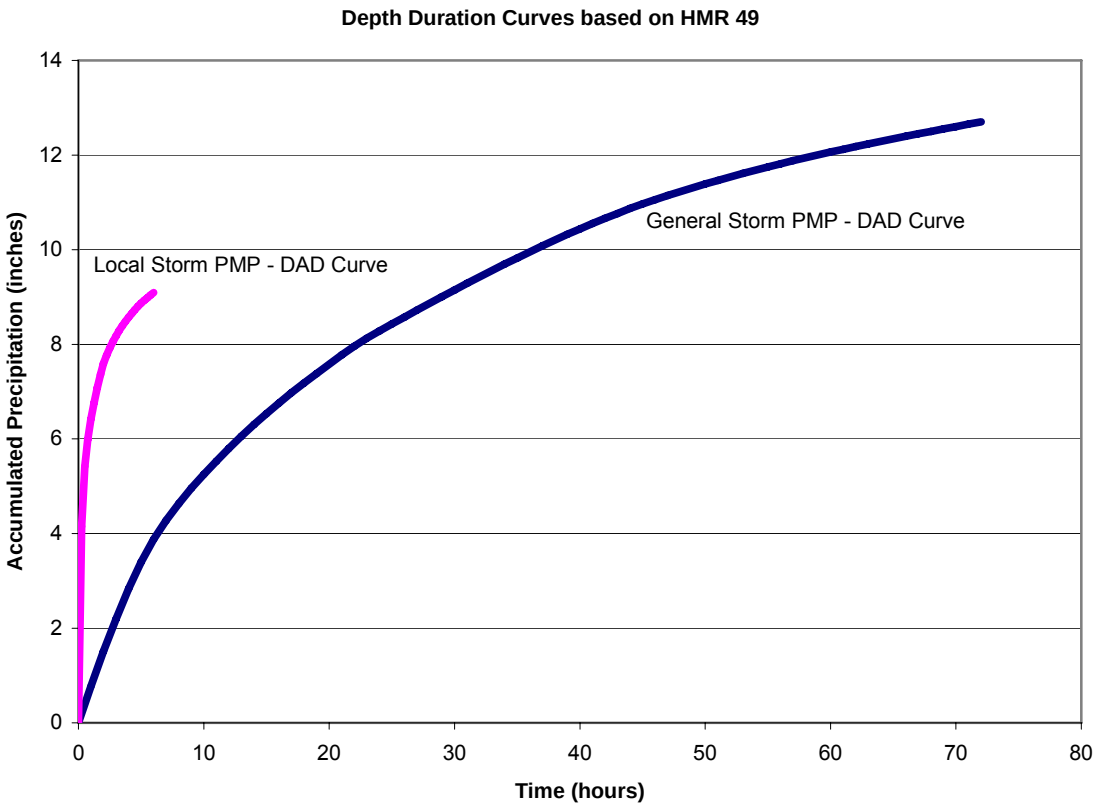

Figure 4-14.—Typical PMP depth duration curves.

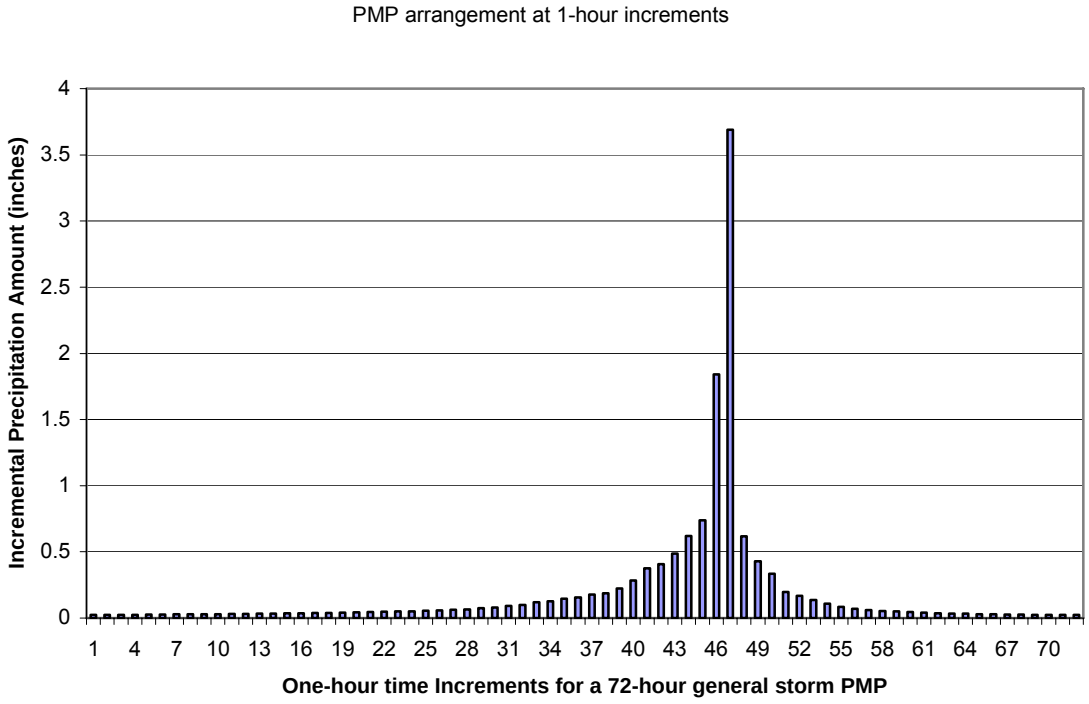

Figure 4-15.—Typical PMP temporal arrangement for a 72-hour storm.

In Reclamation PMF studies for large basins with many subbasins, consideration is given to "centering of the PMF" by allowing the spatial distribution of the PMP to vary across the entire basin. The storm is centered over a single subbasin, and the PMP for that subbasin is calculated along with the PMP for the total basin. PMP amounts over the other subbasins are calculated in a manner that preserves the total basin volume of the PMP but allows for heavier amounts over the subbasins nearest the assumed center of the storm. This calculation is termed "successive subtraction."

Another unique feature of some PMF studies is the use of an elliptical storm pattern as specified in the appropriate HMR. The use of this pattern allows the PMP to vary within a large area in a manner that does not depend on subbasin sizes or shapes but, rather, on historic observations of large storms. This type of PMP calculation is most common for large general storms in the HMR 51-52 area. Local storm PMPs can also use a theoretical elliptical shape in the HMR 49 and HMR 59 areas. For these types of studies, the PMP storm is usually centered over the basin. Volume centerings will center the entire PMP storm over the subbasin nearest to the center of the entire basin. Peak centerings will allow the PMP to be centered over the subbasin nearest to the dam. Volume centerings will typically produce a PMF hydrograph with the largest total volume. Peak centerings will typically produce a PMF with a higher peak but a lower volume. Both PMF hydrographs need to be routed through the dam.

Another good feature of the PMF calculations is that a series of dams all on the same river can be studied. By allowing the PMP to be centered over subbasins above each individual dam, the most severe cause of flooding at that dam can be obtained. Concurrent floods above the other dams on the same river basin can then be calculated for the same PMP event. Once the PMF hydrographs from the upstream dams are formed and routed through those structures, the complete PMF series of hydrographs for the downstream dam can be obtained. This is not always the case with methods based solely on statistical methods.

Local storm hydrographs are also computed for most Reclamation Dams if the HMR series provides specific data for deriving a separate local storm. Such storms are generally considered the thunderstorm type events and are limited in durational and areal coverage. Local storms will generally produce more intense precipitation and are usually most critical for smaller drainage basins. Local storms are considered spring or summer events and may become critical for dams and reservoirs where higher water surfaces are allowed in the spring and summer.

Lag time computations and unit hydrographs for Reclamation PMF studies have come from many past investigations of large floods. Basic unit hydrographs for sites where sufficient data were recorded were derived by standard means after subtracting out any base flows or snowmelt flows. With some knowledge of the basin average rainfall total and duration, the resulting hydrograph could be reduced to a unit hydrograph. The unit hydrographs were then put into a dimensionless form unique to Reclamation, with the flow ordinate being expressed as a dimensionless volume of flooding divided by the amount of flooding that could be expected from 1 inch of excess runoff in 24 hours (expressed as ft^3/s-day/ft^3 or often abbreviated as q). The time ordinate was expressed as a percent of the basin lag time plus 1/2 of the unit duration for that basin. Figure 4-16 is a sample of such a dimensionless graph. Over 60 dimensionless graphs are available to Reclamation engineers, but this number has been reduced to only 6 for publication in the *Flood Hydrology Manual* and in the latest edition of *Design of Small Dams*.

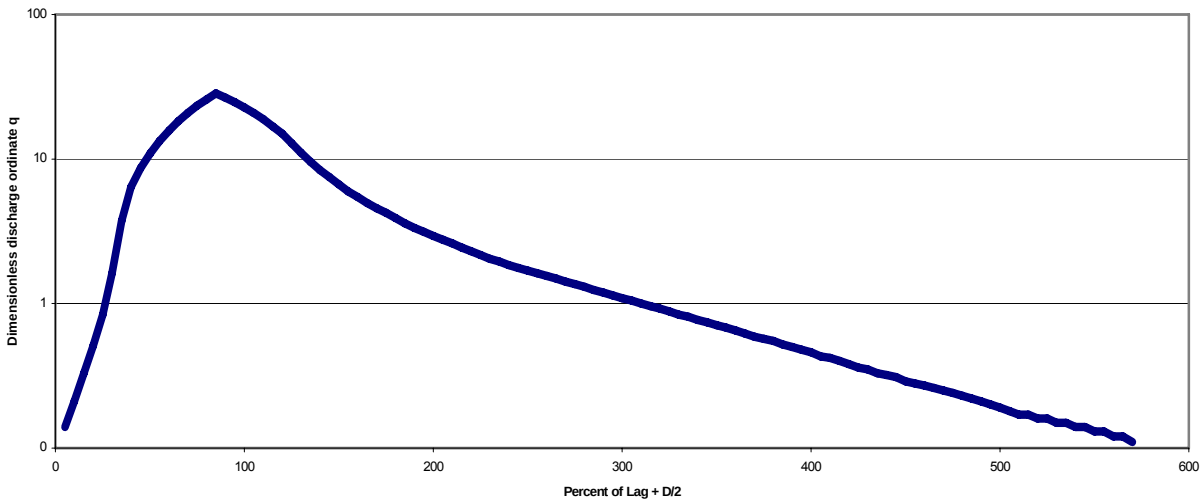

Figure 4-16.—Reclamation dimensionless graph.

The physical definition of the lag time used by Reclamation in PMF studies is the time from the mid-point of a unit of excess rainfall to the mid-point of the total volume of the resulting runoff hydrograph from that single unit of excess rainfall. With this definition in mind, much of the historic flood data collected by Reclamation engineers in the 1940 to 1970 time period was used to define relationships between the physical measurement of the length of the main channel, L (mi); the length to a point on the main channel opposite of the basin centroid, Lca (mi); and the basin slope S (ft/mi). Such data were plotted on log-log graphs, and straight lines on these plots were drawn based on different basin vegetation and soil conditions. Figure 4-17 is a sample of such a plot from the Reclamation *Flood Hydrology Manual*.

The locations of the straight lines on these plots reflect the basin vegetation, soils, overall slope and, to some degree, the basin area and type of PMF being calculated. Such graphs also represent floods from much smaller events than the PMF, and some judgment must be applied to the lag numbers. PMF conditions will usually require somewhat shorter lag times than computed for more common flood events. Lag times for Reclamation PMF studies are based on this type of historical knowledge, reviews of other PMF studies prepared in the area, and reviews of pertinent basin topography, soils, and land use maps. For final design-level PMF studies, field trips to the basin in question by a qualified hydrologist are required to ensure that the information from the maps and other calculated basin parameters is valid. Generalized equations based on the lag curves such as displayed above are available to provide a computational procedure for calculating a lag time for each subbasin and storm type in the basin being studied.

With a computed lag time, drainage area, and dimensionless graph selected, the final unit hydrograph for each subbasin can be calculated. Procedures for this calculation are given in Reclamation's *Flood Hydrology Manual* and in *Design of Small Dams*.

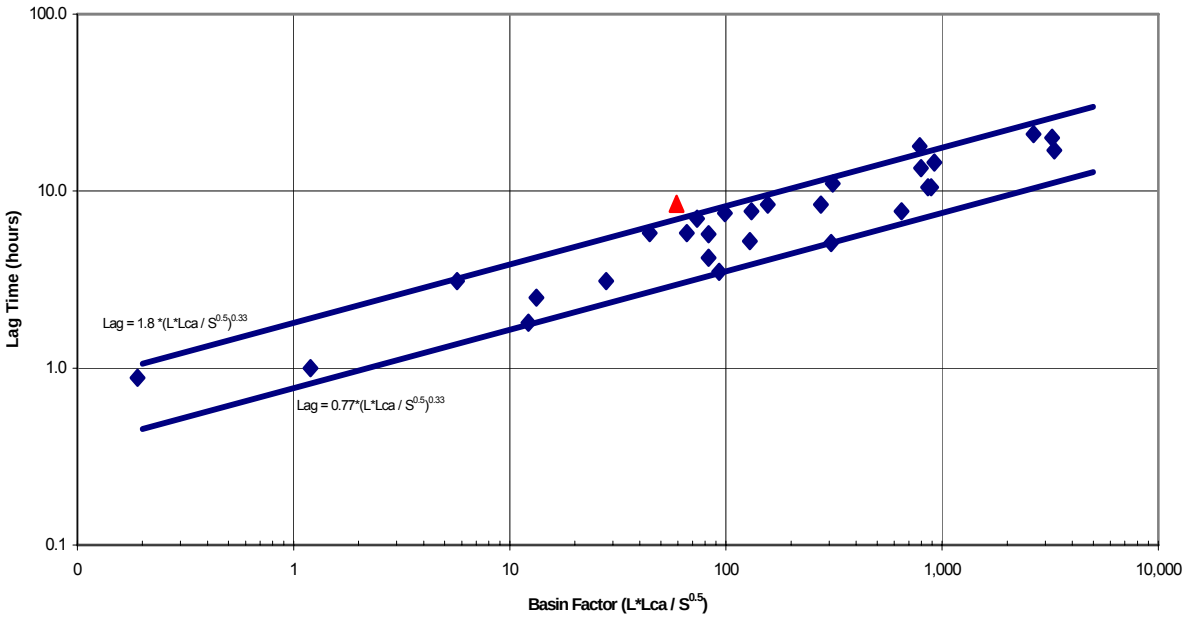

Figure 4-17.—Lag time versus basin factor data for Great Plains Region (from Reclamation *Flood Hydrology Manual*, 1989).

Loss rates for Reclamation studies always assume saturated basin conditions caused by antecedent flooding. Most often, the loss rates are derived by studying basin soils maps available from previous Soil Conservation Service (SCS) soils mapping reports or, more recently, by using the NRCS STATSGO computerized soils database. The SCS divided the soils into four basic hydrologic groups for rainfall-runoff studies. These groups range from very porous sandy or gravely soils with high infiltration rates to very tight clay soils with very low porosity and low loss rates for rainfall-runoff studies. The four soil categories have been assigned ranges of minimum loss rates. By determining the percentage or amount of the different soils in each subbasin, an average minimum loss rate can then be selected or computed from the published minimum loss rate ranges. Initial loss rates are seldom used in Reclamation PMF studies. The definition of the PMF requires the assumption of saturated soil conditions before the onset of the PMP. Under such an assumption, the initial loss rates will be completely satisfied, leaving only a minimum constant loss rate to be considered. It is recognized that more sophisticated loss rate algorithms exist, but for the PMF computation and assumptions, the use of a constant loss rate based on minimum losses for the upper soil layers in the basin is considered adequate.

Antecedent flooding is also considered for general storm PMF hydrograph computations in Reclamation studies. The general storm most often occurs at a time of the year when flooding is most likely to occur in the basin. It is likely that previous storms or a melting snow pack will provide some antecedent streamflow before the onset of the PMP. For basins without significant snowmelt contributions, the antecedent flood comes from a study of applicable streamflow records or, if needed, rainfall-runoff analysis. The desire is to derive a 100-year event by statistical means or by rainfall-runoff modeling if limited stream gage data are available. This derived flood hydrograph is then placed a number of days in front of the onset of the PMP. In

most parts of the central United States, there is a 3-day separation between the peak of the antecedent flood and the start of the PMP. This length of time between the antecedent rain flood hydrograph peak and the start of the PMP will vary near coastal areas. Figure 4-18 displays this type of application.

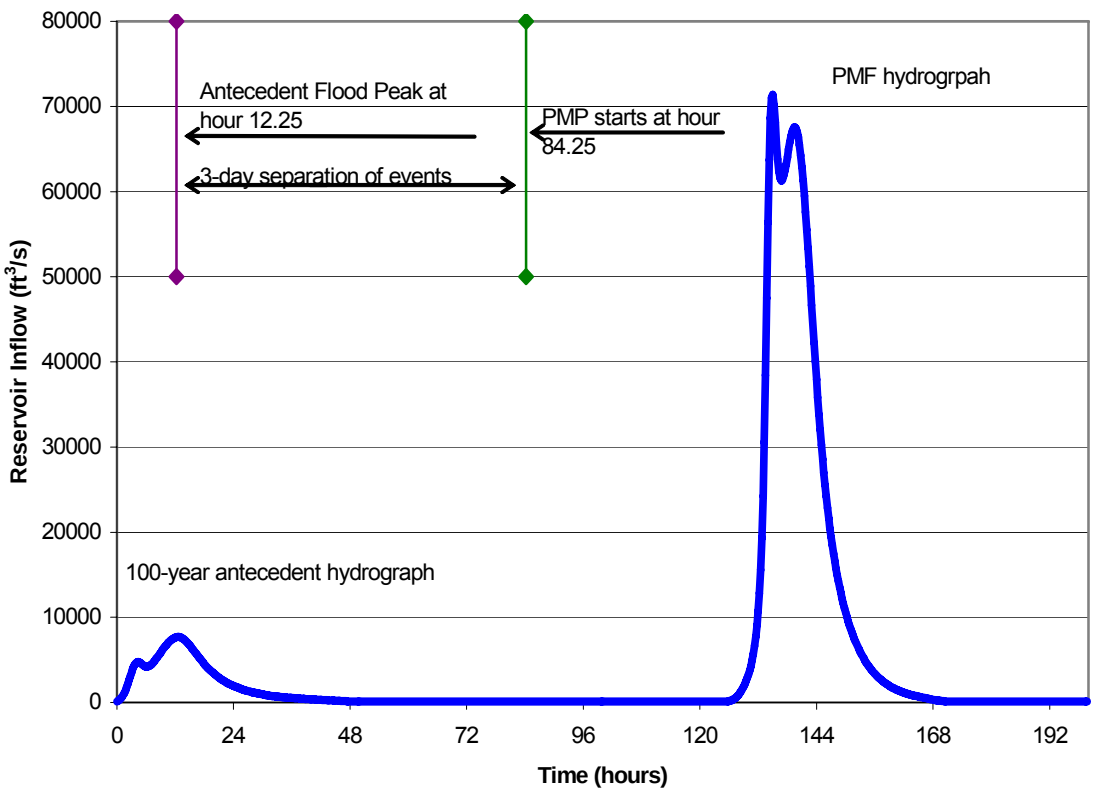

Figure 4-18.—PMF with antecedent 100-year floods.

In areas where snowmelt adds to the volume of the flooding, the antecedent flood in Reclamation PMF studies is often derived from an analysis of stream gage data. The data are limited to the season when snowmelt adds to the flooding. A large historic snowmelt flood may be selected or a statistical analysis of the volume of flooding for several different durations may be undertaken. The result is a 100-year flood volume for a duration longer than the base length of the calculated PMF rain flood hydrograph. If a historic flood event is selected, the daily flows for the hydrograph will be adjusted by a ratio such that the resulting volume for the selected duration is a 100-year volume that can be determined by a statistical analysis of appropriate gage records. The historic flood hydrograph may also be rearranged in time to provide a more normal hydrograph shape. If no historic flood hydrograph is appropriate, then the antecedent snowmelt hydrograph may be derived as a balanced hydrograph. If a snowmelt hydrograph is used as an antecedent flood, Reclamation places that hydrograph under the rain-generated portion of the PMF such that the peaks will exactly coincide. Figure 4-19 displays such a derived historic flood hydrograph for a 15-day antecedent event. Figure 4-20 displays the placement of the derived balanced hydrograph to coincide with the peak of the rain-generated PMF hydrograph for a basin.

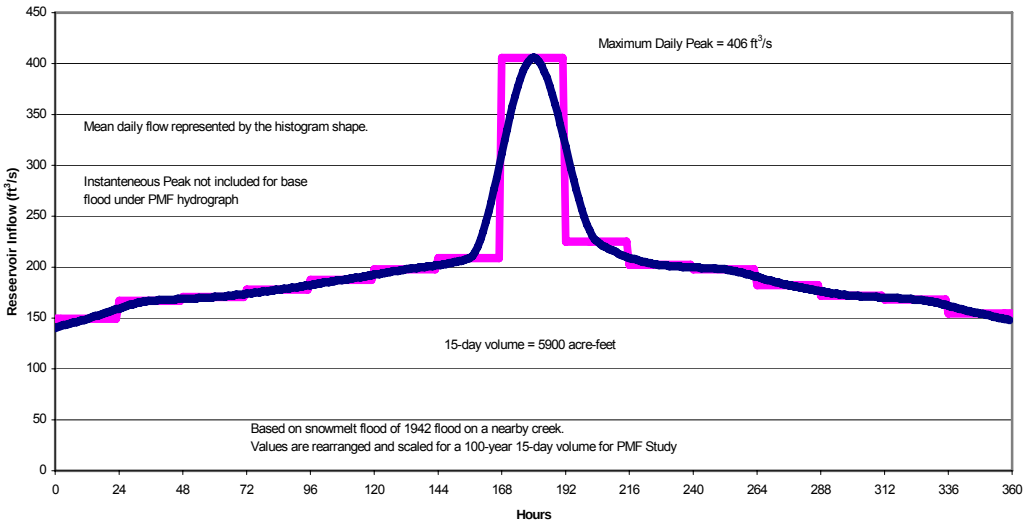

Figure 4-19.—100-year 15-day derived snowmelt flood from an historic snowmelt flood event.

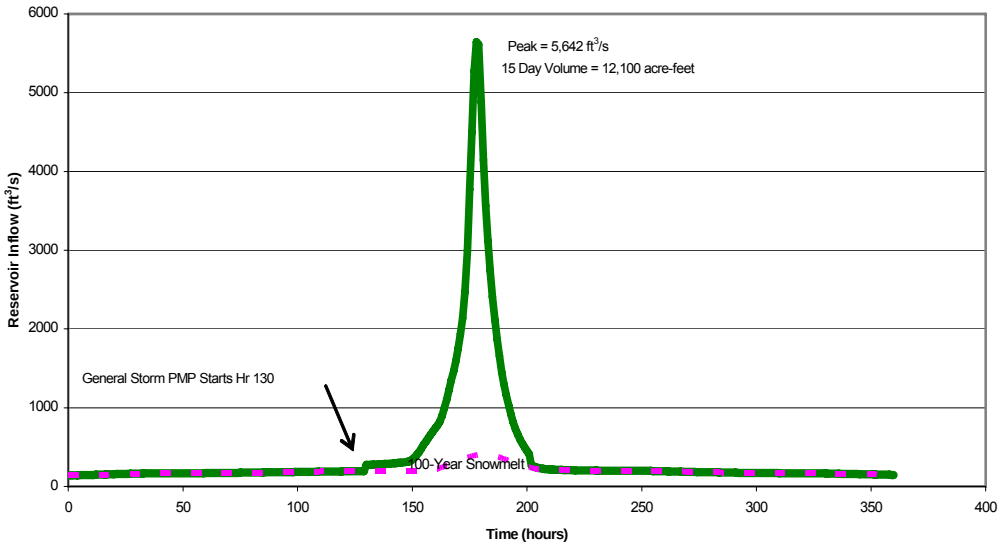

Figure 4-20.—General storm PMF with 100-year snowmelt antecedent flood.

When a snowmelt flood is required, the loss rate for the measured or estimated snow-covered areas in the basin is reduced to a constant 0.05 inch per hour. This is assumed to account for the fact that the 100-year snowmelt is already in progress at the start of the PMP and most of the available constant loss rate will have already been taken up by the melting snow.

In areas where deep snow packs are known to occur, such as the high Sierra Mountains in California, Reclamation will use a snowmelt computation program. This program will account for the effect of rain falling on a snow pack that may not be fully ripened and ready to melt at the start of the PMP. The snowmelt program requires additional meteorology data such as wind and temperature sequences compatible with the PMP derivation. The HMR series will provide good

information in this regard. Additional study of the historic snow gage records in or near the basin being studied is also required. The depth and initial density of the snow pack at various elevations in the basin are used in the computations. Such snowmelt computations provide additional depths of water to be added to the arranged PMP sequence. The combined snowmelt and PMP depths are then used with the unit hydrograph procedures to derive the desired PMF hydrographs.

Routing of floods between subbasins is also required. If sufficient data for several large flood events are available, routing coefficients for the Muskingum Method can be derived. Usually, little information is available for large floods at more than one gage in the basin being studied. In lieu of Muskingum routing, the most common procedure is an analytical procedure referred to as the Tatum Method. This method requires only an assumed travel time between the various subbasins and will attenuate the routed hydrographs by a simple arithmetical computation. This procedure does not affect the volume of the flooding, but it does have some impact on the PMF peak.

Envelope curve comparisons of the PMF peak and sometimes a volume comparison are also made. The comparison is only to ensure the hydrologist that the PMF peak or volume is not below experienced flood levels in the region of concern. If the PMF peak falls below an envelope curve of peak flows for a nearby region, some additional thought must be given to the derivation of the PMF or the derivation of the envelope curve values. There are no rules about how far above the envelope curve a PMF peak should be. This would depend on the derivation of the envelope curve. Envelope curves covering large regions might have additional high-peak flows and the PMF comparison would be closer. Smaller regions would most likely have PMF peaks much higher than the envelope curve.

5. Characterization of Hydrologic Hazards

Since no single approach is capable of providing the needed characterization of extreme floods over the full range of AEPs required for risk assessment, results from several methods and sources of data should be combined to yield a hydrologic hazard curve. The application of several independent methods applicable to the same range of AEPs will increase the credibility and resulting confidence of the results. The previous section of this report describes the available approaches for estimating the hydrologic hazard.

To compute a hydrologic hazard curve for risk assessment, Reclamation makes use of all available studies for the site of interest. Often, the PMF and initial flood frequency studies are the only hydrologic studies available before the start of a probabilistic investigation. If a dam passes the PMF or a flood with an AEP low enough to meet dam safety risk criteria, no further studies may be needed. The intent of the hydrologic hazard analysis is to provide information that will allow decision makers to make appropriate dam safety decisions while minimizing study costs.

The remainder of this section of the report describes Reclamation's approach toward characterizing the hydrologic hazard for a particular site of interest. The process begins with

Flood frequency analysis and hydrograph scaling for the CFR and may progress through more detailed studies if the need arises. If detailed studies are already available for a particular site, these studies would be summarized and would form the basis of the hydrologic hazard characterization.

5.1 Integration of the PMF into Hydrologic Hazard Evaluations

The PMF is recognized as the upper limit of flood potential at a site, for storm durations defined by the Probable Maximum Precipitation (Bureau of Reclamation, 2002b). This means that Reclamation uses the PMF as the upper limit to extreme floods for risk assessment, corrective action decisions, and dam safety modifications. If a dam does not have a hydrologic safety deficiency using the PMF as the hydrologic loading condition, no further hydrologic studies are warranted for evaluation of spillway capacity. If a flood frequency analysis produces peak flows or volumes that exceed the PMF, then the PMF should be used in evaluating the hydrologic risk and as a theoretical and practical upper limit to statistical extrapolations. Before applying the PMF as the upper limit, the hydrologist should ensure that it has been developed using current procedures with up-to-date data and for a PMP duration suitable for the site of interest.

5.2 Characterization of Hydrologic Risk for the CFR

Hydrologic hazard curves and flood hydrographs are developed for use in evaluating and prioritizing the need for dam safety modifications at Reclamation and other U.S. Department of the Interior facilities. These guidelines provide an approach for developing hydrologic hazard information that will allow decision makers to determine appropriate courses of action to assure the safety of the dam while minimizing study costs. If detailed hydrologic studies are available for the site of interest, they should be reviewed for adequacy, then summarized and used for the CFR. If no study is available, a hydrologic hazard curve and flood hydrographs should be constructed using flood frequency analysis and hydrograph scaling procedures. The remainder of this section explains the procedure for developing hydrologic hazard information for the CFR using these procedures.

The characterization of hydrologic risk is first provided in terms of estimated peak discharges, which are then used to estimate flood volumes. Peak discharges are estimated using a mixed-population model, and the results are subsequently input to a linear scaling algorithm of volume critical hydrographs. The hydrologic hazard characterization for the CFR is usually accomplished with minimal effort, about 15 staff days or less.

The mixed-population model used to estimate peak discharges is described by an LP-III distribution for return periods from 1 to 100 years and an LN-2 distribution for return periods greater than 100 years. Historical data are used to calibrate the LP-III model, and an informed hydrologic estimate of a single flood potential point (SFPP) with a return period of greater than 100 years must be made. The fitting of the LP-III model to the historical data is carried out using the Method of Moments (MOM) with which a regional skew may be used to fix or weight the distribution. The fitting of the LN-2 distribution is carried out analytically between the 100-year flood estimate and the SFPP. The value of the SFPP may take the form of a paleoflood non-exceedance bound, a paleoflood estimate, a historical data point with an estimated return period

greater than 100 years, or any other estimate of a flood with a return period greater than 100 years believed to characterize the extreme values of the flood potential. Once the initial data requirements are met, a list of peak-discharge estimates is calculated and the results are provided in tabular as well as graphical format (table 5-1, figure 5-1). The analysis is extended beyond the SFPP to floods with AEPs as small as 10^{-8}, but peaks are not allowed to exceed the PMF for the site of interest.

Table 5-1.—Example peak-discharge estimates in tabular form

Annual exceedance probability	Return period	Peak discharge estimate (ft^3/s)
0.01	100	26,500
0.005	200	31,100
0.002	500	37,700
0.001	1,000	43,200
0.0005	2,000	49,200
0.0002	5,000	57,700
0.0001	10,000	64,800
0.00005	20,000	72,300
0.00002	50,000	83,200
0.00001	100,000	92,000
0.000001	1,000,000	126,000
1E-07	10,000,000	167,800
1E-08	100,000,000	218,800

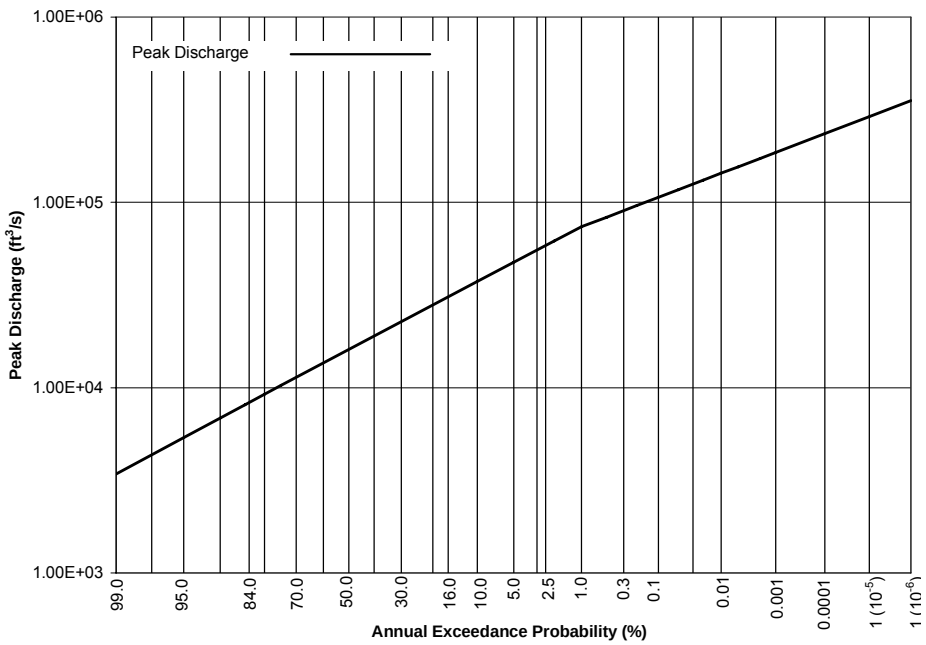

Figure 5-1.—Example of peak-discharge estimates in graphical form.

To make a volume estimate, a time series flood hydrograph estimate is required. The time series flood estimate may be a historical flood hydrograph or a PMF hydrograph; either must be believed to describe the rainfall-runoff response of the basin of interest at a wide range of return periods. Flood hydrographs are linearly scaled so that the peaks match the estimates from the peak-discharge analysis described above. When the hydrograph duration permits, 1-, 3-, 5-, 7-, and 15-day volumes are calculated as the maximum volume of water transported during the desired continuous time period.

When a rain-on-snow PMF hydrograph or rain-on-snow historical hydrograph is used as an input, the analysis technique requires two hydrograph ordinates for each time period – one for the snowmelt portion and one for the rainfall portion. Only the rainfall portion of the hydrograph is scaled to achieve the desired peak. The snowmelt portion of the hydrograph remains intact. For all other input hydrographs, the entire hydrograph is scaled linearly to match peak estimates. When volume estimates are calculated, the results are provided in tabular as well as graphical format (table 5-2, figure 5-2). Graphical results (peak and volume flows) can be provided within a combined chart (figure 5-2) or as two separate charts, as desired by the client (figure 5-1 and figure 5-3).

The major assumption involved with scaling PMF hydrographs to various return periods is that the dimensionless hydrograph used to develop the PMF is appropriate to describe the rainfall-runoff response of the basin at all return periods. Because the generation of a hydrograph from a dimensionless hydrograph involves only the convolution with time distributed rainfall rates, assumptions are further made that the rainfall spatiotemporal distribution does not change with flood magnitude, and soil characteristic response is linear in nature. The latter assumption may be appropriate when floods of interest begin with saturated ground conditions. When these assumptions are violated, predictions may be inaccurate by orders of magnitude. Similar assumptions can be made when scaling a historical hydrograph. These assumptions are mainly that the storm that caused the flood is similar in temporal and spatial characteristics to the storm that would cause a flood of a different return period and that the initial soil conditions are also similar. Soil infiltration responses are known to be highly non-linear in the unsaturated condition; therefore, as with the PMF scaling, it is the initial condition for which the soils are saturated that this type of scaling is most appropriate.

Reclamation uses the PMF as the upper limit of flood potential at a site for storm durations defined by the PMP. If peak flows or volumes calculated using probability or statistical-based hydrology methods exceed those of the PMF, the PMF is used in evaluating the hydrologic risk and as a theoretical and practical upper limit to statistical extrapolations. If the PMF has been properly developed, it represents the upper limit to runoff that can physically occur at a particular site.

Hydrograph ordinates for the peak-scaled hydrographs, produced during the volume analysis, are also provided for the CFR hydrologic hazard evaluation. Ordinates for the scaled hydrographs have the same duration and number of ordinates as hydrograph data provided for input to scaling analysis, thus requiring no interpolation. The scaled hydrographs have an AEP associated with them, so they are suitable for use in reservoir routing studies and for dam safety risk assessment.

Table 5-2.—Example of combined peak-discharge and volume estimates in tabular form

RETURN PERIOD (yrs)	PEAK DISCHARGE (cfs)		1-Day Volume (acre ft)	3-Day Volume (acre ft)	5-Day Volume (acre ft)	7-Day Volume (acre ft)	15-Day Volume (acre ft)
100	73,890		44,300	72,530	84,940	101,130	137,590
200	83,220		49,740	80,900	94,070	109,920	146,730
500	96,140		57,260	92,470	106,690	122,070	159,360
1,000	106,370		63,220	101,640	116,700	131,710	169,380
2,000	117,040		69,440	111,200	127,120	141,740	179,810
5,000	131,840		78,060	124,450	141,590	155,670	194,290
10,000	143,590		84,910	134,980	153,070	166,730	205,790
20,000	155,840		92,040	145,950	165,050	178,260	217,770
50,000	172,830		101,940	161,180	181,650	194,250	234,400
100,000	186,310		109,800	173,250	194,830	206,940	247,590
1,000,000	235,220		138,290	217,070	242,630	252,970	295,450
10,000,000	291,000		170,790	267,040	297,150	305,460	350,020
100,000,000	354,340		207,690	323,780	359,050	365,060	411,990

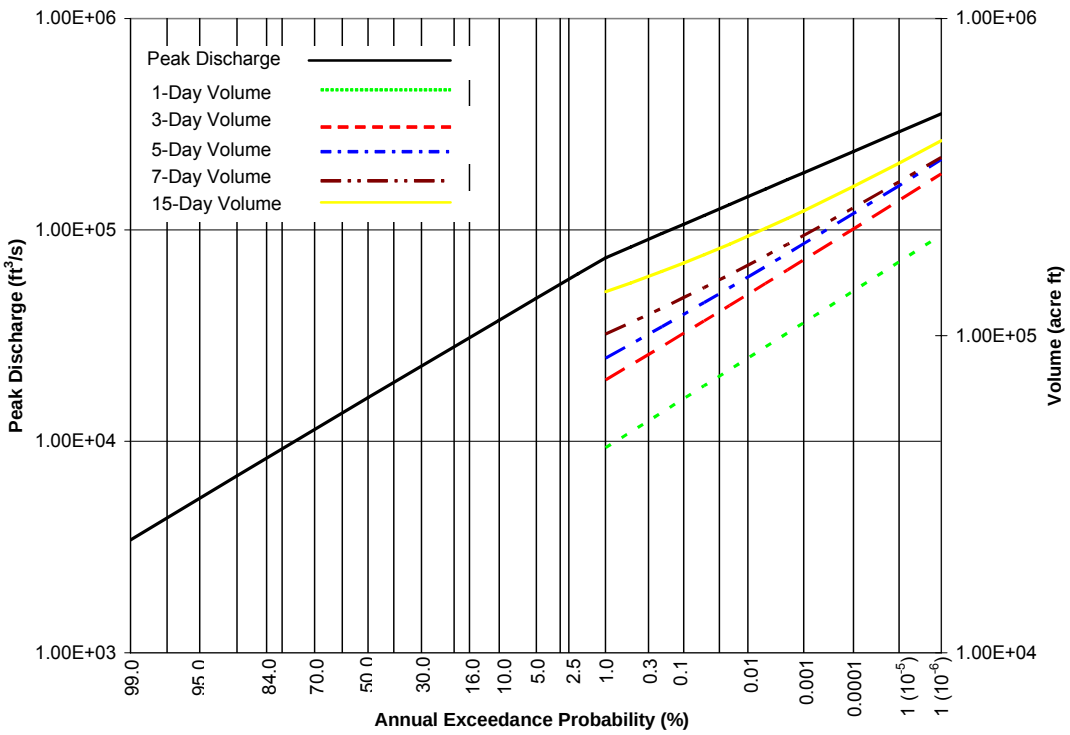

Figure 5-2.—Example of combined peak-discharge and volume estimates in graphical form.

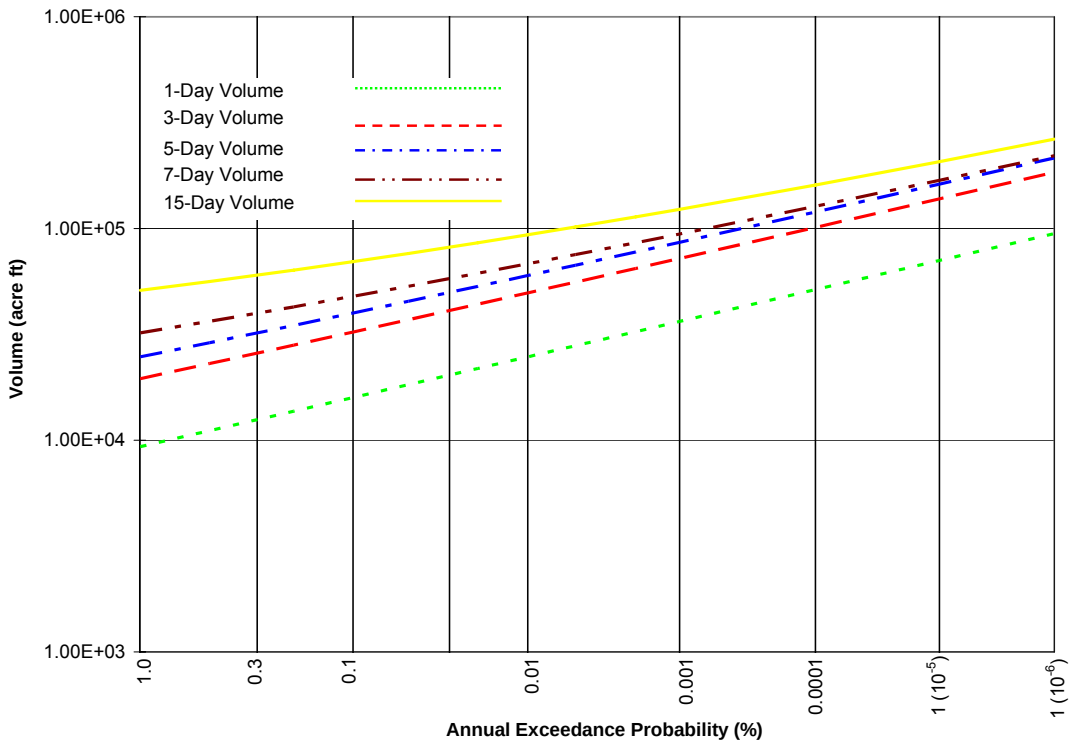

Figure 5-3.—Example of volume only chart.

5.3 Detailed Hydrologic Studies

After developing the hydrologic hazard information for the CFR, more detailed hydrologic studies may be necessary to better define the hydrologic problem, reduce uncertainty, develop solutions, or make decisions. The amount of effort expended on analyzing a hydrologic hazard is dependent on the nature of the problem and potential cost of the solution. Reclamation uses a staged approach toward solving this problem. Initially, very little effort is expended to determine the magnitude of the hydrologic hazard. If additional hydrologic studies are needed, a project plan is developed that tailors the study to the particular needs of the project. The scope of work attempts to address the specific dam safety issue at the least possible cost while providing sufficient information to the dam safety decision makers to make informed safety decisions.

Flood characterization for risk assessments uses the length of the data record and other characteristics of the data to determine the range of credible extrapolation used in the flood frequency analysis. Because Reclamation risk assessments may require estimation of floods with AEPs as small as 1 in 100,000,000, extrapolation of flood frequency relationships is required well beyond the limits warranted by the data. These guidelines provide a reasonable and reliable approach for characterizing the hydrologic hazard beyond the range of extrapolation suggested by the data for use in dam safety risk assessments. Reliable flood frequency estimates are needed for very small AEPs for dam safety decisionmaking, and these estimates should convey an estimate of the uncertainty in the analysis for the consideration of the decision maker.

The uncertainties associated with flood estimates are likely to be substantial and an important attribute to convey into the risk assessment. Flood characterization should include a "best estimate" of the AEP of floods of different magnitudes and a description of the uncertainty in such results. Such uncertainties need to be honestly represented and considered throughout the risk assessment process.

Further analyses might be necessary if the hydrologic hazard information provided during the CFR is insufficient to define and quantify hydrologic dam safety issues and make decisions. Additional studies may become necessary to address reservoir routing effects of upstream dams, reduce uncertainty in the flood estimates, verify statistical results before committing large capital expenditures to dam safety modifications, etc. When planning the next study, the goal is to achieve a balance between the amount of hydrologic analysis needed to address the issues and make decisions, and the level of effort required to conduct the study. Generally, studies progress from those requiring a low level of effort to those requiring a higher level of effort. As the studies get more detailed, the results should become more precise and contain less uncertainty.

Since each study site is different, no single approach can be identified to address all hydrologic issues. The method chosen should consider climatic and hydrologic parameters, drainage area size, amount of upstream regulation, data availability, and level of confidence needed in the results. The previous chapter of this report described the methods available in Reclamation to develop hydrologic hazard curves.

Table 5.3 lists various methodologies that were considered for characterizing extreme floods to support dam safety risk assessment. A flood frequency analysis must be combined with each of these methodologies to assign annual exceedance probabilities to the floods.

Table 5-3.—Alternative methods to develop hydrologic hazard information

Method of Analysis and Modeling	**Risk Analysis/Design Level[1]**	**Level of Effort[2]**
Flood frequency analysis with historical/paleoflood data - Graphical method; - EMA; - FLDFRQ3	CFR, IE, CAS, FD	Low
Hydrograph Scaling and Volumes	CFR, IE, CAS, FD	Low
GRADEX Method	IE, CAS, FD	Moderate
Australian Rainfall-Runoff Method	IE, CAS, FD	Moderate
Stochastic Event-Based Precipitation Runoff Modeling with SEFM	CAS, FD	High
Stochastic Rainfall-Runoff Modeling with CASC2D	CAS, FD	High
Probable Maximum Flood	CFR, IE, CAS, FD	Moderate

Notes:

1. Risk analysis/design level: CFR - Comprehensive Facility Review, IE - Issue Evaluation, CAS - Corrective Action Study, and FD - Final Design.
2. Level of effort: Low requires 10-20 staff-days to complete the analysis; moderate requires 21-75 staff-days; and high requires more than 75 staff-days.

Hydrologic studies usually will proceed from flood frequency and hydrograph scaling to an analysis using either the GRADEX or Australian Rainfall-Runoff Method. The advantages of these approaches are that they use regional rainfall data in developing flood estimates and yield flood hydrographs. Both approaches work best on small drainages (less than 2,000 mi^2), and the Australian Rainfall-Runoff Method works better if snowmelt floods are important.

The SEFM and CASC2D are more sophisticated hydrologic modeling techniques used to address hydrologic issues driven by large potential dam failure consequences. These approaches are more suitable for CAS and final designs. They also allow for better quantification of hydrologic uncertainty. These approaches require a lot of data and are generally more costly than the others.

When multiple methods have been used to determine the hydrologic hazard, sound physical and scientific reasoning for weighting or combining results is needed. Clearly, a measure of judgment is required to ensure that appropriate information is included in the dam safety decisionmaking process. Reclamation develops various weighting schemes to evaluate the results of multiple analyses and uses a team of hydrologists to select the weighting scheme that best characterizes the hydrologic conditions for the site of interest. The selection is based on the experiences of the team members and the assumptions used in each of the analyses. The A.R. Bowman case study that follows illustrates this approach.

6. Case Studies

Three case studies are presented to illustrate the use of several methods. The study sites are Los Banos, Fresno, and A.R. Bowman Dams. Each study begins with a flood frequency analysis and hydrograph scaling. Different solution techniques were used to answer follow-up questions for each of the following case studies.

6.1 Los Banos Dam

Los Banos Dam is an earthfill structure with an uncontrolled spillway. It was completed between 1964 and 1965. This dam is located 7 miles southwest of the town of Los Banos, California, on Los Banos Creek. The drainage basin upstream from the dam has a total area of approximately 156 mi^2. The spillway crest elevation is 353.5 feet and the dam crest elevation is 384.0 feet. The spillway discharge capacity at the dam crest elevation is 11,800 ft^3/s. The total capacity of the dam at water surface elevation 353.5 feet is 34,600 acre-feet. The active capacity between elevations 296 and 353.5 feet is 26,300 acre-feet. The current PMFs for Los Banos Dam were computed in 1996 (Bureau of Reclamation, 1996a). The results of these PMF studies are summarized in table 6-1.

The PMFs were computed using a standard storm arrangement. Routing results indicate that Los Banos Dam would be overtopped by a flood with a magnitude about 37.6 percent of the general

storm PMF (Bureau of Reclamation, 1996b). The general storm PMF would overtop the dam by 6.2 feet for 30 hours. The thunderstorm PMF does not overtop the dam.

Table 6-1.—Probable maximum flood summary for Los Banos Dam, California

Flood Type	HMR	Peak Inflow (ft^3/s)	Volume (acre-feet)	
			With 100-year antecedent storm event	Without 100-year antecedent storm event
General storm	58	75,800	146,700 (7 day)	138,000 (4.3 days)
Thunderstorm	58	75,200	27,400 (24 hr)	—

Because Los Banos Dam is overtopped by the general storm PMF, flood probabilities are needed to understand and quantify the risk of overtopping and hydrologic hazard for dam safety. A flood frequency and volume frequency analysis was completed using the procedures recommended for a CFR.

6.1.1 Los Banos Hydrologic Hazard Curves Using Flood Frequency Analysis and Hydrograph Scaling

A recent flood study, including paleoflood data, peak-flow frequency, and hydrographs, was completed for a risk analysis (Weghorst and Klinger, 2002). The streamflow and paleoflood data from that study are summarized below and then used to compute a hydrologic hazard curve. Because streamflow gage records for Los Banos Creek at Los Banos Dam were limited, the records from the gaging station on Orestimba Creek, near Newman, California (U.S. Geological Survey [USGS] station No. 11274500), were used in this example. This gage is located just north of Los Banos Dam, has a drainage area of about 134 mi^2, and provides 71 years (1932 to 2002) of unregulated peak discharge records. The peak discharge data were adjusted by the square root of the drainage area ratio (Cudworth, 1989); this factor was $(156/134)^{0.5} = 1.078$.

There are several sources of regional and reconnaissance-level paleoflood data that may be applied to estimates of flood frequency at Los Banos Dam including reconnaissance-level data on Los Banos Creek and supporting data from site-specific soil stratigraphic information on the Cantua Stream Group, located about 55 miles south of Los Banos Creek (Weghorst and Klinger, 2002). The hydrometeorologic setting for the Cantua Stream Group and Los Banos Creek, in general, appears to be very similar to each other; however, no field verification was performed to confirm this. Each of the basins is located on the eastern flank of the central Coast Ranges, and the drainage basin areas are close enough in size that differences in their runoff characteristics would be negligible given similarities in rock types, basin aspect and slope, average elevation, and vegetation and land use. Therefore, the paleoflood peak discharge bounds and age estimates from the Cantua Stream Group are believed to be applicable to Los Banos Creek (Weghorst and Klinger, 2002). Paleoflood bounds were established by scaling (Cudworth, 1989) the peak discharges for paleoflood data on the Cantua Stream Group to the Los Banos basin. It appears that peak discharges in the range of 42,000 to 60,000 ft^3/s on Los Banos Creek have not been exceeded in the last 1,800 – 2,800 years.

A hydrologic hazard curve for Los Banos Dam was constructed using the flood frequency analysis and hydrograph scaling procedures recommended for a CFR. Peak flow and paleoflood data and a PMF general storm hydrograph were used to estimate the hazard curve (figure 6-1). Estimated peak flows and volumes can exceed the PMF peak and volume at this site (table 6-1 and table 6-2). Therefore, the PMF was considered as an upper limit for design and risk analysis (Bureau of Reclamation, 2002).

Hydrographs for these floods were then routed through Los Banos Reservoir and spillway. The results from this characterization of the hydrologic hazard and flood hydrograph routing indicate that Los Banos Dam may potentially be overtopped by a flood with a return period of about 2,800 years (based on peak flow). Using these results, Los Banos Dam does not meet Reclamation hydrologic hazard criteria for overtopping because it does not pass a 10,000-year flood (at a minimum). Because this dam does not meet Reclamation criteria, additional studies for Los Banos are required to further assess the flood risk, estimate the need for any structural modifications, and determine suitable probability-based design hydrographs for any potential modification or design alternatives.

6.2 A.R. Bowman Dam

The case study for A.R. Bowman Dam is an example of a project where several hydrologic studies have been completed. The dam is an earthfill structure with an uncontrolled spillway. It was completed in 1961. This dam is located about 20 miles upstream from Prineville, Oregon, on the Crooked River. The drainage basin upstream of the dam has a total area of approximately 2,635 mi^2. The spillway crest elevation is 3234.8 feet, and the dam crest elevation is 3264.0 feet. The spillway capacity at the dam crest elevation is 11,500 ft^3/s. The total capacity of the dam at water surface elevation 3257.9 feet is 233,100 acre-feet. The current PMFs for A.R. Bowman Dam were computed in 1994. The results of these studies are summarized in table 6-3.

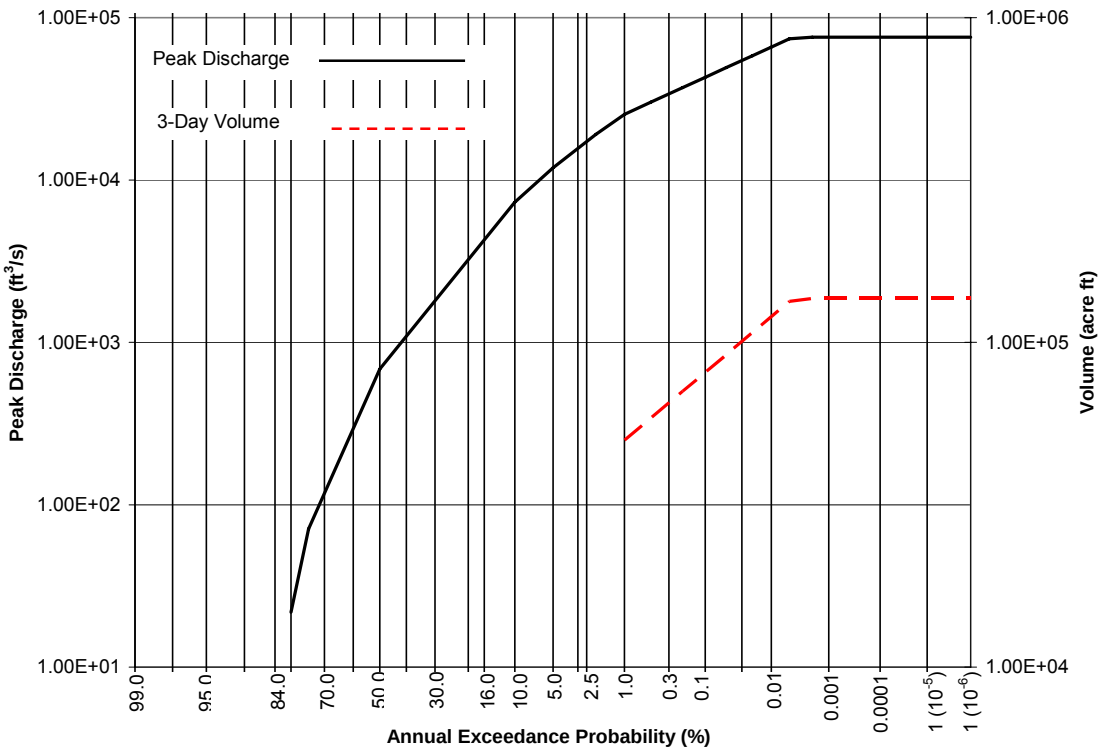

Figure 6-1.—Example of hydrologic hazard curve for Los Banos Dam, California.

Table 6-2.—Peak and volume (3-day) estimates at Los Banos Dam, California, for specified probabilities

Annual exceedance probability	Return period	Peak discharge estimate (ft^3/s)	3-day volume estimate (acre-feet)
0.01	100	25,400	50,100
0.005	200	30,100	58,500
0.002	500	37,000	70,800
0.001	1,000	42,800	80,800
0.0005	2,000	49,100	91,700
0.0002	5,000	58,300	107,200
0.0001	10,000	65,900	120,000
0.00005	20,000	74,100	133,700
0.00002	50,000	75,800	136,500

Table 6-3.—Probable maximum flood summary A.R. Bowman Dam, Oregon

Flood type	HMR	Peak inflow (ft^3/s)	Volume (acre-feet)	
			With 100-year antecedent conditions	Without 100-year antecedent conditions
February General storm	57	255,000	770,000 (15 day)	138,000 (4.3 days)
June General storm	57	83,300	185,600 (15 day)	185,600 (15 day)
August General storm	57	75,000	—	112,600 (15 day)

Routings of the PMF hydrographs indicated that the February general storm event, with a starting water surface elevation of 3211.17 feet, would overtop the dam by 18.6 feet. A.R. Bowman Dam would only pass approximately 10 to 25 percent of the February general storm PMF without overtopping. The June and August PMFs did not overtop the dam.

Because A.R. Bowman Dam is overtopped by the February general storm PMF, flood probabilities are needed to understand and quantify the risk of overtopping and hydrologic hazard for dam safety. Several investigations have been undertaken since about 1991 to determine PMF design modification alternatives, including overtopping protection and parapet walls. In addition, detailed probabilistic flood studies using alternative approaches (stochastic event flood modeling and paleoflood studies) were completed at A.R. Bowman Dam to better estimate the flood risk.

Three methods for estimating the hydrologic hazard are discussed in this section: flood frequency analysis and hydrograph scaling, SEFM, and peak-flow frequency analysis with detailed paleoflood data. Flood risk results from the three methods and implications for potential modifications at A.R. Bowman Dam are then discussed.

6.2.1 A.R. Bowman Hydrologic Hazard Curves Using Flood Frequency Analysis and Hydrograph Scaling

Peak flow data at A.R. Bowman Dam were obtained from three USGS gaging stations:

- Crooked River at Post, Oregon (USGS station No. 14079500)
- Crooked River above Prineville Reservoir near Post, Oregon (USGS station No. 14079800)
- Crooked River near Prineville, Oregon (USGS station No. 14078050)

These gages are the closest to A.R. Bowman Dam; the first two are located upstream from the reservoir, and the gage near Prineville is located downstream from the dam. The combined gages provide 38 years (1909 to 1972, with many missing years) of unregulated peak discharge records. The peak discharge data for the upstream gages were adjusted by the square root of the drainage area ratio (Cudworth, 1989). The December 1964 flood was treated as a historic peak with a return period of approximately 140 years (largest since about 1860).

The reconnaissance-level paleoflood data that may be applied to estimate flood frequency at A.R. Bowman Dam are based on the detailed paleoflood data presented below. Information from the paleohydrologic bound with the largest discharge was used and was dated based on the Mount Mazama volcanic eruption 7,600 years ago. Based on this information, it appears that a peak discharge between 27,000 and 36,000 ft^3/s on the Crooked River has not been exceeded in the last 7,600 to 10,000 years.

A hydrologic hazard curve for A.R. Bowman Dam was constructed using the techniques outlined in section 5. This example calculation was completed after the results of the SEFM and paleoflood study were completed. This example represents the first step that might be performed in future flood studies for Reclamation dam safety. Peak flow and paleoflood data and a general storm PMF hydrograph were used to estimate the hazard curve (figure 6-2 and table 6-4).

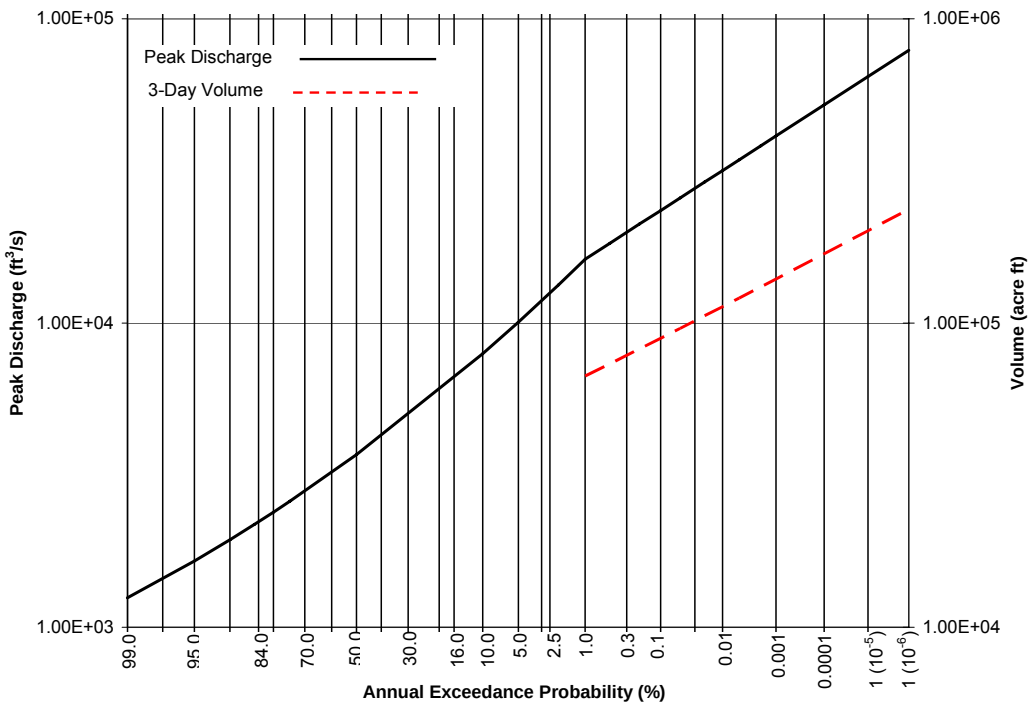

Figure 6-2.—Example of a hydrologic hazard curve for A.R. Bowman Dam, Oregon.

6.2.2 A.R. Bowman Hydrologic Hazard Estimates Based on a Stochastic Event Flood Model

MGS Engineering Consultants, Inc., was contracted in November 1997 (Schaefer and Barker, 1997) to perform a hydrologic stochastic analysis to develop magnitude frequency curves for flood peak discharge, runoff volume, and maximum reservoir elevation. The SEFM was developed using a deterministic rainfall-runoff model (HEC-1) and treated input parameters that were selected by Monte Carlo sampling procedures as a probabilistic distribution of values rather than fixed values. The SEFM is described in section 4.5. After the initial modeling runs and results at A.R. Bowman Dam were made, the precipitation depth-area relationships were refined (Schaefer and Barker, 1998) and new flood frequency relationships were developed.

The results from the SEFM include peak flow, volume, and reservoir elevation frequency curves. These graphs are shown below and include polynomial equations fitted to model output that are applicable in the range of AEP from 10^{-2} to 10^{-5} (Schaefer and Barker, 1998). From these results, the dam may be potentially overtopped with a maximum reservoir elevation probability equal to 0.0005 (2,000-year return period) (figure 6-4).

Peak discharge estimate:

$$Q_p = 2322\,Log^2(AEP) - 2241\,Log(AEP) + 950$$

Flood Runoff Volume Estimate:

$$Q_v = 8385\,Log^2(AEP) - 59700\,Log(AEP) - 39490$$

Maximum reservoir elevation estimate:

$$ELEV_{\max} = -0.3993\,Log^4(AEP) - 6.193\,Log^3(AEP) - 33.67\,Log^2(AEP) - 89.81\,Log(AEP) + 3159.2$$

Table 6-4.—Peak and volume (3-day) estimates at A.R. Bowman Dam, Oregon, for specified probabilities

Annual exceedance probability	Return period	Peak discharge estimate (ft^3/s)	3-day volume estimate (acre-feet)
0.01	100	16,000	65,100
0.005	200	18,100	71,300
0.002	500	21,000	79,700
0.001	1,000	23,300	86,300
0.0005	2,000	25,700	93,000
0.0002	5,000	29,100	102,100
0.0001	10,000	31,700	109,200
0.00005	20,000	34,500	116,600
0.00002	50,000	38,400	126,600
0.00001	100,000	41,500	134,500
0.000001	1,000,000	52,700	162,400
1E-07	10,000,000	65,500	193,200
1E-08	100,000,000	80,300	227,300

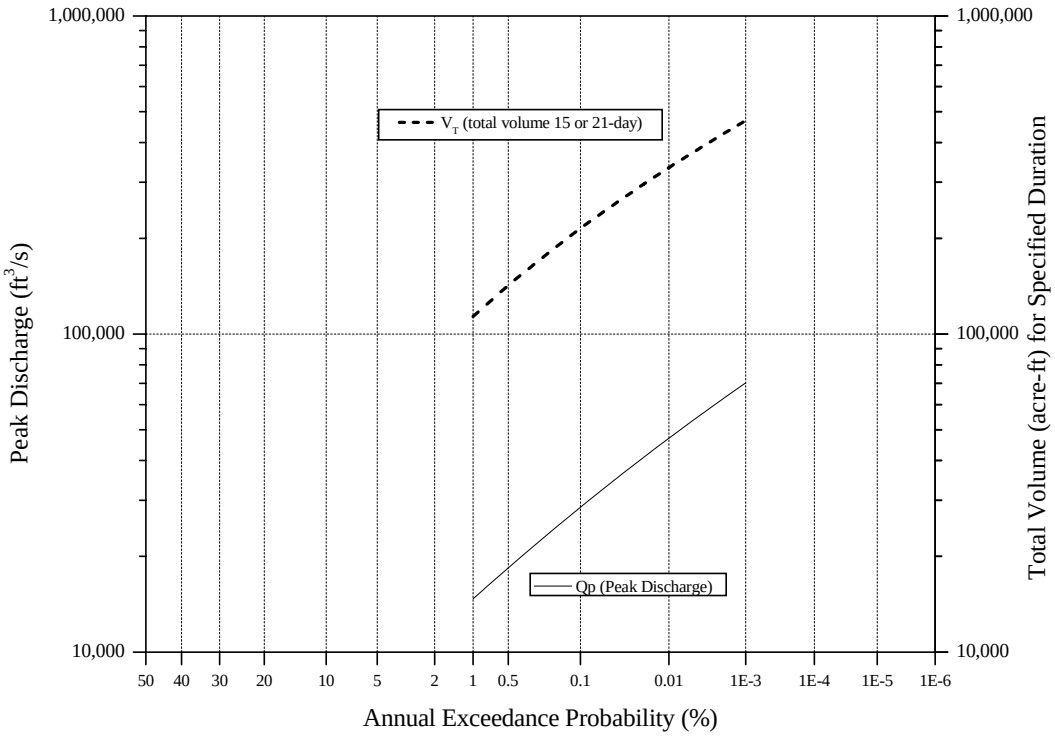

Figure 6-3.—Peak discharge and total volume frequency curves from the SEFM for A.R. Bowman Dam, Oregon.

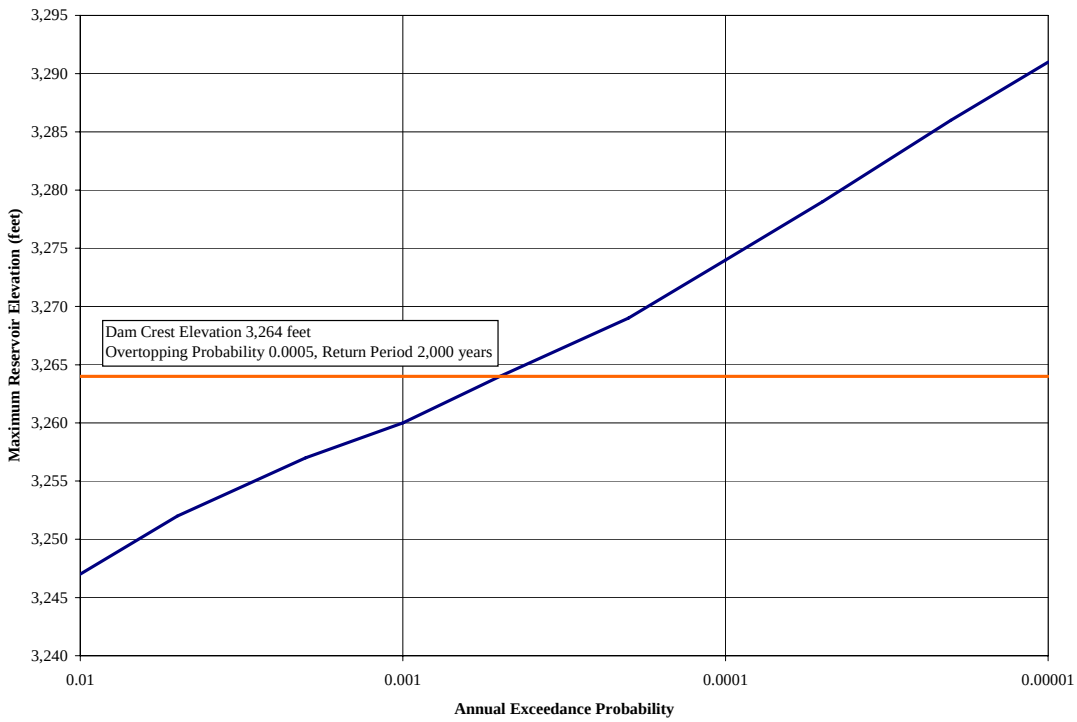

Figure 6-4.—Maximum reservoir elevation frequency curve from the SEFM for A.R. Bowman Dam, Oregon.

6.2.3 A.R. Bowman Hydrologic Hazard Estimates Using Bayesian Statistical Estimation

To gain information on the magnitude of low probability floods for risk analysis, the Dam Safety Office, in 1995, requested a paleoflood study on the Crooked River at A.R. Bowman Dam. The paleoflood study was an alternative approach to the SEFM. The paleoflood report has not been published because of changing priorities in the Dam Safety Office.

For the Crooked River paleoflood study, two reaches downstream from A.R. Bowman Dam were selected for detailed hydraulic modeling. These reaches are about 2 and 35 miles downstream from the dam. At these study reaches, there is geomorphic, stratigraphic, and botanic evidence to limit the paleostage of floods throughout the Holocene epoch (the past 10,000 years). In one of the downstream reaches, it is possible to reconstruct the magnitude of the December 1861 flood based on the presence of driftwood piles. In addition to these downstream study reaches, a third study reach was identified just upstream from Prineville Reservoir, where the stage of paleofloods during the late Holocene epoch can be related to the peak discharge estimates from the December 1964 flood.

Stratigraphy and soils were described at eight sites along the Crooked River, including the three study reaches. At these sites, the soils were described in detail and material was collected for radiocarbon dating. In total, there are 54 radiocarbon ages for the Crooked River paleoflood study. In addition to these radiocarbon ages, the Mazama ash forms an important stratigraphic datum along the river downstream from A.R. Bowman Dam. The calibrated ages of radiocarbon samples associated with the Mazama ash throughout the western U.S. yield an age of about 7,650 years. The hydraulic geometry of the Crooked River channel is remarkably consistent at all the study sites. This implies long-term stability of the Crooked River channel and of the mechanisms that produce large floods on the Crooked River.

The paleoflood data that have been collected and analyzed to date are summarized in table 6-5.

Table 6-5.—Detailed paleoflood estimates at A.R. Bowman Dam, Oregon

Type	Preferred age (years before present)	Age range (years)	Preferred discharge (ft^3/s)	Discharge range	
				Low (ft^3/s)	High (ft^3/s)
1862 historical bound	140	—	15,000	13,000	18,000
Paleohydrologic bound	1,100	980 – 1,210	20,700	18,000	25,000
Paleohydrologic bound	3,150	2,650 – 3,650	25,000	22,000	30,000
Paleohydrologic bound	3,500	3,350 – 5,050	27,000	25,000	32,500
Paleohydrologic bound	9,000	7,600 – 10,000	30,000	27,000	36,000
Historical flood	140	—	20,700	18,000	25,000
Paleoflood	1,100	980 – 1,210	20,700	18,000	25,000

The paleoflood data were combined with peak flows from three gaging stations. A frequency analysis was performed using FLDFRQ3 (O'Connell, 1999; O'Connell et al., 2002). The peak-flow frequency results for a particular distribution (log-Pearson) are shown in figure 6-5 and summarized in table 6-6. Based on this peak-discharge frequency curve and scaling model hydrographs, the dam might be overtopped at exceedance probabilities greater than 0.0001 (more frequently than the 10,000-year return period).

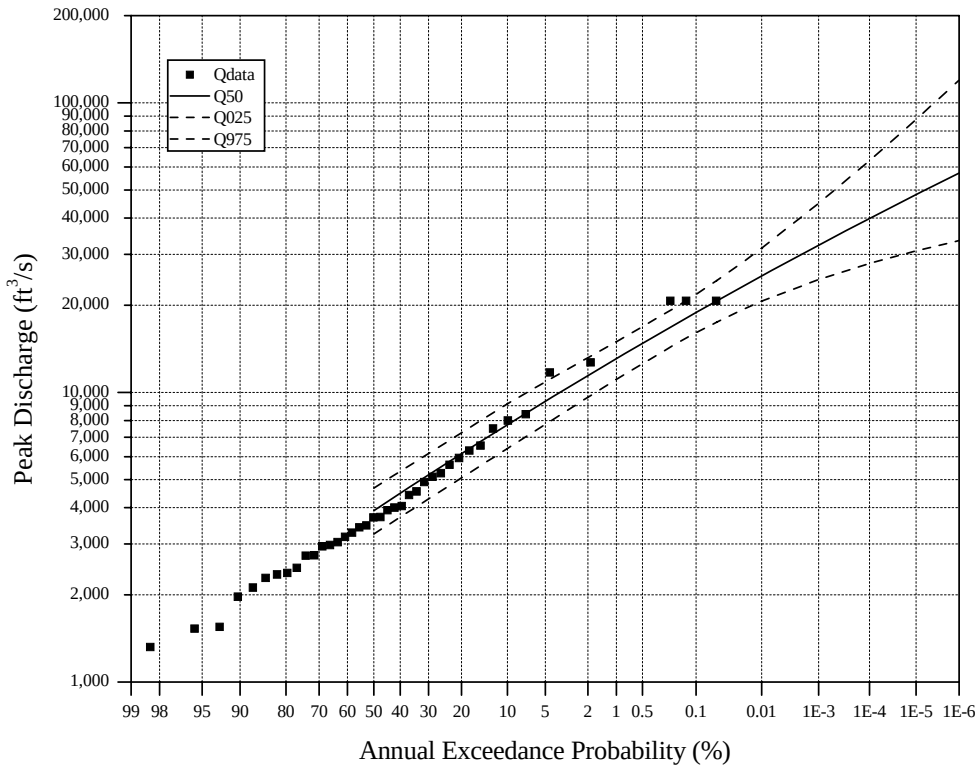

Figure 6-5.—Peak-discharge frequency curve based on paleoflood data for A.R. Bowman Dam, Oregon.

Table 6-6.—Peak discharge estimates based on paleoflood data at A.R. Bowman Dam, Oregon, for specified probabilities

Annual Exceedance Probability	Return Period	Median (50%) LP3 Model Peak Discharge Estimate (ft^3/s)	2.5% LP3 Model Peak Discharge Estimate (ft^3/s)	97.5% LP3 Model Peak Discharge Estimate (ft^3/s)
0.01	100	13,100	11,100	15,000
0.002	500	17,100	14,600	19,500
0.001	1,000	18,900	16,100	21,800
0.0005	2,000	20,700	17,500	24,400
0.0002	5,000	23,300	19,300	28,200
0.0001	10,000	25,200	20,700	31,500
0.00001	100,000	32,200	24,500	44,900
0.000001	1,000,000	39,800	27,900	63,200
0.0000001	10,000,000	48,200	30,800	87,500
0.00000001	100,000,000	57,100	33,400	119,700

6.2.4 Combined Hydrologic Hazard Estimates for Risk Analysis and Dam Safety Implications

Both the SEFM and paleoflood study indicate that A.R. Bowman Dam potentially might not meet Reclamation hydrologic hazard criteria for overtopping because it does not pass the 10,000-year flood (at a minimum). The results from the three approaches (flood frequency analysis and hydrograph scaling, SEFM, and paleoflood study) were combined to provide a single estimate of the hydrologic hazard curve for risk analysis. The weighting schemes are simple numerical weights given to each model result. Three weighting schemes were analyzed for A.R. Bowman: (1) enveloping all results (figure 6-6), (2) weighting the SEFM and paleoflood peak flow curves equally (50 percent each) (figure 6-7), and (3) choosing a "best estimate" between the SEFM and paleoflood peak flow curves equal to the upper 97.5 percent confidence limit from the Bayesian analysis (figure 6-8). The third option was selected to represent the peak discharge flood frequency based on results and decisions made in 1999 for a risk analysis. A "combined" hydrologic hazard curve is shown in figure 6-9, which also includes a volume frequency analysis based on results of the SEFM.

Because A.R. Bowman Dam might not meet Reclamation criteria, additional engineering studies are ongoing to estimate the need for any potential structural modifications at this site. Probability-based design hydrographs for any potential modification or design alternatives were selected based on a combination of the weighting results shown above. Corrective action studies are ongoing.

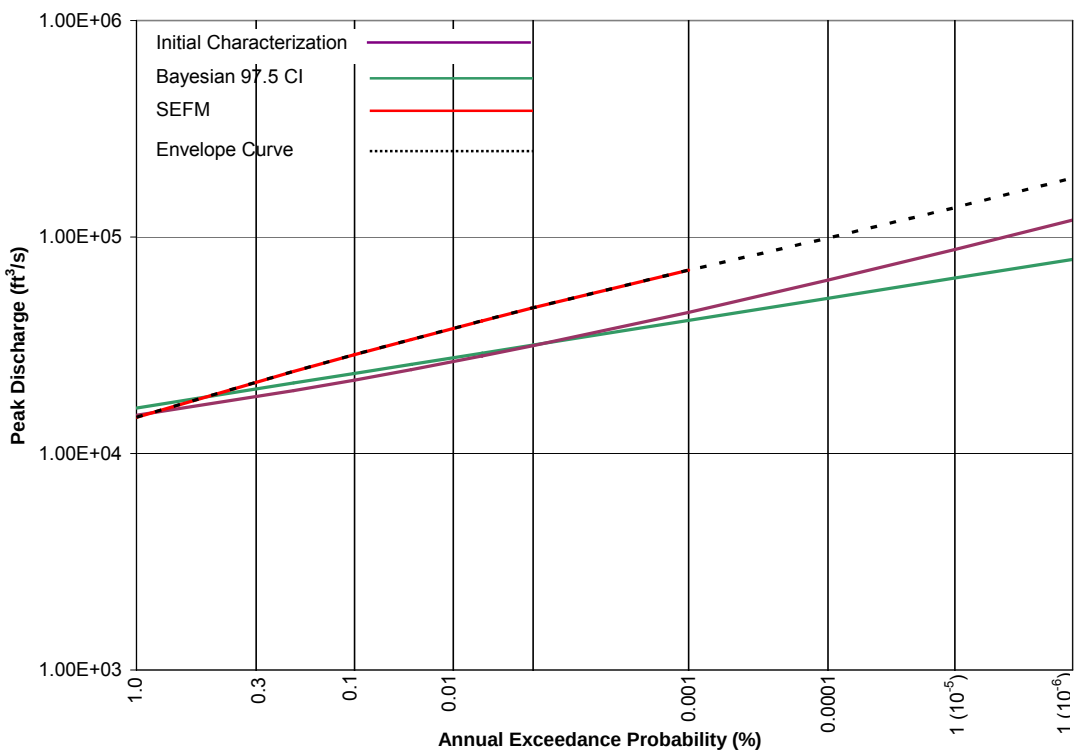

Figure 6-6.—Weighted peak-discharge frequency curve based on enveloping results for A.R. Bowman Dam, Oregon.

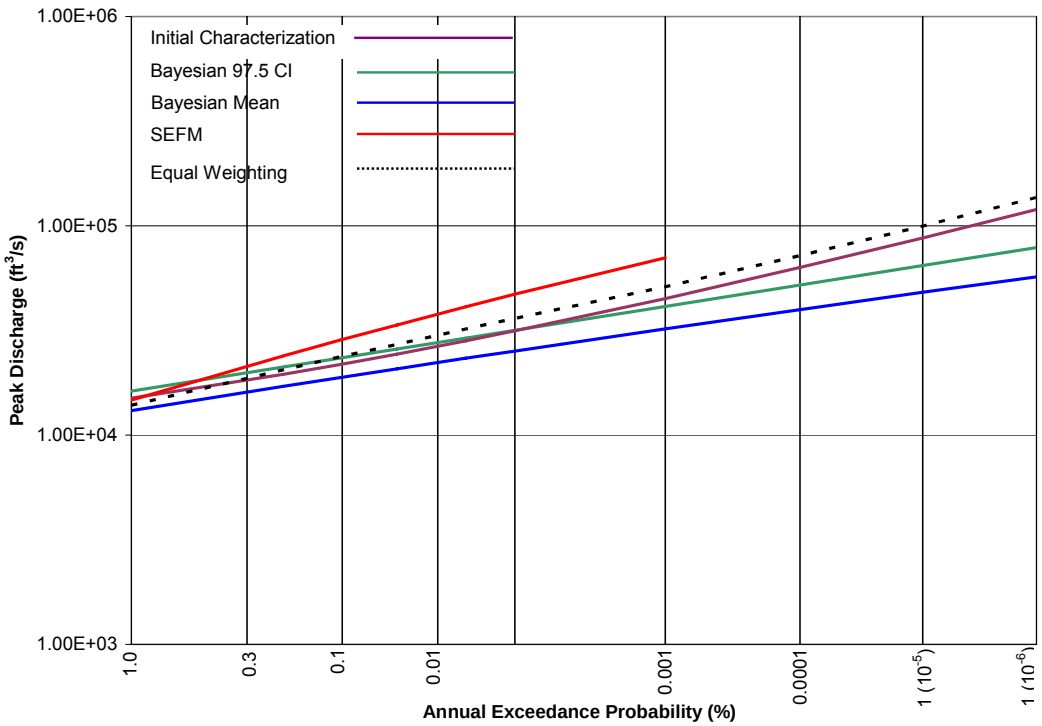

Figure 6-7.—Equal (50 percent) weighted peak-discharge frequency curve for A.R. Bowman Dam, Oregon.

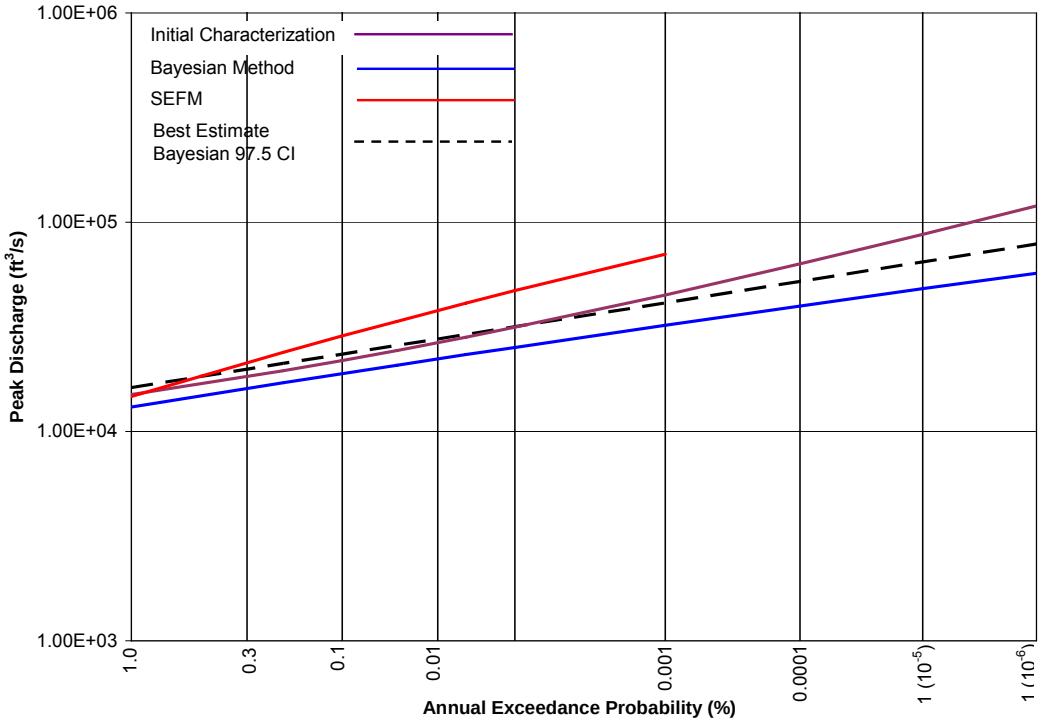

Figure 6-8.—Best estimate (Bayesian 97.5 percent) weighted peak-discharge frequency curve for A.R. Bowman Dam, Oregon.

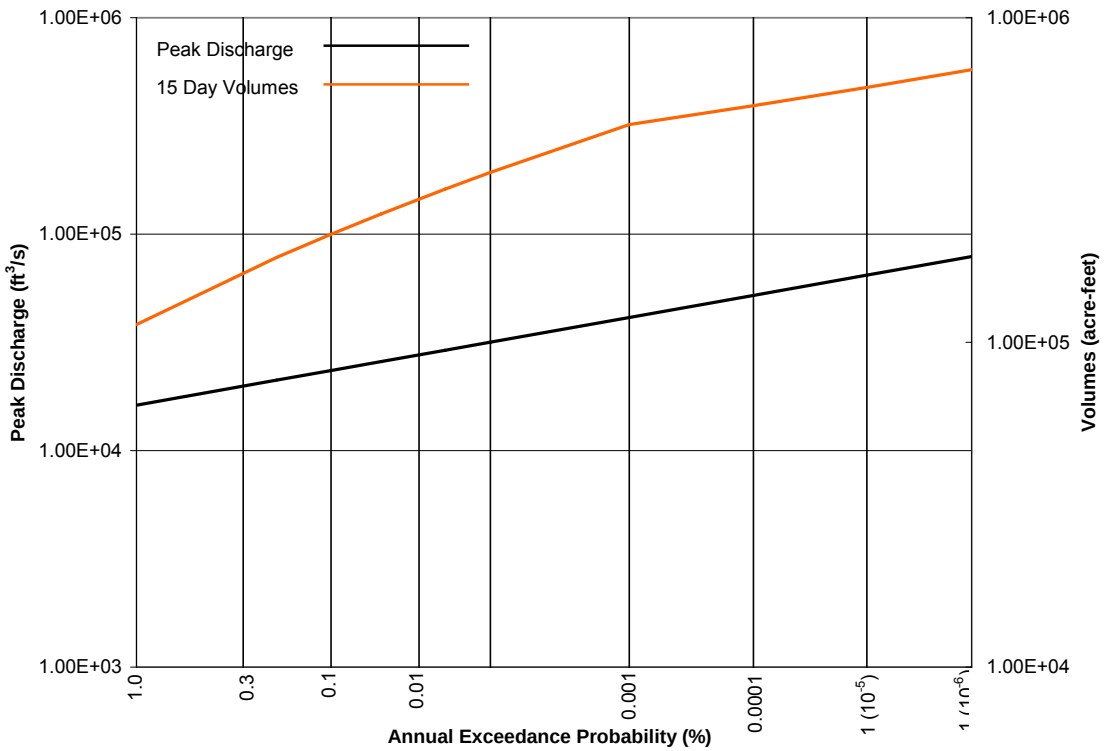

Figure 6-9.—Final combined hydrologic hazard curve for A.R. Bowman Dam, Oregon.

6.3 Fresno Dam

Fresno Dam is located on the Milk River in north-central Montana, approximately 14 miles west of the town of Havre (figure 6-10). The compacted earthfill dam was constructed from 1937 to 1939. It is approximately 110 feet high. Previous studies have indicated that the dam could safely pass Reclamation's 1985 PMF hydrograph. Normally, Reclamation would not have conducted additional hydrologic studies for this dam; however, project beneficiaries wanted to examine the possibility of enlarging the dam.

6.3.1 Fresno Dam Hydrologic Hazard Curves Using Flood Frequency Analysis and Hydrograph Scaling

A hydrologic hazard curve for Fresno Dam was constructed using flood frequency analysis techniques. Peak flow and paleoflood data and a general storm PMF hydrograph were used to estimate the hazard curve (figure 6-11 and table 6-7).

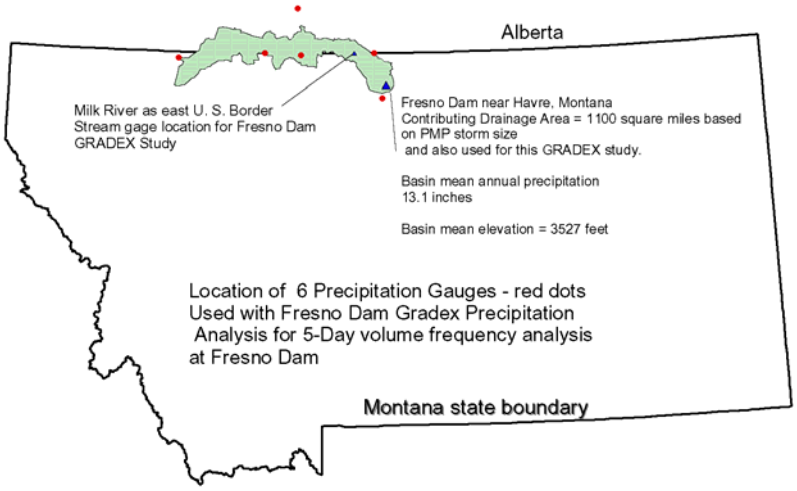

Figure 6-10.—Site and watershed depiction of Fresno Dam, Montana.

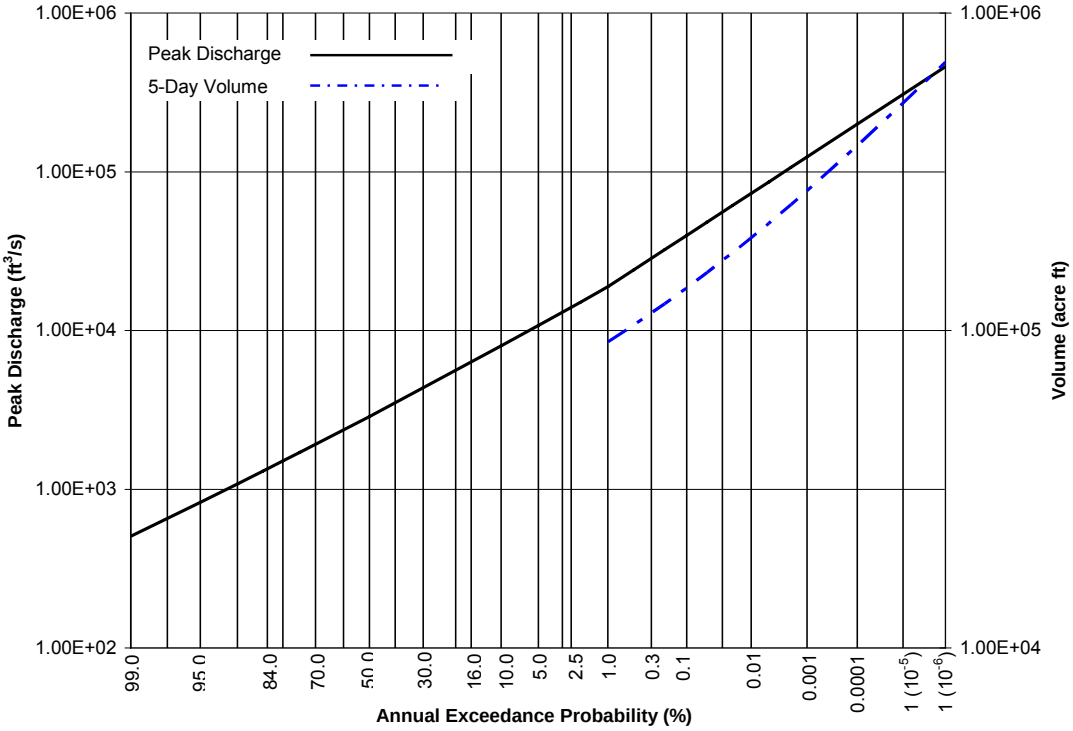

Figure 6-11.—Example of hydrologic hazard curve for Fresno Dam, Montana.

Table 6-7.—Peak and volume (5-day) estimates at Fresno Dam, Montana, for specified probabilities

Annual Exceedance Probability	Return Period	Discharge Estimate	5-Day Volume Estimate (acre ft)
0.01	100	18,900	92,000
0.005	200	24,100	103,900
0.002	500	32,300	121,400
0.001	1000	39,700	136,100
0.0005	2000	48,200	152,300
0.0002	5000	61,500	176,000
0.0001	10000	73,100	195,900
0.00005	20000	86,400	217,600
0.00002	50000	106,600	249,500
0.00001	100000	124,200	276,000
0.000001	1000000	199,600	382,100
0.0000001	10000000	307,800	520,800
0.00000001	100000000	459,400	700,300

Peak flow data at Fresno Dam were obtained from the USGS gaging station, Milk River at Eastern U.S. Border Crossing (USGS station No. 06135000). The gage provides peak flow records from 1910 through 2002. No adjustments were performed on these data, as the associated drainage area and streamflow records were believed to accurately represent the conditions at Fresno Dam.

The reconnaissance-level paleoflood data that may be applied to estimate flood frequency at Fresno Dam are based on detailed chronology developed at archaeological sites on the Milk River. Based on this information, it appears that a peak discharge between 40,000 and 70,000 ft^3/s on the Milk River has not been exceeded in the last 2,000 to 4,000 years.

Fresno Dam has a spillway discharge capacity of 51,360 ft^3/s at the maximum water surface elevation of 2591 feet. Comparing this value with the peak-discharge flood frequency curve indicates that the spillway is capable of passing a flood with a return period of only slightly longer than 2,000 years. At the time of this study, the methods for determining the volume frequency relationship had not been developed, so hydrographs were not available for reservoir routing. Therefore, we decided to develop hydrographs using the GRADEX Method.

6.3.2 Fresno Dam Hydrologic Hazard Analysis Using the GRADEX Method

Daily flow records that approximate the daily inflows to Fresno Dam come from the stream gage on the Milk River at the Eastern U.S. Border Crossing (USGS station No. 06135000). This gage record has daily flow data from August 1909 through September 2001. A total of 86 years of daily flow data is available, with a few missing years. The contributing drainage area at the gage site is considered to be 2,525 mi^2, and the gage is located about 54 river miles upstream from Fresno Dam. The total possible contributing drainage area at Fresno Dam is considered to be 2,911 mi^2 for the rainfall-runoff modeling calculations in this study. These daily flow records were checked with available reservoir inflow records for concurrent years.

The information needed from the stream gage record is a set of independent flood volumes, representing the volume of flooding caused by rainfall for independent storms in the basin. The process starts by setting a threshold flow value and then counting the number of events that have continuous days of flow above that value. A threshold flow value was set and adjusted until about 86 independent flood events above this value were determined. The average duration of all of the independent flood events was then calculated. This average duration of independent flood events for the Milk River above Fresno Dam was found to be between 4 and 5 days, with a threshold value of 1,307 ft^3/s. The value of 5 days was selected for further use in the study as the critical duration for both rainfall and flood events.

The GRADEX Method required that the number of multi-day rain events at each gage be equal to the number of common years of record for the selected gages. The number of multi-day rain events was made equal by selecting a threshold 5-day total rainfall for each gage such that exactly fifty 5-day rain totals were above the threshold value for each gage. Figure 6-10 displays the locations of the six precipitation gage stations selected for the GRADEX analysis at Fresno Dam. Table 6-8 lists the six stations used for the remainder of this analysis along with some pertinent information including the MAP.

Table 6-8.—Rainfall data description

Selected Rainfall Stations with a Period of Record Including 1950-2000 Used with Fresno Dam GRADEX Study for high return period volumes Fresno basin mean basin elevation = 3527 feet Fresno basin mean annual precip. = 13.1 inches					
Station	**State/Prov.**	**Latitude**	**Longitude**	**Elevation (feet)**	**M.A.P. (inches)**
Babb	MT	48.93	113.36	4300	18.27
Gold Butte	MT	48.98	111.4	3498	13.38
Kremlin	MT	48.52	110.1	2860	11.56
Simpson	MT	49	110.22	2815	10.24
Sweetgrass	MT	49	111.97	3466	13.98
Foremost	ALB	49.48	111.45	2899	14.68

Using the 5-day precipitation totals, 5-day flow volumes, drainage area, and other data described above, the GRADEX Method computations were made as outlined in Naghettini (1994) and Naghettini et al. (1996). For this study, a special situation arises with respect to selection of the contributing drainage area above Fresno Dam.

Based on all previous Reclamation IDF and PMF studies for Fresno Dam, the authors of this study believe that a single-storm precipitation pattern will not cover the entire basin. For this GRADEX study, a total contributing drainage area of 1,100 mi^2 was selected, based primarily on the use of the hypothetical elliptical patterns used with the PMP studies.

Table 6-9 and figure 6-12 display the summary results from the calculations for Fresno Dam. The volume of the PMF volume centered hydrograph from the 1985 Reclamation PMF study is plotted as a straight line on the frequency curve for comparison. It should be noted that the PMF is based on a 3-day rainfall, and the Fresno Dam frequency curve is based on 5-day total rainfall amounts. The GRADEX Method is intended to give flood volume data for very large return periods, in excess of 200 years. More common methods, such as a LP-III distribution fit to available streamflow data, should be used for the lower return periods, 100 years and less in this study.

Table 6-9.—Five-day volume frequency curve for Fresno Dam, Montana, based on GRADEX Method

Return Period (years)	Five-day Volume (acre-ft)	Upper Bound (acre-ft)	Lower Bound (acre-ft)
500	188,700	320,100	112,400
1,000	214,700	358,700	131,100
5,000	275,100	448,200	174,700
10,000	301,100	486,800	193,500
20,000	327,100	525,400	212,300
50,000	361,600	576,400	237,200
100,000	387,600	615,000	256,000
500,000	448,000	704,600	299,600
1,000,000	474,100	743,100	318,500

In most normal years, snowmelt may contribute a large amount to the total volume of inflows to Fresno Reservoir. Another assumption in the GRADEX Method is that the precipitation that is measured at the gages is in liquid form and ready for runoff, or if it is snow, the snow will melt during the critical storm-time period and add to the runoff volume. A simple scheme to separate purely snowmelt runoff from rain or rain-on-snow runoff was used with the 86 years of daily flows at the Milk River at Eastern U.S. Border Crossing gage. The 7-day maximum of purely snowmelt flow was then determined for each year and expressed as a constant flow for 7 days. The 86-year-long series of 7-day maximum snowmelt flows at the gage site was then analyzed using a standard LP-III analysis. The lower curve in figure 6-13 displays the frequency curve of maximum 7-day purely snowmelt flows. The curve is quite flat. This is expected for purely snowmelt driven volumes. There is a physical limit to how much snow can be melted with normal ranges of climate variables. The snowmelt volume frequency curves will become quite flat for larger return periods. The 7-day flows were then converted to volumes in acre-feet and a similar frequency curve was also plotted in figure 6-13.

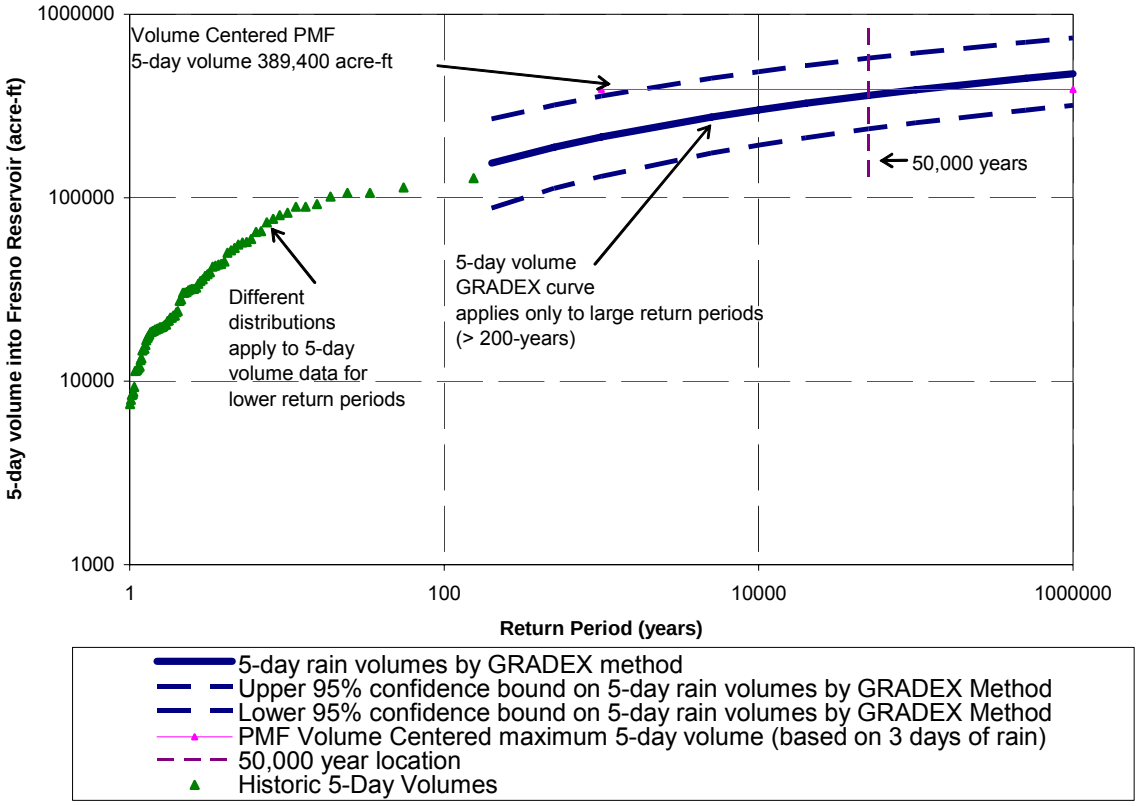

Figure 6-12.—Fresno Dam, Montana, 5-day rain-generated volumes by GRADEX Method.

The 5-day volumes of the GRADEX rain flood volumes from this study were then added to the plot. This is the blue curve near the top of figure 6-13. A combined probability curve was then computed and is shown as the red curve on the top of figure 6-13. The combined probability curve expresses the probability (or return period) of getting a particular total flood volume by combining a 7-day snowmelt with a 5-day rain flood volume. The combined probability curve very closely follows the 5-day rain flood volume curve for the large return periods.

Hourly inflow hydrographs were developed for Fresno Dam for the desired return periods by applying indexed rainfall amounts to the HEC-HMS models calibrated to the 1906 and 1964 rainfall flood events. The rainfall amounts for the 1964 and 1906 storm hyetographs were uniformly increased in each subbasin until the 5-day volume of the respective return periods, determined by the GRADEX Method, was achieved. Figure 6-14 displays the results for the 1964 flood computations. Figure 6-15 displays the results of increasing the 1964 precipitation amount to calculate hydrographs with the GRADEX volumes for specified return periods.

The peak versus volume relationship was developed using a LOWESS technique (Cleveland, 1979). The 86 years of peak and 5-day volume data from the USGS gage at the Milk River at Eastern Border were plotted. In addition, the 30 peak-flow and 5-day volumes generated from

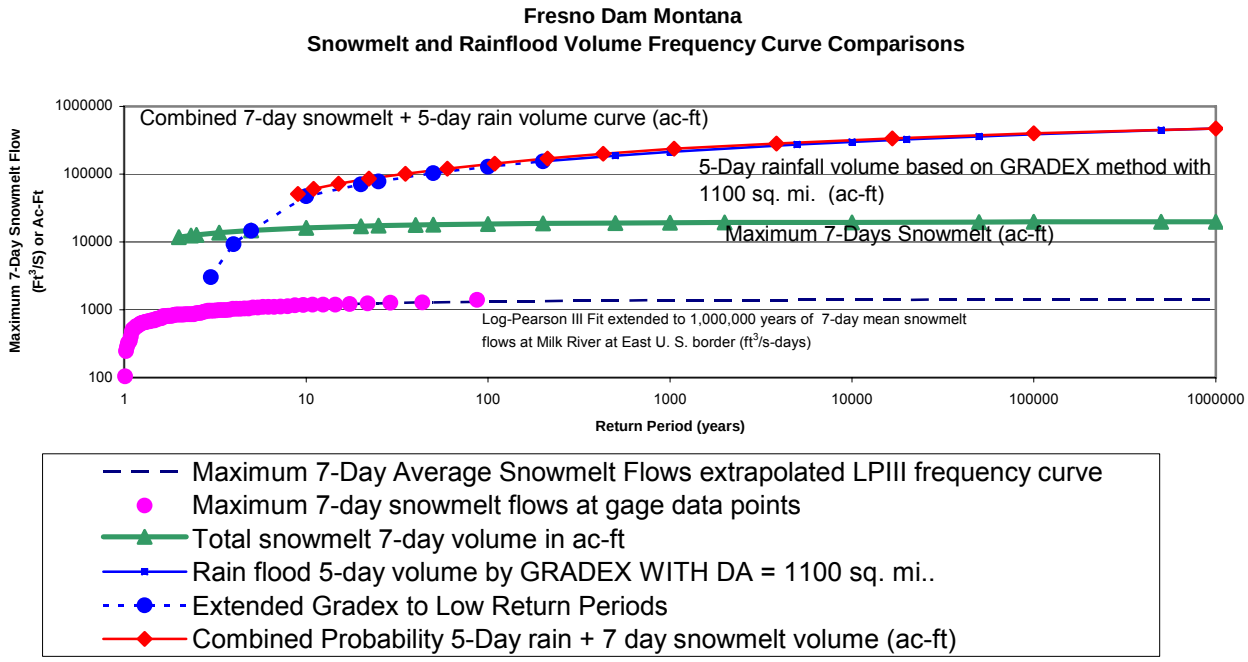

Figure 6-13.—Fresno Dam, Montana, snowmelt and rainflood volume frequency curve comparisons.

the HEC-HMS models originally calibrated to both the 1906 and 1964 flood events were run. In each run, the lag time was varied or the rainfall was increased in a systematic manner to cover a range of possible values for greater storm events. A LOWESS fit was done using the entire set of peak versus volume points and is shown in figure 6-16. This relationship was used to determine the peak discharge estimates that correspond to the selected 5-day return period volumes computed using the GRADEX Method.

The results of this calculation can be used to derive a peak-flow flood frequency curve. The information for the peak flow and volumes for the return periods of interest in this study is summarized in table 6-10. In this study, the reported paleohydrology information indicated a non-exceedance peak flood bound with an age between 2,000 and 4,000 years and a magnitude between 40,000 and 70,000 ft^3/s. The paleohydrology information was in agreement with the GRADEX results.

For Fresno Dam, the results of the GRADEX Method were used for the risk assessment because flood volumes were needed to evaluate the safety of the dam. Since only peak flow data were available at the time of the first analysis, the GRADEX study was conducted. Hydrographs could then be routed through the reservoir to take advantage of the flood attenuation effects of the large surcharge storage in the reservoir.

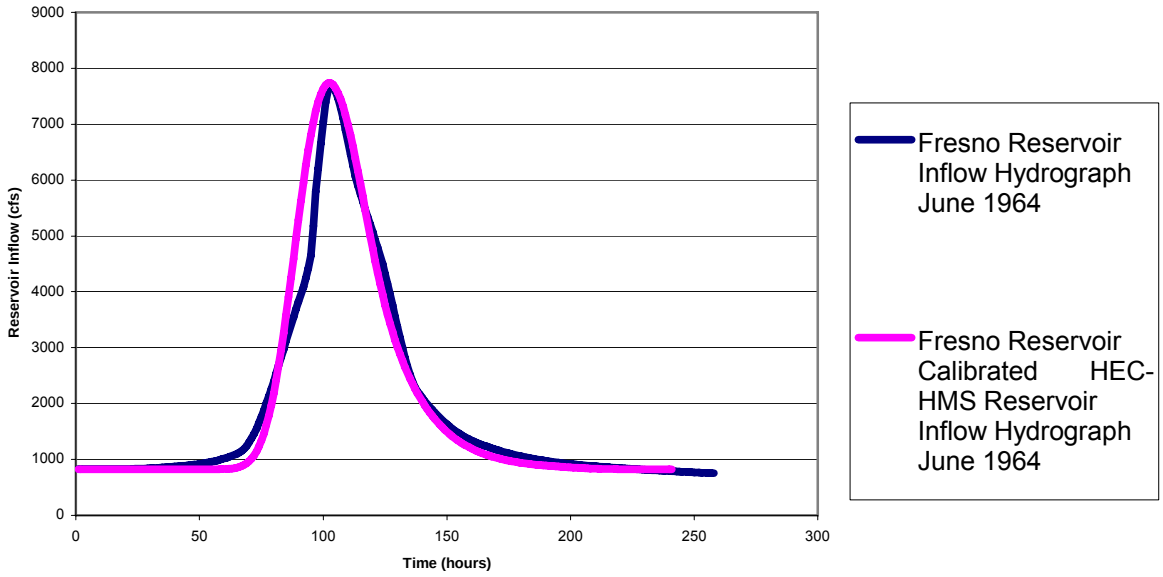

Figure 6-14.—Comparison of gage record and HEC-HMS calibration run.

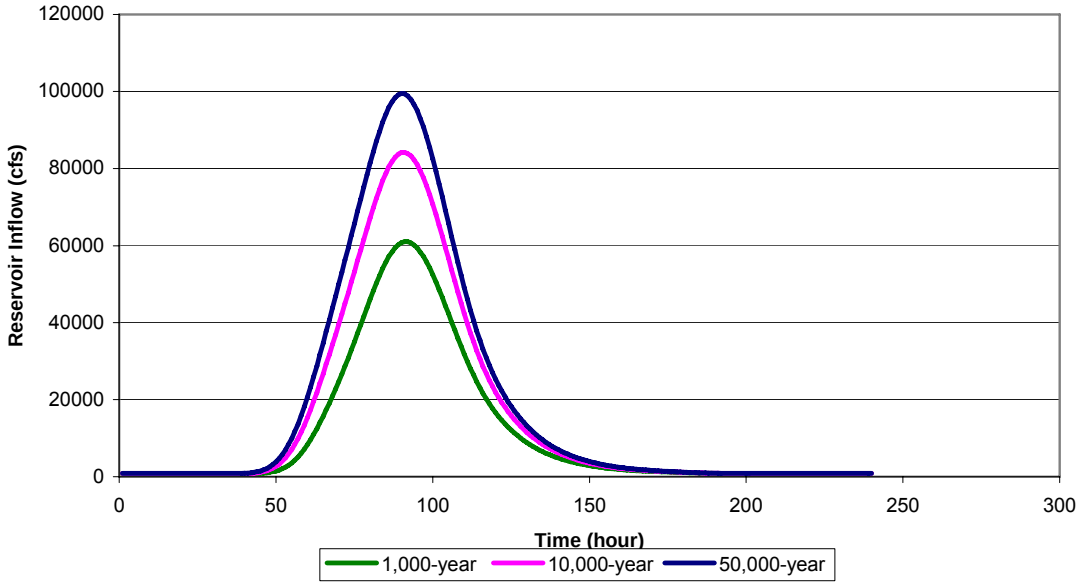

Figure 6-15.—Fresno Dam, Montana, 1,000-, 10,000-, and 50,000-year hydrographs. Hydrographs from 1964 precipitation pattern with volume from GRADEX Method.

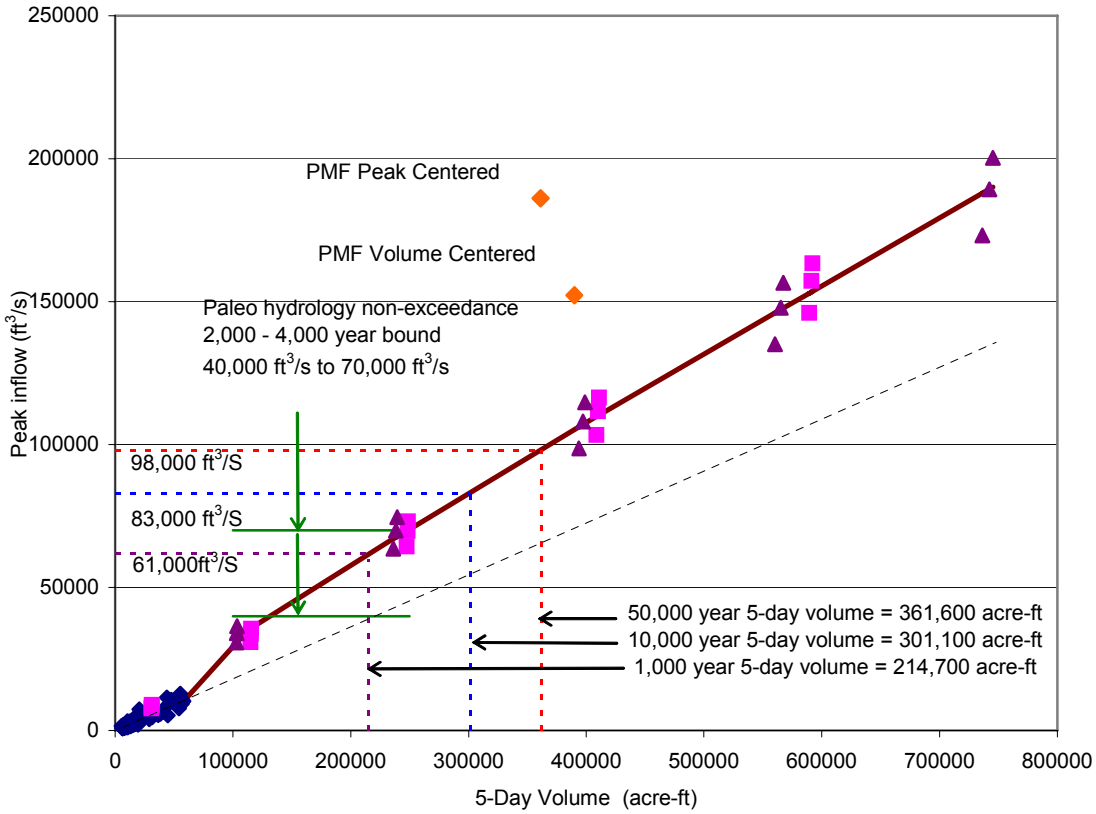

Figure 6-16.—Fresno Dam, Montana, 5-day volumes versus peaks.

Table 6-10.—Fresno Dam, Montana, GRADEX Method results

Return Period (year)	Peak Inflow (cfs)	5-Day Volume (acre-ft)
1,000	61,000	214,700
10,000	83,000	301,100
50,000	98,000	361,600

7. Summary

This report summarizes Reclamation's approach toward developing hydrologic hazard curves for use in evaluating dam safety issues. The procedure relies on extracting information from existing studies to the fullest extent possible. The characterization of hydrologic risk for a CFR can usually be accomplished with minimal effort. CFR hydrologic hazard curves display peak

flow and volume relationships for a full range of Annual Exceedance Probabilities necessary for decision making. The procedures and analysis techniques defined in this report allow for the possibility, and even plausibility, that peak discharge and volume estimates may exceed the PMF. This is a function of the uncertainty and inconsistency among and between analysis techniques. Therefore, in these cases, the PMF is believed to represent the upper limit to hydrologic risk.

The procedure for developing hydrologic hazard curves considers the dam safety decision criteria, potential dam failure mode, and dam characteristics, available hydrologic data, possible analysis techniques, resources available for analysis, and tolerable level of uncertainty. Dam safety decision criteria determine the probabilistic range of floods needed to address hydrologic issues (Bureau of Reclamation, 2003a). The potential dam failure mode and dam characteristics impact the type of hydrologic information needed to assess the problem. The specific elements selected to be incorporated in an analysis of hydrologic hazards should consider the tolerable level of uncertainty. Reducing the uncertainty in the estimates may require additional data collection and use of more sophisticated solution techniques. It is believed that increasing the level of effort and the sophistication of analysis techniques increases the reliability and level of confidence associated with the results.

Reclamation currently uses a combination of seven hydrologic methods to develop hydrologic hazard curves. These general techniques include: flood frequency analysis with historical and paleoflood data, hydrograph scaling and volumes, the GRADEX Method, the Australian Rainfall-Runoff Method, stochastic event-based precipitation runoff modeling with the SEFM, stochastic rainfall-runoff modeling with CASC2D, and the PMF. Each method is described within these guidelines.

The hydrologic methods described in these guidelines are not all inclusive. New techniques for developing hydrologic hazard information can be added to these guidelines as they are developed by the hydrology community.

The amount of effort expended on analyzing a hydrologic hazard depends on the nature of the problem and the potential cost of the solution. A staged approach toward evaluating a hydrologic safety issue is recommended. Initially, very little effort is expended to determine the magnitude of the hydrologic hazard. Reclamation attempts to make use of all available studies for the site of interest. Often, the PMF and initial flood frequency studies are the only hydrologic studies available before the start of a probabilistic investigation. When other hydrologic studies have been performed, available data will be used to decrease uncertainty in results as well as to provide an overall assessment of hydrologic risk.

Dam safety evaluations usually begin with a characterization of hydrologic risk. If detailed studies have been conducted for the site of interest, they are summarized and presented to the risk assessment team. About two-thirds of Reclamation’s dams can safely accommodate the PMF; when the PMF is selected as the IDF, no additional work may be required unless other hydraulic issues need evaluation. Additional hydrologic work begins with a flood frequency analysis and hydrograph scaling to develop a peak-discharge relationship and frequency flood hydrographs. It is believed that this type of information is sufficient to address hydrologic issues and make dam safety decisions at about 80 percent of the remaining dams. For the sites that still

have potential safety problems, project plans can be developed for studies to address the specific hydrologic issues. These studies require more work and more sophisticated solution techniques than flood frequency analysis and hydrograph scaling.

When planning more detailed studies, it is recommended that the goal be to achieve a balance between the amount of hydrologic analysis needed to address the issues and the level of effort required to conduct the study. As the studies get more detailed, the results should become more precise and contain less uncertainty. An example is a hydrologic study that proceeds from flood frequency analysis and hydrograph scaling to an analysis using the GRADEX Method, the Australian Rainfall-Runoff Method, a stochastic rainfall-runoff model, or any other statistical estimation technique available.

When multiple methods are used, alternative hazard curves are developed by weighting results from the individual analyses. A team of hydrologists evaluates the alternatives and selects a weighting scheme that is most representative of the site for use in the risk assessment. Selection of the final hydrologic hazard curve depends on the experience of the hydrologists and the assumptions that went into each analysis.

Three case studies, Los Banos, Fresno, and A.R. Bowman Dams, have been presented to illustrate the variety of methods available. These sites were chosen to demonstrate the use of the various techniques for characterization of the flood hazard. The A.R. Bowman example shows how multiple studies were combined into a single-flood hazard curve for use in risk assessment.

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A FRAMEWORK FOR CHARACTERIZATION OF EXTREME FLOODS FOR DAM SAFETY RISK ASSESSMENTS

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Abstract

Risk-based decisions require different types of information than standards-based decisions. Traditional sources of information used for estimating probabilities of extreme floods include gaged streamflow records, indirect discharge measurements, and precipitation records. Generally these data sources have records that are less than 100 years in length. This framework for flood characterization for risk assessments uses the length of the data record and other characteristics of the data to determine the credible extrapolation limits used in the flood frequency analysis. Because risk assessments require estimation of floods with annual exceedance probabilities of 1 in 10,000, or less, emphasis is placed on developing probabilistic estimates using regional hydrometeorological data and paleoflood information. The uncertainties associated with descriptions of flood flow exceedance probabilities are likely to be substantial and an important attribute to convey into the risk assessment.

No single approach is capable of providing estimates of extreme floods over the full range of annual exceedance probabilities required for risk assessment. Therefore, results from a number of approaches need to be combined to yield a composite flood characterization; this means several methods and sources of data are needed. The application of several independent methods applicable to the same range of annual exceedance probabilities will increase the credibility and resulting confidence in the results.

Introduction

The U.S. Bureau of Reclamation is now making extensive use of quantitative risk assessment in support of dam safety decision making (Von Thun and Smart, 1996). An important input to Dam Safety Risk Assessment is the development of probabilistic extreme flood estimates. This shifts the focus for dam safety flood evaluation from routing a single "maximum" event (i.e. the probable maximum flood, PMF) to consideration of the entire range of plausible inflow flood events, and ultimately to the magnitude-frequency relationship of maximum reservoir stages.

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For floods, the risk assessment process involves selecting a spillway evaluation flood (SEF) based on the probability of dam failure and the severity of the incremental consequences of dam failure. Past practice also examined consequences, but without formal consideration of probability of failure; if consequences were judged to be large, the SEF was chosen as the PMF.

Reclamation has identified the need for a review of its present procedures for developing probabilistic extreme flood estimates and their associated uncertainties for use in dam safety risk assessment. Where practical, Reclamation would like to develop improved procedures. The overall objective is to develop a practical, robust, consistent, and credible framework for developing probabilistic extreme flood estimates for Dam Safety Risk Assessment. The desired outcome is a robust framework in which components can be improved in the future as the state-of-the-art develops.

The framework was developed by inviting a group of approximately 20 professionals with extensive experience in the theoretical and practical aspects of physical, paleo-, and statistical flood hydrology and hydrometeorology to participate in a one-week workshop held at Utah State University in June 1997. Participants from North America, Australia, and the United Kingdom reviewed current Reclamation practice, and evaluated various advances in developing probabilistic extreme flood estimates for their potential role in the needed framework. A smaller group met in Denver to develop the details of the framework. This paper summarizes the findings of these groups.

Risk Assessment Stages

Present Reclamation risk assessment practice uses a staged approach for conducting risk assessments (USBR, 1997a). Project schedule and budget constraints are considered in determining the type of flood assessment prepared at each stage. While each risk assessment is unique, the following stages are generally used in Reclamation risk assessments:

a) **Screening Level Risk Assessment**: An evaluation of risk that includes definition of load probabilities and consequences for all load classes (flood, earthquake, and static). Structure failure probabilities and associated uncertainties are also considered in a global sense, but detailed event trees are not usually prepared. An emphasis at this stage is to maximize the use of available information, without conducting new analyses or collecting additional data. The intent is to identify areas where risks are potentially high and to determine the need for further evaluations and data collection. Results of these evaluations are used to determine Reclamation's risk profile and to "screen" out dam safety issues where additional funding and effort appears to have little potential for reducing dam safety risks.

b) **Scoping Level Risk Assessment**: A more detailed evaluation of risks is performed for the dam safety issues identified in a screening level risk assessment. This level of risk assessment typically involves more detailed treatment of event trees, load probabilities, structural response, and consequences. The intent is to invest sufficient effort so that the risk assessment team understands the major contributors to risk to enable formulation of risk reduction strategies and to determine the need for additional analyses and investigations.

c) **Decision Level Risk Assessment**: At this level, more detailed evaluation of risks is performed to provide decision makers with the information necessary to reach a dam safety decision for a structure. The decision may be related to continuing project operations, correcting dam safety deficiencies, selecting among risk reduction alternatives, or determining the need for interim actions to reduce risk while long term plans are developed. The intent is to provide decision makers with sufficient pertinent risk information such that the risk reduction objective can be effectively considered along with other Reclamation objectives. At this level of risk assessment, detailed loading information, structural response analyses, and consequence evaluations are developed for all significant issues. This type of risk assessment focuses on reducing uncertainties in the risk estimates and evaluating risk reduction actions.

Data Sources

The proposed framework for developing probabilistic extreme flood estimates for risk assessment uses the length of record and other characteristics of the data to determine the extrapolation limits for flood frequency analysis. Traditional sources of information used for flood hazard analyses include streamflow and precipitation records. Generally, these data sources have records that are less than 100 years in length, although in some cases these records can be extended to about 150 years using historical information. Regional precipitation and streamflow data can create pooled data sets from short periods of observation, and paleoflood data can extend records of floods to periods of up to several thousand years.

Streamflow Data

Many different types of streamflow information are used in developing probabilistic extreme flood estimates for risk assessment. Streamflow data are used in flood hazard assessment as input for frequency studies or as the basis for developing flood hydrographs. The usual source of these data is the streamflow records collected and maintained by the U.S. Geological Survey. However, similar data are collected and archived by many other Federal and State government agencies and some non-government organizations. Streamflow records consist of data collected at established gaging stations and indirect measurements of streamflow at other sites. Streamflow data can include estimates of peak discharge, as well as average or mean discharge for various time periods. Most streamflow measurements on U.S. streams began after 1900 with only a few records dating back that far. Most often, streamflow records at a single site range in length from about 20 to 60 years. Completeness of the data set may vary from station to station.

Climate Data

Precipitation and weather data used in hydrologic models can include rainfall, snowfall, snow water equivalent, temperature, solar radiation, and wind speed and direction from individual weather stations, as well as remote sensing information and radar information for broader regions. Data types available from various sources vary greatly in record length and quality throughout the United States. Some of these types of data (i.e., snowfall, snow water equivalent, solar radiation, and wind)

are limited to record lengths of less than about 30 years; basic rainfall and temperature data are available for some stations for up to 150 years, but in most cases are limited to less than 100 years.

Historical Data

Historical data can provide a means for extending the length of record for many types of data, in particular for observations of the most extreme events. These data are most commonly used to extend streamflow records of peak discharge prior to organized stream gaging. Historical observations can provide information for other types of data such as weather patterns and the frequency of extreme storm events, or changes in land use or vegetation that may be significant to runoff modeling calculations. However, as with any type of historical data, the accuracy and validity of the observations must be carefully assessed and compared to the other types of data used in the analysis.

Paleoflood Data

Paleoflood hydrology is the study of past or ancient flood events which occurred prior to the time of human observation or direct measurement by modern hydrological procedures (Baker, 1987). Unlike historical data, paleoflood data do not involve direct human observation of the flood events. Instead, the paleoflood investigator studies geomorphic and stratigraphic records (various indicators) of past floods, as well as the evidence of past floods and streamflow derived from historical, archeological, dendrochronologic, or other sources. The advantage of paleoflood data is that it is often possible to develop records that are 10 to 100 times longer than conventional or historical records from other data sources in the western United States. In addition, the paleoflood record is a long-term measure of the tendency of a river to produce large floods. In many cases, paleoflood studies can provide a long-term perspective, which can put exceptional annual peak discharge estimates in context and assist in reconciliation of conflicting historical records.

Paleoflood data generally include records of the largest floods, or commonly the limits on the stages of the largest floods over long time periods. This information can be converted to peak discharges using a hydraulic flow model. Generally, paleoflood data consist of two independent components. One component is a peak discharge estimate; the second is a time period or age over which the peak discharge estimate applies. Paleoflood studies can provide estimates of peak discharge for specific floods in the past, or they can provide exceedance and non-exceedance bounds for extended time periods. Each of these differing types of paleoflood data must be appropriately treated in flood frequency analyses.

Extrapolation Limits for Different Data Types

The primary basis for a limit on credible extrapolation of extreme flood estimates derives from the characteristics of the data and the record length used in the analysis. The data used in the analysis provide the only basis for verification of the analysis or modeling results, and as such, extensions beyond the data cannot be verified. Different risk assessments require flood estimates for different ranges of annual exceedance probability (AEP), and therefore analysis procedures and data sources should be selected to meet project requirements. The greatest gains to be made in providing

credible estimates of extreme floods can be achieved by combining regional data from multiple sources. Thus, analysis approaches that pool data and information from regional precipitation, regional streamflow, and regional paleoflood sources should provide the highest assurance of credible characterization of low AEP floods.

For many Reclamation dam safety risk assessments, flood estimates are needed for AEPs of 1 in 10,000 and ranging down to 1 in 100,000, or even lower. Developing credible estimates at these low AEPs generally require combining data from multiple sources and a regional approach. Table 1 lists the different types of data which can be used as a basis for flood frequency estimates, and the typical and optimal limits of credible extrapolation for AEP, based on workshop discussions or subsequent communications. The limits presented in the table represent a general group consensus; however, opinions differed amongst workshop participants. In general, the optimal limits are based on the best combination(s) of data envisioned in the western U.S. in the foreseeable future. Typical limits are based on the combination(s) of data which would be commonly available and analyzed for most sites.

Many factors can affect the equivalent independent record length for the optimal case. For example, gaged streamflow records in the western United States only rarely exceed 100 years in length, and extrapolation beyond twice the length of record, or to about 1 in 200 AEP, is generally not recommended (IACWD, 1982). Likewise, for regional streamflow data the optimal limit of credible extrapolation is established at 1 in 1,000 AEP by considering the number of stations in the region, lengths of record, and degree of independence of these data (Hosking and Wallis, 1997). For paleoflood data, only in the Holocene epoch, or the past 10,000 years, is climate judged to be sufficiently like that of the present climate, for these types of records to have meaning in estimates of extreme floods for dam safety risk assessment. This climatic constraint indicates that an optimal limit for extrapolation from paleoflood data, when combined with at-site gaged data, for a single stream should be about 1 in 10,000 AEP. For regional precipitation data, a similar limit is imposed because of the difficulty in collecting sufficient station-years of clearly independent precipitation records in the orographically complex regions of the western United States. Combined data sets of regional gaged and regional paleoflood data can be extended to smaller AEPs, perhaps to about 1 in 40,000, in regions with abundant paleoflood data. Analysis approaches that combine all types of data are judged to be capable of providing credible estimates to an AEP limit of about 1 in 100,000 under optimal conditions.

In many situations, credible extrapolation limits may be less than optimal. Typical limits would need to reflect the practical constraints on the equivalent independent record length that apply for a particular location. For example, many at-site streamflow record lengths are shorter than 100 years. If in a typical situation the record length is only 50 years, then the limit of credible extrapolation might be an AEP of about 1 in 100. Similarly, many paleoflood records do not extend to 10,000 years, and extensive regional paleoflood data sets do not currently exist. Using a record length of about 4,000 years, a typical limit of credible extrapolation might be an AEP of 1 in 15,000 based on regional streamflow and regional paleoflood data.

The information presented in Table 1 is intended as a guide; each situation is different and should be assessed individually. The limits of extrapolation should be determined by evaluating the length of record, number of stations in a hydrologically homogeneous region, degree of correlation between stations, and other data characteristics which may affect the accuracy of the data.

Ideally, one would like to construct the flood frequency distribution for all floods that could conceivably occur. However, the limits of data and flood experience for any site or region place practical limits on the range of the floods to which AEPs can be assigned. There does not appear to be sufficient data to justify computation of AEPs less than 1 in 100,000. In general, the scientific limit to which the flood frequency relationship can be credibly extended, based upon any characteristics of the data and the record length, will fall short of the probable maximum flood (PMF) for a site. PMF estimates provide a useful reference to past practice and can be compared with extreme floods characterized for risk assessment. However, the workshop participants concluded that there is limited scientific basis for assigning an AEP to the PMF. For precipitation data, similar limitations apply to extrapolations that approach values described by probable maximum precipitation.

Table 1. Hydrometeorological Data Types and Extrapolation Limits for Flood Frequency Analysis

Type of Data Used for Flood Frequency Analysis	Limit of Credible Extrapolation for Annual Exceedance Probability	
	Typical	Optimal
At-site streamflow data	1 in 100	1 in 200
Regional streamflow data	1 in 750	1 in 1,000
At-site streamflow and at-site paleoflood data	1 in 4,000	1 in 10,000
Regional precipitation data	1 in 2,000	1 in 10,000
Regional streamflow and regional paleoflood data	1 in 15,000	1 in 40,000
Combinations of regional data sets and extrapolation	1 in 40,000	1 in 100,000

Methods of Analysis

At Site Flood Frequency Analysis

Frequency analysis is an information problem: if one had a sufficiently long record of flood flows, or perhaps rainfall for a basin, then a frequency distribution for a site could be determined with good precision, so long as change over time due to anthropogenic or natural processes did not alter the distribution of floods. In most situations available data are insufficient to precisely define the annual exceedance probability of large floods. This forces hydrologists to use practical knowledge of the physical processes involved, and efficient and robust statistical techniques, to develop their estimates (Stedinger et al., 1993).

Fitting a distribution to data sets allows both a compact and smoothed representation of the frequency distribution revealed by the available data, and a systematic procedure for extrapolation to frequencies beyond the range of the data set. Given a family of distributions, one can estimate the parameters of that distribution so that required quantiles and expectations can be calculated with the "fitted" model. Appropriate choices for distribution functions can be based upon examination of the data using probability plots, the physical origins of the data, previous experience, or prescriptive guidelines.

Several general approaches are available for estimating the parameters of a distribution. A simple approach is the *method of moments,* which uses the available sample to compute estimators of the distribution's parameters. The Federal guidelines published in Bulletin 17B (IACWD, 1982) recommend fitting a Pearson type 3 distribution to the common base 10 logarithms of the peak discharges. It uses at-site data to estimate the sample mean and variance of the logarithms of the flood flows, and a combination of at-site and regional information to estimate skewness.

Another method that may be used to estimate the parameters of a distribution for at-site frequency analysis is the Expected Moments Algorithm (EMA). EMA (Cohn et al., 1997) is a moments-based estimation procedure and is identical to the existing Bulletin 17B (IAWCD, 1982) approach when no high or low outliers are present. The EMA method was developed to utilize historical and paleoflood information in a censored data framework. This approach explicitly acknowledges the number of known and unknown values above and below a threshold, similar to a maximum-likelihood approach. Three types of at-site flood information are used: systematic stream gage records; information about the magnitudes of historical floods; and knowledge of the number of years in the historical period when no large flood occurred.

Still another method, which has strong statistical motivation, is the *method of maximum likelihood*. Maximum likelihood estimators (MLEs) have very good statistical properties in large samples, and experience has shown that they generally do well with records available in hydrology. In many cases MLEs cannot be reduced to simple formulas, so estimates must be calculated using numerical methods (Stedinger et al., 1988; O'Connell, 1997).

L-moments are another way to summarize the statistical properties of hydrologic data. Sample estimators of L-moments are linear combinations (and hence the name L-moments) of the ranked observations, and thus do not involve squaring or cubing the observed values as do the product-moment estimators. As a result L-moment estimators of the dimensionless coefficients of variation and skewness are almost unbiased and have very nearly a normal distribution (Hosking and Wallis, 1997).

Regional Flood Frequency Analysis

In hydrology, sufficient information is seldom available at a site to adequately determine the frequency of rare events using frequency analysis. This is certainly the case for the extremely rare events which are of interest in dam safety risk assessment. The National Research Council (1988) has proposed several general strategies, including substituting space for time for estimating extreme floods. One substitutes space for time by using hydrologic information at different locations in a region to compensate for short records at a single site.

Three approaches (Cudworth, 1989) have been considered for regional flood frequency analysis: (1) average parameter approach; (2) index flood approach; and (3) specific frequency

approach. With the average parameter approach, some parameters are assigned average values based upon regional analyses, such as the log-space skew or standard deviation. Other parameters are estimated using at-site data, or regression on physiographic basin characteristics, perhaps the real or log-space mean. The index flood method is a special case of the average parameter approach. The specific frequency approach employs regression relationships between drainage basin characteristics and particular quantiles of a flood frequency distribution.

Index Flood Method. The index flood procedure is a simple regionalization technique with a long history in hydrology and flood frequency analysis (Dalrymple, 1960). It uses data sets from several sites in an effort to construct more reliable flood-quantile estimators. A similar regionalization approach in precipitation frequency analysis is the station-year method, which combines precipitation data from several sites without adjustment to obtain a large composite record to support frequency analyses. The concept underlying the index flood method is that the distributions of floods at different sites in a "region" are the same except for a scale or index-flood parameter which reflects the size, rainfall and runoff characteristics of each watershed. Generally the mean is employed as the index flood (Hosking and Wallis, 1997).

Average Shape Parameter. As at-site records increase in length, procedures that estimate two parameters, with at-site data to be used with a regional shape parameter, have been shown to perform better that index flood methods in many cases (Stedinger and Lu, 1995). For record lengths of even 100 years, 2-parameter estimators with a good estimate of the third shape parameter, are generally more accurate than are 3-parameter estimators (Lu and Stedinger, 1992; Stedinger and Lu, 1995). However, whether or not it is better to also regionalize the coefficient of variation depends upon the heterogeneity of the regions and the coefficients of variability of the flows. In regions with high coefficients of variation (and high coefficients of skewness) index flood methods are more attractive.

Regional Regression. Regional analysis can be used to derive equations to predict the values of various hydrologic statistics (including means, standard deviations, quantiles, and normalized regional flood quantiles) as a function of physiographic characteristics and other parameters. Stedinger and Tasker (1985, 1986a, 1986b) developed a specialized Generalized Least Squares (GLS) regression methodology to address the regionalization of hydrologic statistics. Advantages of the GLS procedure include more efficient parameter estimates when some sites have short records, an unbiased model-error estimator, and a better description of the relationship between hydrologic data and information for hydrologic network analysis and design.

Design Event-Based Precipitation-Runoff Modeling

Precipitation-runoff modeling is typically used as an event-based method for determining extreme floods. A single set of hydrometeorological parameters and watershed characteristics are used to simulate a design flood event. The major inputs to a design event-based precipitation-runoff model are: (1) climate data (rainfall, snowfall, and other variables needed to predict snowmelt); (2) losses (infiltration/interception); (3) physical watershed characteristics for runoff and routing simulations (drainage areas, watershed and channel slopes, lag times, antecedent moisture, etc.); (4) precipitation-runoff transformation function; and (5) runoff conveyance/routing mechanisms. Model output includes runoff hydrographs at user-specified locations, maximum peak discharges, and total

runoff volumes. Examples of this type of model include HEC-1 (USACE, 1990) and RORB (Laurenson and Mein, 1995).

Stochastic Event-Based Precipitation-Runoff Modeling

In the stochastic approach, hydrologic model inputs are treated as random variables. Monte Carlo sampling procedures are used to allow the input variables to vary in accordance with their observed distributions, including the observed dependencies among some climatic and hydrologic parameters. The use of the stochastic approach with regional precipitation information allows the estimation of flood magnitude-frequency curves for flood peak discharge, flood runoff volume, and reservoir level. An example of this type of model is discussed by Barker et al. (1997).

Atmospheric Storm Modeling and Continuous Precipitation-Runoff Modeling

This method combines the work of atmospheric modelers and regional precipitation analysis to derive a precipitation magnitude-frequency curve (Chin et al., 1997). The atmospheric model is used to generate storms over the watershed, and the findings from the regional analysis are used to estimate the annual exceedance probability of point and areal precipitation generated by the model. Using distributed precipitation-runoff modeling, snowpack and other antecedent conditions can be combined to estimate a simulated flood frequency curve using a Monte Carlo approach.

Data Generation and Continuous Simulation Modeling

The data generation and continuous simulation modeling approach is based on Monte Carlo generation of long and detailed sequences of hydrometeorological variables, including precipitation, air temperature, and wind speed and direction. In order to represent spatial differences across the watershed adequately, it is necessary to generate hydrometeorological variables for several sites concurrently. Hydrological models of watershed behavior and hydraulic models of confluences, wave effects and reservoir outlets are used to simulate the reservoir water level continuously. An estimated magnitude-frequency relationship of maximum reservoir stages is input to the risk assessment (Calver and Lamb, 1996).

Combining Methods and Data Types

No single approach is capable of providing the needed characterization of extreme floods over the full range of annual exceedance probabilities that may be required for risk assessment. In particular, characterization of floods with AEPs less than 1 in 10,000 can be expected to require that results from a number of approaches, based on multiple data sources, need to be combined to yield a composite flood frequency description. The application of several independent methods and types of data applicable to the same range of annual exceedance probabilities will increase the credibility and resulting confidence in the results.

Table 2 lists various methodologies that were considered for characterizing extreme floods to support dam safety risk assessment. A flood frequency analysis must be combined with each of these methodologies to assign annual exceedance probabilities to the floods.

The framework developed for Reclamation does not propose a specific methodology for rigorously combining information from these differing data sources and methodologies in an overall statistical framework. In some cases the information may be combined statistically, and in other cases one set of results may be used as a bound on the frequency distribution obtained by analysis of other data. Clearly, this process will require a measure of judgement. Regardless of the approach taken for combining results, it should incorporate sound physical and scientific reasoning for weighting or combining results.

All floods characterized for the risk assessment process should display the uncertainties resulting from the analysis. As the risk assessment moves from the screening and scoping levels to the decision level, uncertainty should be reduced and better quantified so that appropriate information is included in the dam safety decision-making process.

Table 2. Applicability of Hydrologic Methods of Analysis to Various Risk Assessment Levels

Method of Analysis	Risk Assessment Level		
	Screening	Scoping	Decision
Flood frequency analysis	Yes	Yes	Yes
Design event-based precipitation-runoff modeling	No	Yes	Yes
Stochastic event-based precipitation-runoff modeling	No	Yes	Yes
Distributed simulation modeling	No	No	Yes
Atmospheric modeling and distributed precipitation-runoff modeling	No	No	Yes

Evaluation of Uncertainty

Uncertainty can be evaluated by applying Monte Carlo analysis to the overall risk assessment calculations. For example, consider the estimation of threat to life consequences and probability of failure associated with an existing dam and various risk reduction alternatives. One is concerned with uncertainty due to such risk assessment inputs as flood frequency distribution parameters, system response estimates, population at risk, warning time, and estimated loss of life. Then in each iteration of Monte Carlo analysis, one could generate likely values of each of these inputs and evaluate the threat to life and probability of failure. The expected annual life loss and the annual exceedance probability of failure, which are both used as Reclamation Public Protection Guidelines (USBR, 1997b), could be computed for each iteration. By generating many replicates, one obtains samples that describe the possible values of these risk measures (performance metrics).

Averaging over the replicates provides "expected" values of the quantities reflecting both the modeled probability distributions of the phenomena (risk assessment inputs) that are considered to be random variables, and the uncertainty in the parameters describing those distributions. The sample standard deviations describe the variability of the performance metrics. Replicates can be

used to estimate frequency distributions which can be used for describing and evaluating the decision implications of uncertainty in the risk assessment inputs.

Calibration to Flood Frequency Quantiles

The ability of a flood event model to reproduce historic events certainly gives some confidence to the validity of subsequent estimates. However, even in a well gaged watershed the annual exceedance probabilities of the calibration floods are likely to range between 1 in 5 to 1 in 20, and only occasionally up to 1 in 100. While it would be expected that floods of these magnitudes will activate some floodplain storage, the non-linear nature of drainage basin flood response is such that the routing characteristics of larger events may be considerably different. Thus, while calibration of a model provides valuable information on the flood response of a drainage basin, caution is needed when using the calibrated model to estimate floods of much larger magnitudes (Pilgrim and Cordery, 1993).

Calibration to flood frequency quantiles using design rainfall inputs can provide important information on flood response characteristics for extreme design events (Nathan and Bowles, 1997; Nathan, 1992). With this approach, design rainfall information is prepared for a specified AEP, and then used with a given set of model parameters and input assumptions to derive a design hydrograph. The peak (or volume) of the design hydrograph can then be compared to the corresponding quantile obtained from a combined at-site/regional flood frequency analysis. The model inputs associated with the greatest uncertainty can be varied within appropriate limits to ensure agreement with the selected flood quantile. Model calibration should be undertaken for a range of AEPs to ensure a consistent variation of parameters with flood magnitude or AEP.

For risk-based studies based on a "design storm concept", it is necessary to adopt an AEP-neutral approach, where the objective is to derive a flood with an AEP equivalent to its concomitant precipitation (Nathan and Bowles, 1997). The factors that influence the transfer between precipitation and runoff can be characterized by probability distributions, and ideally the design hydrograph should be determined by considering the joint probabilities of all the input factors. Monte-Carlo methods are ideally suited to the AEP-neutral objective, as they accommodate the observed variability of the inputs while still preserving the interdependencies between parameters. Simpler approaches may be appropriate, where the decrease in rigor is offset by the computational convenience and the transparency of the adopted functional relationships. For the least important parameters it may be appropriate to adopt a single representative (mean) value instead of the full distribution. However, the relationship between rainfall and runoff is non-linear, and adoption of a single representative value for the major inputs will introduce bias into the transformation. Accordingly, for more important inputs it is necessary to adopt a joint probability approach. The nature of the method can be tailored to suit the relative importance of the parameter concerned.

Conclusions

A framework has been developed for characterizing extreme floods for the purposes of dam safety risk assessment. By incorporating regional information on precipitation, floods, and paleofloods with good at-site records, it is possible to provide scientifically credible flood estimates to annual exceedance probabilities as low as 1 in 100,000, although higher AEP limits may exist in

many cases. In general, the scientific limit to which the flood frequency relationship can be extended based upon available data will fall short of the PMF for a site. PMF estimates provide a useful reference to past practice and can be compared with floods characterized for risk assessment; however, there is limited scientific basis for assigning an annual exceedance probability to the PMF.

No single approach is capable of providing the needed characterization of extreme floods over the full range of AEPs required for risk assessment. Therefore, the results from several methods and sources of data should be combined to yield a composite characterization. The application of several independent methods applicable to the same range of AEPs will increase the credibility and resulting confidence of the results.

Uncertainties associated with descriptions of flood flow exceedance probabilities are likely to be substantial and an important attribute for the characterization of extreme floods. Flood characterization should include a "best estimate" of the annual exceedance probability of floods of different magnitudes and a description of the uncertainty in such results. Such uncertainties need to be honestly represented and considered throughout the risk assessment process.

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The U.S. Bureau of Reclamation's Dam Safety Office sponsored the Workshops and other activities which have resulted in the proposed framework for characterization of extreme floods for dam safety risk assessment. Some twenty professionals from the U.S., Canada, Australia and Europe participated in this effort and each contributed in some way to the resulting framework. These individuals were: David Achterberg, Victor Baker, David Bowles, David Cattanach, Sanjay Chauhan, John England, David Goldman, Chuck Hennig, Don Jensen, Lesley Julian, Jong-Seok Lee, Dan Levish, Jim Mumford, Rory Nathan, Dan O'Connell, Dean Ostenaa, Duncan Reed, Mel Schaefer, Lou Schreiner, Jery Stedinger, Robert Swain, Jim Thomas, and Ed Tomlinson. This paper was also presented at the Eighteenth U.S. Committee on Large Dams Annual Meeting and Lecture held in Buffalo, New York and is included in the conference proceedings.

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Bureau of Reclamation Hydrologic Research

Presented at the FEMA Workshop on Hydrologic Research Needs for Dam Safety

By
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Introduction

The U.S. Bureau of Reclamation is now making extensive use of quantitative risk assessment in support of dam safety decision-making (Von Thun and Smart, 1996). An important input to Dam Safety Risk Assessment is the development of probabilistic extreme flood estimates. The focus has shifted from routing a single "maximum" event (i.e. the probable maximum flood, PMF) to consideration of the entire range of plausible inflow flood events, and ultimately to the magnitude-frequency relationship of maximum reservoir stages.

Reclamation has identified the need for a review of its present procedures for developing probabilistic extreme flood estimates and their associated uncertainties for use in dam safety risk assessment. Where practical, Reclamation would like to develop improved procedures. The following sections of this paper describe ongoing hydrologic research activities for use in dam safety analysis.

Flood Hydrology Database

A variety of hydrologic data are used as input in flood frequency analyses, probabilistic hydrographs, stochastic rainfall-runoff models and other hydrology studies aimed at making flood hazard probability statements for use in risk analyses. The physical understanding and modeling of extreme flood events requires collection and review of a variety of flood hydrology data including peak discharge and mean daily stream flow records, precipitation (rainfall and snowfall) and temperature data, soil data, paleoflood information, extreme rainfall data, and many other sources of information. A searchable database that includes organized and connected lists of available data, visual selection capability, and links to the data would allow faster and more thorough assessment.

The purpose of this research is to continue development of a hydrology database that will include a variety of hydrology data such as peak discharge estimates, paleoflood data, precipitation and temperature data, as well as potential sources of infiltration characteristics and other geologic properties of drainage basins. These data would be used as input into flood frequency analyses, probabilistic hydrograph development, and prediction of basin response in stochastic modeling of extreme flooding. This project focuses on the development of a flood hydrology database that identifies, summarizes, and links hydrologic data that is needed for developing flood frequency analyses and probabilistic hydrographs, as input for stochastic rainfall-runoff models, and other hydrology studies. The eventual goal of this project is to provide spatial and temporal data for use in probabilistic flood hazard studies in the 17 western states.

A variety of data including information on extreme peak discharge have been gathered for the Sierra Nevada region of northern California. Paleoflood data have been gathered throughout the western U.S., as well as in a database at the University of Arizona, tree.ltrr.arizona.edu/~katie/paleofld.html. Currently, Reclamation has a paleoflood database in Microsoft Access and hydrology database in Arcview exist as separate databases. These databases need to be integrated into one database in order to efficiently store and access information for flood-related studies. In addition, computer code and a user interface have been developed for the hydrology database that will allow the user to access records by graphically selecting an area. By the end of 2002, a preliminary user-friendly database is scheduled for completion. This data information system will require updating and continual maintenance.

Revision and Update of Precipitation-Frequency Studies for the United States and Its Possessions

Since the mid 90's, meteorologists in both Reclamation's Flood Hydrology Group and River Systems & Meteorology Group have been addressing the need to revise and update precipitation-frequency estimates for the United States. The demand for this work is obvious in that precipitation-frequency atlases presently used by Reclamation are woefully outdated, with the far majority of the previous studies dating back 30 to 40 years ago, and lack extensions to important meteorological parameters (duration, area, return period, etc.). This information is used in establishing hydrologic design criteria for the safety evaluation of water control structures (dams, canals, levees, culverts, etc.), design of other types of construction (roads, bridges, flood warning systems, etc.), and for establishing project operational criteria. Results of this work will provide consistent precipitation-frequency information that can be incorporated in risk assessments that are used in current/future flood hydrology studies.

The project is a cooperative effort among several federal, state, and local agencies involved in water resource management. The National Weather Service (NWS), Hydrometeorological Design Studies Center (HDSC) is the lead agency for accomplishing the work with participation from other agencies (financial, in-kind-services) dependent on their interest/needs to provide support for the particular region under investigation. Because of the large amounts of data to process and the need to test new meteorological and statistical analysis techniques, the United States and its Possessions were broken into nine separate zones. Presently, four of these zones are under development in varying degrees. These zones include: Semiarid Southwest, the Ohio River Basin and Surrounding States, Hawaii, and Puerto Rico and the Virgin Islands. Work on a fifth study region, the Upper Midwest, has been started by Reclamation to assemble maximum daily precipitation (prior to 1949) data. For the next couple of years, Reclamation expects to continue with data set development for the Upper Midwest, but concentrate its effort on development of Depth-Area (DA) relationships used to adjust point precipitation-frequency values to representative areal average precipitation-frequency estimates.

The Semiarid Southwest, Ohio River Basin and Surrounding States, and Puerto Rico and the Virgin Islands zones are scheduled for completion in 2002. Hawaii is scheduled for completion in 2003. Current updates/progress reports concerning all work underway is available at: http://www.nws.noaa.gov/oh/hdsc. The precipitation-frequency work as completed will be published as NOAA Atlas 14, Volume (#). It is expected that the entire Atlas will be completed by 2006.

Probabilistic Flood Hydrographs

Flood runoff hydrographs integrate the drainage basin and channel response to precipitation and snowmelt, given some initial, variable state of moisture throughout the watershed. Probabilistic flood hydrographs are developed to assess the adequacy of the spillway and reservoir flood/surcharge space to temporarily store a portion of the flood volume, and to attenuate or pass the hydrograph peak without overtopping the dam. Flood hydrographs are needed for situations where: the reservoir inflow peak discharge is greater than the maximum spillway capacity; the reservoir has a large, carry-over storage; and/or the reservoir has dedicated flood control space. The focus of this research is to develop a simplified approach for estimating probabilistic hydrographs that can be used for appraisal or feasibility level studies, and to develop a simplified method of extrapolating flood frequency curves.

Basic streamflow hydrograph methods (e.g., Chow et al., 1988; Bras, 1990) are used to estimate properties for probabilistic hydrographs. These methods include peak and one-day mean discharge identification, selection of hydrograph shape and duration, base flow identification and separation, and direct runoff volume estimation. Peak discharge and mean-daily streamflow records are used because this source is the best information on flood magnitudes that are likely to occur in the future, based on what occurred in the past (Pilgrim and Cordery, 1993).

The key idea is calibration or scaling of hydrographs to match a particular peak discharge for a given probability. The approach relies completely upon the specification of a peak flow frequency curve that describes the probabilities of interest. Peak discharge estimates, n-day maximum mean flows, and observed hydrographs at the site of interest are used as a sample to represent potential extreme flood shapes and volumes. The largest peak and volume hydrographs are utilized as a basis to scale.

There are five major assumptions for developing the hydrographs: (1) the probability of peak discharge is sufficient to represent a probability of the composite hydrograph; (2) unit hydrograph (e.g., linearity) assumptions apply to the basin; (3) direct runoff volumes can be estimated from daily flow hydrographs; (4) peak discharge - maximum mean n-day flow relationships can be extrapolated; and (5) the recorded streamflow observations, historical information, and paleoflood data provide an adequate sample to base extrapolations to extreme floods.

The anticipated completion date for this project is early 2002.

Improved Flood Frequency Extrapolations and Runoff Modeling

The purpose of this research project is to develop improved methods to extrapolate flood frequency curves and develop extreme flood hydrographs. The major approach to flood frequency extrapolation will be based on a combination of rainfall extrapolation and derivation from physically based runoff mechanisms. Rainfall-runoff models will be used to derive the peak discharge frequency distribution from input basin characteristics and precipitation, and be used as the basis for frequency curve extrapolation. The CASC2D rainfall-runoff model will be evaluated and tested for application at Reclamation sites, and compared with a stochastic event runoff model (SEFM) developed by Dr. Melvin Schaefer for Reclamation. CASC2D is a 2-dimensional, distributed rainfall-runoff model that has successfully reproduced the 1997 Fort Collins flood. The main precipitation and stochastic components used in SEFM will be added to CASC2D. It is anticipated that model selection and extrapolation functions can be derived from the watershed topography, hydraulic routing characteristics, and precipitation characteristics at Reclamation dams. Input rainfall will be derived from frequency analysis or from stochastic storm generation. Flood frequency and hydrograph uncertainty bounds will be approximated by simulation. Models will be compared on a large (>500 mi^2) basin where paleoflood data are available.

Progress has been made in developing hydrograph-scaling techniques for appraisal and feasibility studies that require low effort and expense. These techniques have been applied to several projects such as Pineview/Deer Creek, Red Willow, North Platte, and Folsom Dams. Internal and external reviewers have pointed out several shortcomings of that work including, assumptions of linear runoff and extrapolation, use of observed hydrographs, failure to separate rainfall and snowmelt, and the challenges of using the techniques at larger basins (greater than about 500 mi^2). This research project attempts to address many of these concerns.

This research can be applied to Dam Safety projects where flood peaks and hydrographs are needed with return periods that exceed 1,000 years. The extrapolation research can be applied to sites where loss of life is large, as floods with return periods greater than 10,000 years are sometimes needed. The research will be span three fiscal years and conclude in 2004.

Rainfall-Runoff Modeling Using National Weather Service 1,000-year Return Period Precipitation Estimates at Causey Dam, Utah

Recently, the Flood Hydrology Group completed a study where the frequency estimates were extrapolated to a return period of 200,000 years. The method used a two-point extrapolation to 200,000 years using the mean of the gage data and the mean of the paleoflood range. This was the first attempt to extrapolate frequency data beyond a 10,000-year return period and several assumptions were made that may not be accurate. For example, the stream gage data in the Wasatch Range are dominated by snowmelt events, yet the distribution selected in the study was based on rainfall distributions. The

intention of this study is to provide another independent data point and to verify the conclusions reached in the previous analysis.

This investigation will use the draft precipitation values developed by the National Weather Service (NOAA Atlas 14, Vol 1, DRAFT) to produce a 1,000-year assumed thunderstorm event at Causey Dam, Utah. These values will be input to a rainfall-runoff model (HEC-1 or FHAR) to develop the 1,000-year thunderstorm flood event for Causey Dam. This peak discharge estimate will be compared with the frequency analysis developed for Causey Dam. Causey Dam was selected as the test case because there is existing at site paleoflood data to use in comparison, the drainage area is relatively small (137 mi^2), and it has a significant amount of streamflow and other comparative data developed. The estimated completion date would be June 2001.

Probabilistic Flood Hazard Workshop

The introduction of risk analysis for dam safety signaled a significant change in the way the Dam Safety Office and the Technical Service Center conduct flood hazard assessments. The purpose of this project is to compile, review, and evaluate current state-of-the-knowledge on probabilistic techniques used in flood hazard assessment. External experts in various aspects of flood hazards will be brought in to Reclamation on an individual basis. It is intended that each person present a Technical Update Lecture (1-2 hours). Members of the Flood Hydrology Group will subsequently meet with them to discuss their research in detail and potential technology transfer to Reclamation.

About 12 experts participated in the workshop last year. These experts have helped the Flood Hydrology Group map out future methods, improve current methods, and plan a program for probabilistic flood hazard analysis to meet Dam Safety Office needs.

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GUIDELINES FOR ACHIEVING PUBLIC PROTECTION IN DAM SAFETY DECISIONMAKING

UNITED STATES DEPARTMENT OF THE INTERIOR
Bureau of Reclamation
Denver, Colorado

June 15, 2003

(Replaces Interim Guidelines dated April 4, 1997)

TABLE OF CONTENTS

I. Introduction …….….1

II. Risk Assessment Framework for Dam Safety Decisionmaking …….…...……. 3
- A. Background …….3
- B. Terminology ……4
- C. Risk Framework ……..5
- D. Decisionmaking ……...6

III. Public Protection (Risk Evaluation) Guidelines ……8
- A. Evidence of a Developing Failure Mode ……8
- B. Annual Probability of Failure ……8
- C. Estimated Risk ……9

IV. Determining Appropriate Actions ……..…….……..12
- A. Development and Presentation of Risk Estimates ……..12
- B. Assessment of Risk Results ……....14
- C. Large Downstream Populations …….....16
- D. Examples of Alternative Actions ……...……16
- E. Formulation of Appropriate Risk Reduction Alternatives ……..……18
- F. Unrecognized Risks ……...……..18

I. Introduction

Purpose of Guidelines - The Bureau of Reclamation (Reclamation) is responsible for about 370 storage dams and dikes that form a significant part of the water resources infrastructure for the western United States. As the owner of these facilities, Reclamation is committed to providing the public and the environment with adequate protection from the risks which are inherent to collecting and storing large volumes of water for later distribution and/or release. This document presents:

- The basis and guidance for a risk-based approach to decisionmaking
- Guidelines for evaluating risks at Reclamation dams
- Guidelines for developing and presenting the risk estimates
- Guidelines for interpreting/assessing the risk results
- Example actions that can be taken to address risk at dams
- Guidelines for maintaining a focus on risk reduction when implementing agency actions

The guidelines are intended to ensure adequate and consistent levels of public protection when evaluating and modifying existing dams and appurtenant structures and when designing new dams and/or structures.

Considering a Full Range of Loading Conditions - Historical design and analysis methods have focused on selecting a level of protection based on loadings from extreme events and conditions. These extreme events comprise the upper bound of loadings considered to be reasonably probable. The civil engineering profession generally agrees that dams and dikes designed to withstand extreme loadings meet an acceptable standard of public safety. In addition to ensuring public safety for extreme events, Reclamation also is committed to providing public safety for smaller events and loading conditions, which occur more frequently. For example, an enlarged spillway designed for a probable maximum flood loading condition may increase the risks to the public for lesser events. Risk assessment provides a framework for addressing the most effective way to provide public protection over the full range of loading conditions.

Need for Probabilistic Methods - As a water resources management agency, Reclamation strives to provide decisionmakers with pertinent information that is founded upon current or emerging water resources management and public safety practices. Over the past decade, there has been an increasing trend in water resources analysis toward using probabilistic design methods to evaluate the effectiveness of expending funds for enhancing public safety. There has also been greater recognition that even the most restrictive design standards result in some likelihood of failure even though the likelihood may be very small.

Application - This document addresses the incorporation of risk-based evaluations into Reclamation's dam safety decisionmaking process to help assess public risks and allocate resources. While there are many issues that may be evaluated in a risk context, this document focuses on the life loss and the public trust components of decisionmaking. Similar applications of risk-based analysis techniques may be used to address economic consequences within the

framework of the *Principles and Guidelines* for water resources planning.[1] Risk-based analysis may also be used to evaluate environmental and social issues in accordance with the National Environmental Policy Act (NEPA) by addressing the likelihood of the possible outcomes that may result from the various loads that a dam experiences. The implementation of risk-based analysis should consider both usefulness and cost effectiveness in its use.

[1] Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies from the Water Resources Council, March 10, 1983.

II. Risk Assessment Framework for Dam Safety Decisionmaking

A. Background

The mission of the Reclamation Dam Safety Program is:

> "To ensure that Reclamation facilities do not present unreasonable risks to the public, public safety, property, and/or the environment."

The Dam Safety Program is authorized under the Reclamation Safety of Dams Act of 1978.[2] This Act was passed in response to several dam failures in the 1960's and 1970's, including the failure of Teton Dam, a large Reclamation storage dam. The Act provides for action to be taken when it is determined that a structure presents an unacceptable risk:

> "In order to preserve the structural safety of Bureau of Reclamation dams and related facilities, the Secretary of the Interior is authorized to perform such modifications as he determines to be reasonably required."

To determine the risks associated with its structures, Reclamation has established procedures to analyze data and assess the condition of its structures. Prior to the failure of Teton Dam, consideration of dam safety issues was addressed though periodic examinations and project specific requests for Congressional funding to make necessary modifications to dams. The failure of Teton Dam demonstrated a need for a more comprehensive approach to evaluating and addressing dam safety issues.

In 1979, a committee of Federal agency representatives commissioned by the President developed the *Federal Guidelines for Dam Safety* to promote prudent and reasonable dam safety practices among Federal agencies. While the Federal Guidelines recognized that risk-based analysis was a recent addition to the tools available for assessing dam safety, they encouraged Agencies to conduct research to refine and improve the techniques necessary to apply risk-based analysis to dam safety issues:

> "The agencies should individually and cooperatively support research and development of risk-based analysis and methodologies as related to the safety of dams. This research should be directed especially to the fields of hydrology, earthquake hazard, and potential for dam failure. Existing agency work in these fields should be continued and expanded more specifically into developing risk concepts useful in evaluating safety issues."[3]

Reclamation has established a risk-based framework to meet the objectives of its program, the Dam Safety Act, and the Federal Guidelines. Risk-based procedures are used to assess the safety of Reclamation structures, to aid in making decisions to protect

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[2] The Reclamation Safety of Dams Act of 1978, Public Law 95-578.

[3] Federal Guidelines for Dam Safety, Ad Hoc Interagency on Dam Safety, Federal Coordinating Council for Science Engineering and Technology, Washington, D.C., June 25, 1979.

the public from the consequences of dam failure, to assist in prioritizing the allocation of resources, and to support justification for risk reduction actions where needed. Risk assessment for dam safety decisionmaking integrates the analytical methods of risk-based analysis along with the sound professional judgment of engineers, contractors and review boards in determining reasonable actions to minimize risk at Reclamation facilities.

B. Terminology

The following terminology is provided for terms that are used throughout these guidelines for defining the risk-based framework for dam safety decisionmaking:

Risk – The product of the likelihood of an adverse event and the consequences of that event

Failure Mode - A potential failure mode is a physically plausible process for dam failure resulting from an existing inadequacy or defect related to a natural foundation condition, the dam or appurtenant structures design, the construction, the materials incorporated, the operations and maintenance, or aging process, which can lead to an uncontrolled release of the reservoir.

Risk Analysis – A procedure to identify and quantify risks by establishing potential failure modes, providing numerical estimates of the likelihood of an event in a specified time period, and estimating the magnitude of the consequences. The risk analysis should include all potential events that would cause unintentional release of stored water from the reservoir.

Risk Evaluation – The establishment of Reclamation guidelines for agency response to estimated risks.

Risk Assessment – The use of risk estimation for a given dam in the decisionmaking that leads to agency response according to risk evaluation guidelines.

Consequences – Estimated losses that result from an adverse event leading to a dam failure scenario.

Failure Probability, Consequences, and Risk Estimates – The mean values calculated from Monte Carlo or similar analyses that include explicit treatment of input uncertainty. Also, the calculated numerical values when single point estimates are used in the calculations and the point values are considered reasonable and plausible estimates of the mean rather than extreme values in a range. These estimated mean values are also called the expected values. *(Note: This definition must be applied in order to achieve effective and consistent application of these guidelines.)*

C. Risk Framework

Risk analysis is a tool that enables technical specialists and decisionmakers to better understand possible failure mechanisms and the elements of risk involved in the various issues related to dam safety. It provides an overall picture of risks, the potential impacts of proposed actions, and the resulting costs (economic, social and other). The results of risk analyses can contribute to efficient accomplishment of the dam safety program by quantifying engineering judgments that allow for the evaluation of:

- Factors contributing the greatest risk at a given site,
- The facilities with the greatest risk,
- Identification of additional analyses and/or data collection that are needed to better understand critical uncertainties,
- Anticipated risk reduction effectiveness of alternative courses of action,
- Allocations of dam safety program funds that will contribute the greatest overall risk reductions.

The risk framework consists of several steps leading to agency decisions regarding appropriate actions to be taken to address dam safety risks at Reclamation's high- and significant-hazard dams. These steps are summarized as follows:

Risk Identification – As part of the ongoing dam safety evaluations for each high- and significant-hazard dam, Reclamation identifies the conceivable modes of dam failure. These failure modes are then monitored (through performance monitoring and examinations) for any indication of changes in performance that would be indicative of a dam progressing toward a failure condition or toward a significant risk to the public. If such indications are found, the issue is referred for further evaluation of the estimated risk. If failure modes are deemed likely, action to reduce risk may be taken.

Risk Estimation – Once a dam safety issue has been identified, it is necessary to assess and quantify the risk to the public as information to be used by the decisionmakers. The quantification of risk involves the estimation of the likelihood (probability) of an unintentional release of stored water and an estimation of the consequences resulting from the unintentional release. To facilitate developing the risk estimates, it is frequently convenient to break the estimating process down into three components including: estimating the likelihood of an initiating condition existing or an event occurring, estimating the likelihood of an unintentional release of the reservoir given the event or initiating condition, and estimating the consequences (life loss) given the unintentional release of the reservoir.

Risk Evaluation – Once risks have been estimated for a dam, decisionmakers need a framework for evaluating the risks to determine if action is required to reduce risks. There is currently no commonly accepted industry standard for determining what risks are considered acceptable.

The guidelines portion of this document provides for evaluation of risk by two measures. The first measure, the annual probability of failure, addresses the public's expectation that Reclamation dams should not fail by evaluating the probability of an unintended release of the reservoir. It also addresses the expectation that risk to the most exposed individual will be managed. The second measure addresses the expected value of life loss expressed on an annual basis which combines the annual failure probability estimates with estimates of the expected life loss consequences given a dam failure. The first measure addresses agency and individual risk, while the second measure addresses the life loss component of societal risk.

Risk Reduction Actions – When decisionmakers have determined that a risk reduction action is required, there are usually a number of prudent alternative actions that can be taken. Dam safety decisionmaking involves the selection of an appropriate course of action for a given issue based on the magnitude of the risk, the degree of confidence in (or uncertainties associated with) the estimated risk, and the likelihood of additional information providing a significantly enhanced understanding of the risks associated with the identified issues.

Roles of analysis approaches - Although risk-based and standards-based (design standards, codes or criteria) approaches are often considered to be competing approaches, each have a role in Reclamation's decisionmaking process. Risk assessment is a diagnostic tool used throughout the evaluation, design, and construction process that helps decisionmakers formalize and document dam safety decisions. Standards are used to ensure that the selected corrective actions are well designed and implemented. In other words, risk-based approaches help decision makers choose the appropriate courses of action while standards-based approaches assure sound implementation of those actions.

D. Decisionmaking

Policy - Reclamation policy for dam safety decisionmaking delegates decisionmaking responsibility to the Regional Directors in collaboration with the Chief, Dam Safety Office and the appropriate Area Manager.[4] The Technical Service Center (TSC) staff provides significant technical advice that is critical to decisionmaking. The risk framework serves as a tool for aiding decisionmakers in the determination of needs for risk reduction actions as well as the evaluation of different risk reduction actions that could be taken to address the identified issues.

Public Trust Responsibility - Decisionmaking to accomplish the Dam Safety Program is complex and must consider risk to the public as well as economic, environmental, and cultural impacts. Thus, it is difficult to be prescriptive when developing guidance for making decisions. While the technical analysis of risks associated with a dam can not become the sole decisionmaking factor, it must be recognized that addressing these risks in a technically consistent and timely fashion is an important part of sustaining the public's trust in Reclamation to manage these facilities in the best interest of the nation.

[4] Decisions Related to Dam Safety Issues, Reclamation Manual / Policy FAC P02, Bureau of Reclamation, Denver, Colorado, June 23, 1998.

This public trust responsibility includes operating Reclamation facilities with reasonable assurance of the safety of persons in the vicinity of and downstream of the dams.

Process - Dam safety decisionmaking is similar to many other aspects of water resources management in that decisions regarding reasonable courses of action are not always initially agreed upon by all stakeholders. The most important part of the decisionmaking process is recognizing that it will generally involve building consensus regarding the appropriate actions to be taken. However, in the event of an emergency, the time for developing consensus may be severely shortened or nonexistent. Such a situation would require the Regional Director to act quickly to avoid or minimize consequences.

III. Public Protection (Risk Evaluation) Guidelines

Measures of Risk - These guidelines focus on two assessment measures of risks related to Reclamation structures: 1) the probability of a dam failure and 2) the life loss consequences resulting from the unintentional release. The annual probability of failure guideline addresses agency exposure to dam failure. As a water resource provider, Reclamation must maintain and protect its dams and dikes that store water. The second measure addresses the life loss component of societal risk. Protection of human life is of primary importance to public agencies constructing, maintaining, or regulating civil works.

Risk Analysis Methods - Reclamation's risk analysis process involves the development of event trees that identify all of the known and potential events, states of nature (existing conditions, site characterization, etc), dam responses, exposure conditions, and consequences. The overall risk from the facility is defined as the accumulation of all risks associated with each of the possible paths through the event trees. The methods to analyze the risks associated with annual dam failure probability and life loss are briefly described in the following two sections. Additional information on the methodology for performing risk analysis can be found in *"Dam Safety Risk Analysis Methodology*."[5]

Potential Applications - Although these guidelines focus on life loss as a dam failure consequence, other consequences, such as environmental and economic consequences, may be applied on specific projects where the decisionmaking process would be enhanced by presentation of the entire breadth of consequences and risks. Economic and/or environmental risk assessment may be performed when the potential for life loss does not provide sufficient or appropriate input for a decision regarding modification of a structure.

A. Evidence of a Developing Failure Mode

If there is evidence of a developing failure mode, there is a clear need to take action to reduce risk. These situations should be brought to the immediate attention of the dam safety decisionmakers to assure a timely response by the agency. Once the evidence is determined to be credible, efforts should focus on those risk reduction actions that can be taken to quickly reduce the potential for life loss or an unintended release of the reservoir regardless of any risk estimates.

B. Annual Probability of Failure (Previously Tier 2)

Measurement Purpose - To manage an effective Dam Safety Program on behalf of the Federal government and to assure public confidence in the performance of public works, dam failures and associated large consequences need to be avoided. A high level of national safety and stewardship of public assets is expected of Reclamation as an agency

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[5] Dam Safety Risk Analysis Methodology, Bureau of Reclamation, Denver, Colorado, Version 3.3, September 1999.

specifically entrusted to manage a large inventory of dams. Unintended release of the reservoir can cause significant downstream damage and disruption to routine activities. Once an unintended reservoir release occurs, public trust is compromised and public expectations may impose severe and costly constraints on projects. The greater the inventory of dams and the time of exposure, the more difficult it becomes to ensure that the agency will not experience a dam failure.

Measurement Definition - For comparison to this guideline, the annual probability of failure is defined as the probability of a structural failure or condition that results in an unintentional release of the reservoir that would be expected to result in loss of life. The annual probability of failure is totaled for all specific loading conditions (seismic, static, hydrologic, improper operation, etc.) The probability of events that are not expected to cause life loss are not included, even though there may be some unintended loss of reservoir storage. For example, if a structure accommodates large flows through rockfill without breaching and without causing life loss, then the flow condition would not be included in the probability of failure calculation. Events or conditions that can result in an unintentional reservoir release are referred to as failure modes. These include failure due to loadings from normal and extreme events.

Guideline - To ensure a responsible performance level across the inventory of Reclamation Dams, it is recommended that decisionmakers consider taking action to reduce risk if the estimate of annual failure probability exceeds 1 chance in 10,000. Table 1 provides guidelines to evaluate the need and urgency to implement risk reduction activities based on the annual failure probability estimates:

Table 1. – Guidelines to evaluate Annual Probability of Failure Estimates	
Estimates for annual probability of failure > 0. 0001	The justification to implement risk reduction actions increases as the estimates become greater than .0001. Actions considered reasonable and prudent should be considered for implementation when the annual probability of failure estimate is in this range. A variety of possible actions may be appropriate (see Section IV.D).
Estimates for annual probability of failure < 0. 0001	The justification to implement risk reduction actions diminishes as the estimates become smaller than .0001. Risk reduction action costs, uncertainties in the risk estimates, scope of consequences, operational and other water resources management issues play an increased role in decisionmaking. Actions considered reasonable and prudent should be considered for implementation when the annual probability of failure is in this range.

C. Estimated Risk (Annualized life loss - Previously Tier 1)

Measurement Purpose - Reclamation's primary dam safety concern is to ensure that its structures do not cause life loss. The estimated risk is calculated for each specific loading category (seismic, static, hydrologic, improper operation, etc.) at a dam based on the estimated life loss from dam failure.

Measurement Definition – For dam safety decisionmaking, risk of life loss is measured as the product of the probability of dam failure and the consequences (life loss) associated with that failure. This product is the expected annualized life loss at a given dam for a given loading condition and is referred to as the estimated risk of life loss.

Guidelines - Table 2 provides guidelines to evaluate the need and urgency to implement risk reduction activities based on the estimated risk:

Table 2. – Guidance for Estimated Risk	
Estimated risk is portrayed to be >. 01 lives/year	Reclamation considers that there is justification for taking expedited action to reduce risk. While there is a full range of possible risk reduction actions that can be taken (see section IV.D), Reclamation should focus on those that can quickly reduce risk or improve understanding of the uncertainties associated with the risk. As confidence increases that the risk is in this range, actions considered should concentrate more on reducing the risk than reducing the uncertainties. Any reassessment of the risk should be done prior to increased storage if at all possible, and every effort should be made to complete the reassessment within 90 days of determining the need for expedited risk reduction action.
Estimated risk is portrayed between .01 and .001 lives/year	Reclamation considers that there is justification for taking action to reduce risk. When the range of risk estimates falls in this range, there are a wide variety of possible actions which may be appropriate. However, the actions can be scheduled into the dam safety program and coordinated with other needs at the facility or at other facilities. Actions to reduce risks should be implemented on a schedule that is consistent with budgeting and appropriations processes. Typically, risk reduction should be accomplished within 7 years of a decision that risks need to be reduced. When there is an indicated need for risk reduction, the time spent on additional loading definition, data collection, and risk assessment should be completed in a reasonable timeframe. While it is desirable for this timeframe to be within a year, other times may be considered reasonable by decisionmakers based on the severity of the identified risks. Decisions on adequate time frames should be documented in appropriate decision documents.
Estimated risk is portrayed to be < .001 lives/year	The justification to implement risk reduction actions or conduct additional studies diminishes as estimated risks become smaller than .001. Risk reduction action costs, uncertainties in the risk estimates, scope of consequences, operational and other water resources management issues play an increased role in decisionmaking. Actions considered reasonable and prudent should be considered for implementation when the risk is in this range.

Risk to Small Populations - When life loss estimates are low (less than 10) for a given

loading category, a threshold estimated risk of .001 can potentially expose a small population to failure events with relatively high probabilities. Risk to an individual from dam failure for these cases may be similar to other societal risks such as auto accidents and disease. Accordingly, risks associated with a Reclamation storage facility could contribute significantly to the life risks of an individual in the exposed population. In these cases, the guidelines related to annual probability of failure (section IV.A) serve as an upper limit of exposure to such small populations.

IV. Determining Appropriate Actions

A. Development and Presentation of Risk Estimates

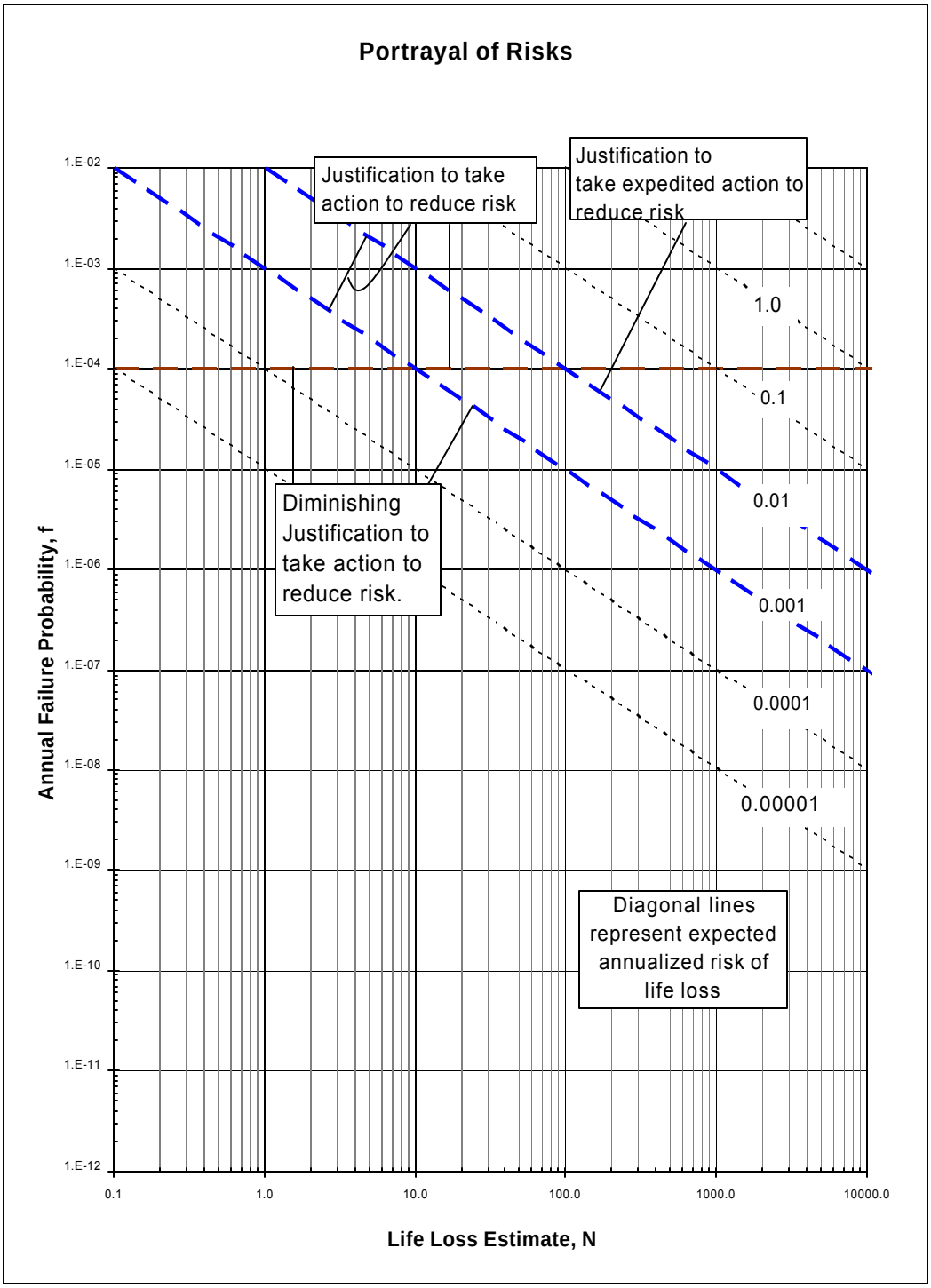
Use of Risk Estimates - Risk analysis provides a means to quantify judgment and to identify the parameters that contribute to risk at a site. The intent of a risk assessment is to review the failure modes for a dam, to decompose the failure modes into separate events, to assign probabilities to the events, and to provide a range of risk estimates so that risks can be compared to these guidelines. Valuable outcomes of the risk assessment include an improved understanding of the critical issues at a dam and a clearer identification of the issues that are the most significant contributors to risk. This knowledge can be used to focus attention on those issues, which, if mitigated, will provide the greatest reduction of risk to the public.

CFR Risk Estimates - Since the risk estimating process during the Comprehensive Facility Review (CFR) is not a detailed team effort, it may have a higher level of uncertainty than an issue evaluation risk analysis. The results of a CFR risk assessment should be presented as the mean estimate or the expected value of risk. If the senior engineer feels that significant assumptions need to be made, resulting in more than one possible scenario to be considered, a range of risk estimates may be presented. If a range of estimates is provided, the CFR must clearly state the specific assumptions or reasons that form the basis of the range of estimates. If the risk estimate is presented as a single point, decision makers should be cognizant of the fact that the estimate actually has a degree of uncertainty associated with it.

Issue Evaluation Risk Estimates - Detailed team risk analyses should also address the uncertainty associated with the risk estimate. Typically in issue evaluation team risk analyses, probability density functions are assigned to the estimates in an event tree. Techniques such as Monte Carlo simulation can be used to show the variability in a number of trials that sample the assigned probability density functions. The risk estimate is defined to be the arithmetic mean of the values computed for all trials. Sensitivity studies may be performed by assigning other reasonable density functions and noting the change in both the variability of the trial estimates and the calculated mean risk estimate. Such sensitivity studies provide the decisionmakers with an estimate of a range of the risk estimate based on the risk model used by the team. If the scatter plots of the Monte Carlo calculation trials are presented, it should be carefully explained that the individual points are not risk estimates as defined in Section II.B. These scatter plots may be useful in analysis and may help communicate the key factors influencing the risk estimates.

Displaying Risk Estimates - The range of risk estimates (annual probability of dam failure and expected annual life loss) should be presented for each load category on an f-N diagram as shown in Figure 1. The f-N diagram illustrates the probability of dam failure, the potential consequences, and the expected annual life loss risk associated with a given load category on one diagram. The guidelines for considering risk reduction action are illustrated as dashed bold lines on the f-N diagram.

Figure 1. - The f-N Chart for Displaying Probability of Failure, Life Loss, and Risk Estimates



Communicating the Basis of Risk Estimates - It is important to note that the key objective of the risk analysis is to communicate the current understanding of risk to the decisionmakers. Decisions will be facilitated by elaboration on the reasons the risk might be higher or lower and the additional information that might better define the risk. For example, the range of risk estimates might not be continuous if there is an important lack of information or if there are alternate interpretations of the available information about a structure. There could be one range of risk estimates that is high and another that is low with the difference being the assumption about the information that is lacking or the interpretation of the available information. This sort of risk estimate communication can be very useful to the decisionmakers when proposals for gathering additional data or for more detailed technical analysis are considered.

B. Assessment (Use and Interpretation) of Risk Results

Action Based Decisionmaking - Dam Safety issues may be identified as Reclamation operates, maintains, monitors, inspects, or analyzes a structure. When issues arise, further data collection, investigation, and related analysis may be required to better understand the public safety or economic implications. Reclamation will address the identified issues by taking an action, prioritizing and scheduling an action, or by documenting a decision that no action is necessary. In general, many issues are raised without implications on continued operation of the facility. The dam safety decisionmakers should consider the potential severity of issues being addressed in the context of the dam safety program objectives and determine if continued normal operation of the facility is appropriate. If a decision is made to continue normal operations while issues are being addressed, then that decision should be documented.

Prioritization - Reclamation has limited financial resources available to address issues. It is critical to not only identify future actions but also to identify the priority or the time frame associated with these actions. The priority for initiating actions to address risks depends in part on available resources and on the risks throughout Reclamation's dam inventory. The intent is to make the greatest reduction in risk throughout the inventory of Reclamation dams within the resource limitations of the program while at the same time assuring that no dam presents an unreasonable risk.

Uncertainty - The quantification of risk estimates is dependent on data and analysis regarding the design, construction, and current condition of a dam, as well as the identified loads that the dam could be subjected to over its operating life. All of this information has some level of uncertainty associated with it. It is acknowledged that the quantification of risk estimates is subjective and is a function of group dynamics, the experience and associated judgment of group members, and the available information for a dam. Thus, uncertainty in the risk estimates is expected. As a consequence, there can be a range of actions that may be suggested for a given range of risk estimates.

Assessing Ability to Reduce Uncertainty - When making a decision regarding future actions, one should consider the risk estimates, the issues most influencing the risks, the

sensitivity of the risks to particular inputs, the cost of additional actions, and the potential for reducing uncertainty. Uncertainty may be reduced by performing additional actions such as collecting more data, by performing more analysis, or by performing a more detailed analysis of the risks. However, there are occasions when additional efforts may not result in significant reduction in uncertainty. It is important to recognize when this is the case and consider the anticipated value of the additional efforts to reduce uncertainty as a factor in selecting a course of action.

Risk Estimate Ranges (range of means) Straddling the Guidelines - In gathering the information necessary for dam safety decisionmaking, the decisionmaker will never have complete or perfect data on which to base the decision. Accordingly, there is some degree of uncertainty in the risk estimates for each dam. When significant uncertainties or assumptions related to a lack of data or interpretations of data result in a range of risk estimates, the results may straddle the guideline values with portions of the risk estimates range portrayed both above and below the guidelines. In these cases, it is important for decisionmakers to assess the portion of the risk estimate range that exceeds the guidelines to determine if it is significant enough to warrant further action or studies. The entire range should be used to assess the need for future actions as well as an aid in setting the priority for initiating the actions. If the range extends into the zone that justifies expedited risk reduction, studies to better define the risk should be the minimum response of the agency.

Level of Analysis Considerations - Because CFR analyses are not detailed team efforts, decisions based on CFR-based risk assessments are typically related to improvements in monitoring, collection of additional data, or performance of additional analyses to reduce uncertainty or improve confidence in the risk estimates. Decisions to change operations or initiate modifications are generally not made as a result of these analyses.
Issue evaluation risk analyses are more extensive analyses of risk and draw on a broader range of expertise. These analyses may require additional data collection, additional analyses, and include a more detailed breakdown and analysis of risks. Risk estimates developed during this activity are often computed using a Monte Carlo simulation and should include sensitivity studies to determine a potential range for the risk estimate.

Risk Reduction Objective - It is important to reduce risk as low as can reasonably be achieved if it is decided to pursue a risk reduction action. As a result, it is desirable to lower the entire range within which the risk estimate would be expected to fall given the uncertainties. An evaluation of the effect of modification alternatives on the range of the risk estimate will enter into the selection of the preferred alternative. In other words, selection of a preferred alternative should focus on moving the range of the risk estimate sufficiently below the guidelines to assure that the dam safety issue doesn't resurface due to slight differences in interpretations of the risk.

Consideration of Future Developments - Future growth in the downstream flood plain, increases in the loading estimates, and changes in the state-of-the-art, may result in increases in risk estimates. Thus, the more risk reduction achieved, the less likely it becomes that future studies will conclude that the risks no longer meet Reclamations guidelines. Risk reduction goals should be considered on a cost versus risk reduction

basis. Ideally, a menu of options, associated costs, and impacts on risk should be considered by the decisionmakers so that prudent decisions can be made.

C. Large Downstream Populations

When the probability of a given loading category is relatively high and there is high potential for downstream life loss, a very low probability of unintended release is required by these guidelines. In such cases, Reclamation focuses on ensuring that there are sufficient protective (defensive design) measures incorporated into the structure. These protective measures either increase confidence in the structure's ability to perform satisfactorily without unintended releases, or increase confidence in Reclamation's ability to detect adverse performance with sufficient lead time to intervene and either prevent an unintended release or provide adequate warning to the public.

In some cases, risk reduction actions may be taken to increase confidence in the performance of the structure even though the dam shows no significant signs of adverse performance. In these cases, decisionmakers should work with the technical experts to ensure that there are sufficient redundancies in the design and operations of the facility to instill confidence in the future performance of the structure.

D. Examples of Alternative Actions

With increased justification for action, there is a need to propose alternative actions that will adequately address the risk and/or probability of failure at the dam. It is important to recognize that there is a broad range of actions that can be taken. These actions can range from further investigations to better understand the uncertainties associated with the risks to decisions to modify structures. In many cases, the chosen course may involve a combination of several actions.

Dam safety decisionmaking generally involves the selection of an appropriate course of action for a given issue based on the magnitude of the risk, the degree of confidence in (or uncertainties associated with) the estimated risk, and the likelihood of additional information providing a significantly enhanced understanding of the issues. The state of knowledge regarding the dam safety issues can lead to a variety of possible actions. While the risks associated with each individual facility pose a unique situation, the following are some of the types of actions which can be taken to either improve Reclamation's understanding of the uncertainties associated with the estimated risk, or to improve confidence in the ability of a structure to perform satisfactorily.

Risk Management Activities:

> **Refine Analyses** – If a risk estimate warrants action primarily due to uncertainties in key elements contributing to the risk, decisionmakers may consider gathering additional information in a timely fashion to assist in quickly reassessing the risk. In pursuing this activity, the decisionmakers should satisfy themselves that there are no immediately developing failure modes. Any expedited reassessment of the risk should be done prior to increased storage if at all possible, and every effort should be made to complete the reassessment within

90 days of determining the need for action. If the reassessment indicates expedited risk reduction action is needed, a decision concerning the risk reduction measures should be made and documented.

Reservoir restrictions - While a reservoir restriction is technically an operational change, it can result in a significant and immediate change in the risk at a dam. The risk reduction results from a reduction in the loading condition and the failure probability. Another benefit is that the reduced storage and reduced head leads to less potential for adverse consequences in the event of poor performance of the structure. However, the loss of storage can have a dramatic impact on water users and the environment. Therefore, consideration of a reservoir restriction requires consideration of both the expected reduction in risk and the certainty of the lost project benefits that accrue from limiting storage in the reservoir.

Increased monitoring - If the risks associated with a failure mode are such that successful intervention would likely be possible or better warning could be provided to local authorities, a potential course of action is to improve Reclamation's ability to detect the existence of the conditions which would be indicative of the failure mode developing (i.e. seepage, deformation, etc.)

Operational changes - In some cases, risks can be reduced at a dam by making changes in the operational and/or maintenance practices at a dam. Examples include establishing minimum gate openings to minimize potential for cavitation, checking gates for drift from their set positions, or alternate procedures for filling reservoirs to lower risks at critical times of the year.

Revised Emergency Action Plan (EAP) - The potential for adverse consequences can be minimized by reviewing the potential failure modes and by developing clear guides to decisionmaking for the types of emergency situations that can be envisioned. Existing EAPs have been developed to detect emergency events based on site specific loading conditions. If a new loading condition or potentially adverse response has been identified for a dam, then the EAP initiating conditions, emergency response levels, expected actions for each response level, and hazard specific appendices can be revised to reflect the current conditions or concerns at the facility. While this course of action will not reduce the probability of an adverse response of the structure, it can help to ensure that people understand the risks at the dam and know how to respond appropriately. This may result in a reduction of the life loss risk.

Loading definition - An important part of understanding risk lies in determining the frequency with which unlikely events affect a dam. In some cases, it is beneficial to gather data that will improve the understanding of the frequency-magnitude relationship of the loading conditions that can potentially lead to failure modes. This information would be used to reanalyze the risks.

Data collection - When there is a lack of knowledge of key properties of a facility, there can be considerable uncertainty in its performance. A prudent

action may be to collect information so the performance of the dam can be better predicted. This additional data would be used to reanalyze the risks.

Structural Modifications - When non-structural actions are not expected to adequately address the risks at a facility, structural modifications to the dam may be considered when additional information will not change the risk outcome. The intent of such modifications is to increase confidence in the satisfactory performance of the structure under the applied loading conditions. Throughout the design and construction process, the risks should be evaluated to assure that design and construction decisions are consistent with the risk reduction objectives.

E. Formulation of Appropriate Risk Reduction Alternatives

Role of Risk Estimates - A key to formulating risk reduction alternatives is using the risk analysis information to assure that proposed alternatives will result in effective risk reduction. When developing the alternatives, the event trees should be reviewed to evaluate which events or conditions are the most significant contributors to the overall risk and/or probability of failure. In some cases, very significant risk reductions can be accomplished by focusing on a specific event or condition. In other cases, with multiple sources of risk, several issues may have to be addressed simultaneously in order to reduce risk and the associated probability of failure to appropriate levels.

Accumulation of Risk Over Time - During a risk reduction action, one should remember that Reclamation's goal is to reduce overall risk. This includes the sum of the risk from before, during, and after a risk reduction action. To minimize this total, it is important to proceed promptly with a risk reduction action when the risk values are high because delay in risk reduction increases the time accumulation of risk. Likewise, it is important to consider risks during construction, because these risks contribute to the accumulation of risk over time. Addressing an annualized potential for dam failure that could be relatively small by incurring a much higher probability of dam failure during the period of time that the dam is being modified may not be appropriate because it raises accumulated risk during the life of the dam to a level higher than would be incurred by not pursuing risk reduction action at all. This factor may influence the choice of modification alternatives and reservoir operations during construction. It should not be used to support a "do nothing'' alternative.

F. Unrecognized Risks

Reclamation recognizes that there will always be a potential for risk associated with unknown conditions at a dam that have not been recognized in the analysis. Therefore, an active examination, monitoring, and evaluation program should be in place to provide a mechanism for early detection of developing and/or potential problems. This early detection information should be used to assess changes in the perceived risks at individual dams, and to prioritize funding for the Dam Safety Program for risk reduction activities. The CFR process provides a framework for assuring that there is a periodic opportunity to reassess risk due to changes in the state-of-the-art of dam design or

changes in dam performance. If no such changes are applicable and no new risks are recognized, then the CFR risk assessment serves as a confirmation of previous risk analyses.