RECLAMATION
Managing Water in the West

Design Standards No. 14

Appurtenant Structures for Dams (Spillways and Outlet Works)

Chapter 2: Hydrologic Considerations
Final: Phase 4

U.S. Department of Interior
Bureau of Reclamation

November 2013

Mission Statements

The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

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

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DS-14(2): Final: Phase 4
November 2013

Chapter 2: Hydrologic Considerations

Foreword

Purpose

The Bureau of Reclamation (Reclamation) design standards present 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 Reclamation facilities in a way that protects the public's health, safety, and welfare; recognizes needs of all stakeholders; and achieves lasting value and functionality necessary for Reclamation facilities. Responsible designers accomplish this goal through compliance with these design standards and all other applicable technical codes, as well as incorporation of the stakeholders' vision and values, that are then reflected in the constructed facilities.

Application of Design Standards

Reclamation design activities, whether performed by Reclamation or by a non-Reclamation entity, must be performed in accordance with established Reclamation design criteria and standards, and approved national design standards, if applicable. Exceptions to this requirement shall be in accordance with provisions of *Reclamation Manual Policy*, Performing Design and Construction Activities, FAC P03.

In addition to these design standards, designers shall integrate sound engineering judgment, applicable national codes and design standards, site-specific technical considerations, and project-specific considerations to ensure suitable designs are produced that protect the public's investment and safety. Designers shall use the most current edition of national codes and design standards consistent with Reclamation design standards. Reclamation design standards may include exceptions to requirements of national codes and design standards.

Proposed Revisions

Reclamation designers should inform the Technical Service Center (TSC), via Reclamation's Design Standards Website notification procedure, of any recommended updates or changes to Reclamation design standards to meet current and/or improved design practices.

Chapter Signature Sheet
Bureau of Reclamation
Technical Service Center

Design Standards No. 14

Appurtenant Structures for Dams (Spillways and Outlet Works)

Chapter 2: Hydrologic Considerations

DS-14(2):[1] Final: Phase 4
November 2013

Design Standards No. 14 is a new document. Chapter 2 of this Design Standard was developed to provide:

- Technical processes used by the Bureau of Reclamation to evaluate and select hydrologic loadings (including Inflow Design Floods [IDF] and construction diversion floods) for modified and new storage and multipurpose dams and for appurtenant structures (spillways and outlet works).

- A list of key technical references used for each major task involved with evaluating and selecting hydrologic loadings.

- Examples of selecting the IDF and construction diversion floods are provided in the appendices.

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

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Chapter 2

Hydrologic Considerations

2.1 Scope

Design Standards No. 14 provides technical guidance concerning the Bureau of Reclamation's (Reclamation) procedures and considerations for analyzing and designing two key types of appurtenant structures associated with storage and multipurpose dams and/or dikes. These appurtenant structures are spillways and outlet works. Chapter 2 provides technical processes for evaluating and selecting hydrologic loadings (floods) for modified and new storage and multipurpose dams and for appurtenant structures (spillways and outlet works). These processes should be followed by Reclamation staff and others involved with analyzing and designing modifications to existing dam, spillway and outlet works or designing new dams, spillways, and outlet works. These processes are used for all design activities such as appraisal, feasibility, and final design levels [1].[2] Specifically, chapter 2 provides methods for sizing dams, spillways, and outlet works based on the selection of an Inflow Design Flood (IDF), along with determining freeboard (above the maximum design reservoir water surface) for dams. Also, chapter 2 provides methods for sizing construction diversion systems based on the identification and/or selection of maximum construction diversion flood levels. It should be stressed that this design standard will not duplicate other existing technical references and, wherever possible, it will reference existing procedures and considerations that should be used for the analysis and design of spillways and outlet works.

Most Reclamation storage and multipurpose dams are classified as significant and high hazard structures (see Section 2.2.3, "Downstream Hazard Classification," in this chapter). Dams with these hazard classifications will typically require the use of quantitative risk analysis methodology to select the design flood loadings. For low hazard dams, the same processes can be used. For additional guidance, the reader is directed to Section 2.4, "Inflow Design Flood," in this chapter.

2.2 Definitions and Concepts

The following definitions and concepts are provided to clarify/explain the terminology used in this chapter. These definitions and concepts are consistent with other technical references used by Reclamation.

[2] Numbers in brackets [ ] indicate a reference list at the end of this chapter.

2.2.1 Floods

Floods are defined as hydrologic events that result in large riverflows and/or reservoir levels exceeding normal levels. For the purpose of this chapter, floods are associated with meteorological conditions and causes, including, but not limited to: spring/summer snowmelt floods; flash floods and thunderstorm floods; rain-on-snow floods; extended-duration rainfall floods; and general storm floods. Types of floods that are considered for design purposes are defined in the following sections. Refer to figures 2.2.1-1 through 2.2.1-4 for general illustrations of the impacts of some flood events that have occurred at Reclamation facilities. Typical flood hydrographs are shown in figure 2.2.1-5 and illustrate flood variables and differences between thunderstorm floods and general storm floods. Flood hydrographs are comprised of three key variables: peak flow, volume, and duration. This multivariate relationship is important when selecting design floods because both peaks and volumes need to be considered. Hydrographs that include ranges of peaks and volumes are typically utilized in assessing the safety of dams and reservoirs.

2.2.1.1 Frequency Floods

Frequency floods are represented by flood hydrographs associated with a specific annual exceedance probability (AEP) or ranges of AEPs. A hydrologic hazard curve relates flood peak and/or volume to AEP [2, 11]. An example peak-flow hydrologic hazard curve with uncertainty is shown in figure 2.2.1.1-1. The AEP assigned to a frequency flood hydrograph may be based on peak or volume. Reclamation uses numerous methods to develop frequency floods; figure 2.2.1.1-2 shows examples of frequency flood hydrographs. These hydrographs are typically provided for both the best estimate and other estimates from a hydrologic hazard curve (peak and/or volume) to represent hydrologic loading uncertainty. For the analysis/design of a given structure, ranges of frequency flood hydrographs are provided for the hydrologic hazard, depending on the site/watershed characteristics, dam/reservoir characteristics, hydrologic hazard method used, level of study, and type of flood (thunderstorm versus general storm). These frequency flood hydrographs and ranges are considered in order to include load (magnitude) uncertainty and the AEP estimate range (based on peak or volume). The maximum frequency flood event magnitude (peak and volume) will not exceed the Probable Maximum Flood (PMF) magnitude, which is considered the maximum hydrologic loading that can reasonably occur at a given site [3].

Figure 2.2.1-1. Example: Flood event (estimated inflow peak of about 89,600 cubic feet per second [ft^3/s]) occurred during construction of a dam safety modification in 1993 (center photograph). This resulted in the existing right abutment service spillway passing about 41,600 ft^3/s (upper left photograph), and the left abutment cellular cofferdam being overtopped, but failure did not occur (lower right photograph). Theodore Roosevelt Dam, Arizona.

Figure 2.2.1-2. Example: Flood event (estimated inflow peak of about 60,000 ft^3/s) occurred in 1964. This resulted in the discharge capacity of the service spillway being exceeded and the gravity arch dam being overtopped by about 3 feet for 20 hours. Gate closures and unsuccessful operations contributed to the overtopping. The dam crest and foundation were modified in 1982 to safely accommodate overtopping for floods greater than the 100-year event. Gibson Dam, Montana.

Figure 2.2.1-3. Example: Flood event (estimated inflow peak of about 298,000 ft^3/s) occurred in 1997. This was one of many significant floods resulting from a weather phenomenon referred to as the "pineapple express." Folsom Dam, California.

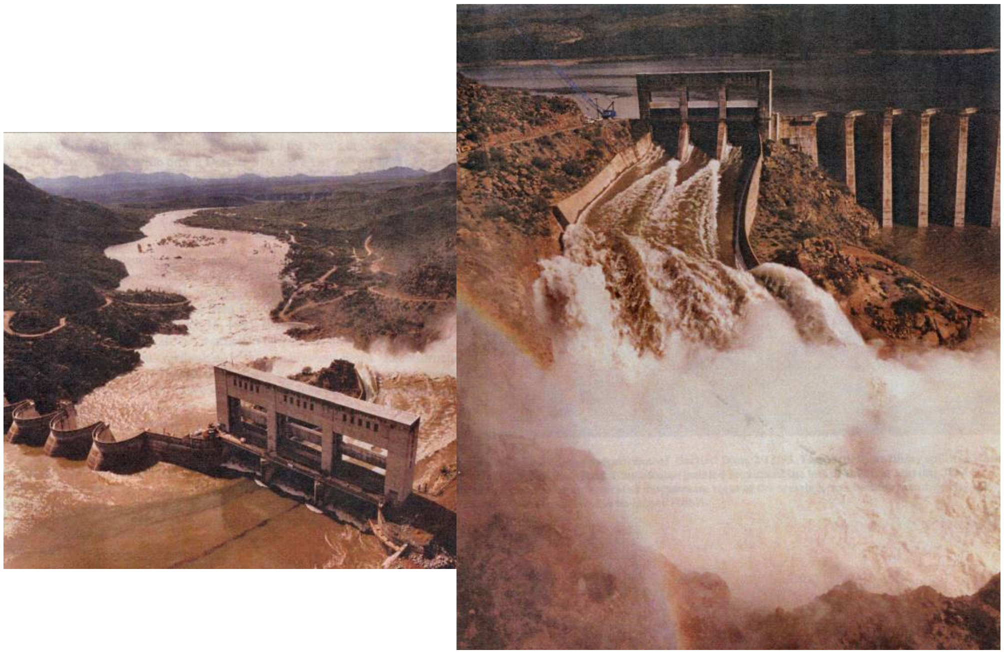

Figure 2.2.1-4. Example: Flood event (estimated inflow peak of about 100,000 ft^3/s) occurred during construction of a dam safety modification in 1995, which swept away cross-river access to construction site. Bartlett Dam, Arizona.

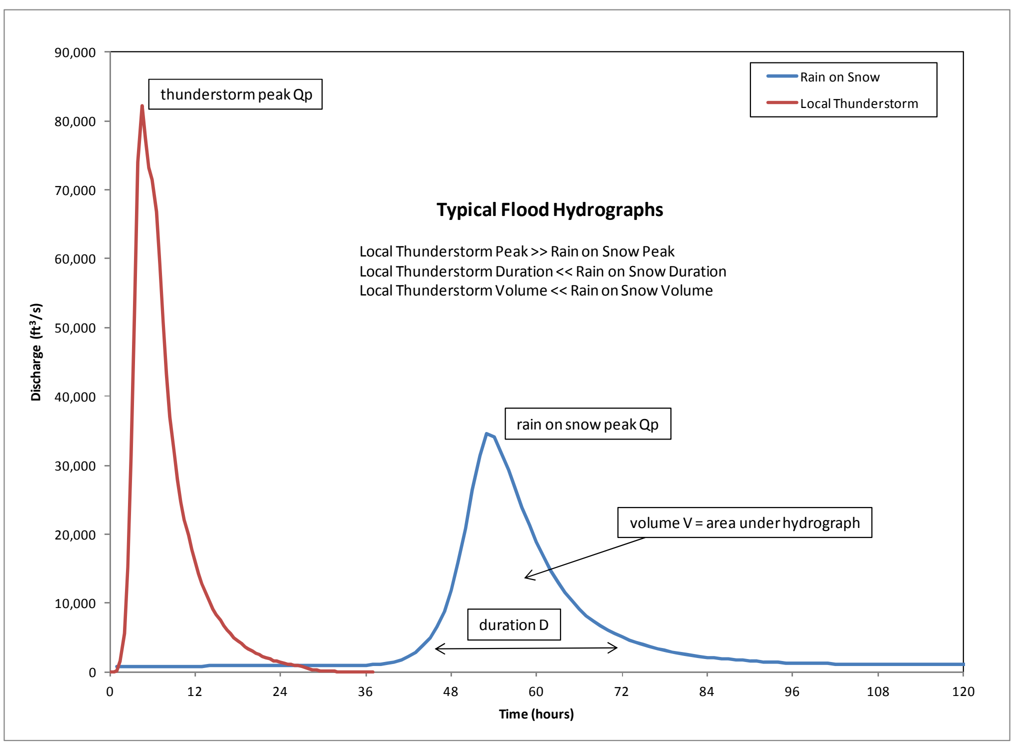

Figure 2.2.1-5. Example: Flood hydrographs showing typical shapes and peaks, volume, and duration relationships.

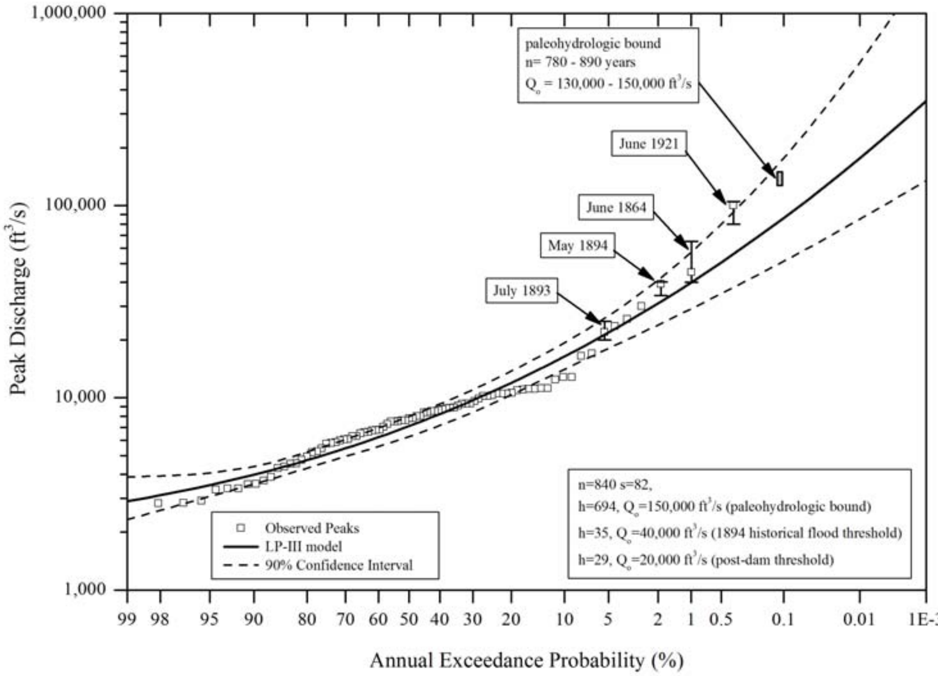

Figure 2.2.1.1-1. Example: Hydrologic hazard curve (with uncertainty) based on peak flows.

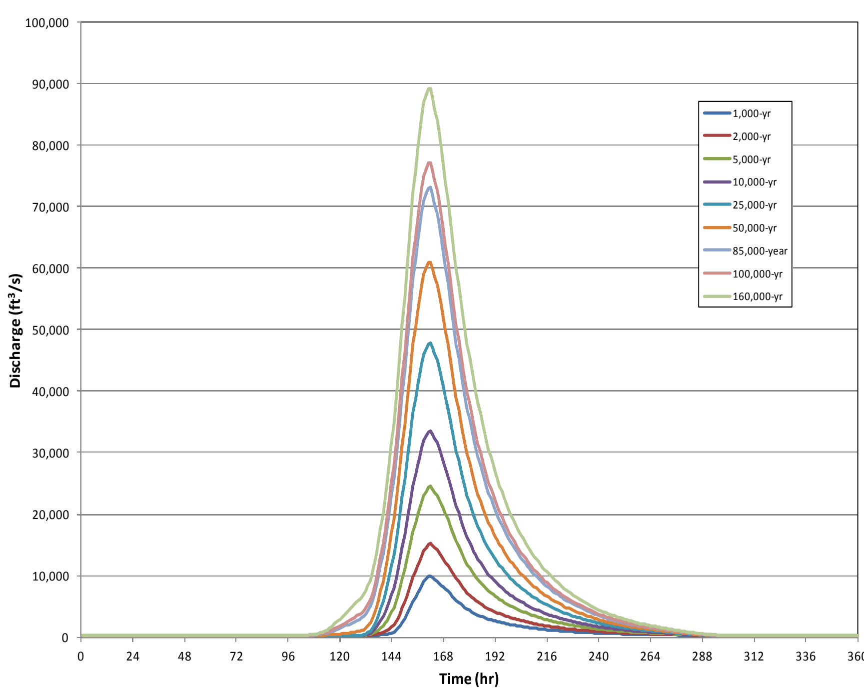

Figure 2.2.1.1-2. Example: Frequency flood hydrographs.

2.2.1.2 Inflow Design Floods

The IDF is defined as the maximum flood hydrograph or a range of flood hydrographs for a given return period, used in the design of a dam and its appurtenant structures, particularly for sizing the dam, spillway, and outlet works. Ranges of IDF hydrographs are considered in order to encompass load uncertainties, and AEP ranges, along with variations in initial reservoir levels, are used in flood routings to determine maximum reservoir elevations. Features are designed to safely accommodate floods up to and including the IDF [4]. The IDF is selected to achieve acceptable levels of hydrologic risk at a dam, which is typically an iterative process as described in Section 2.4.2, "Selection Process," in this chapter. The IDF will be equal to, or smaller than, the PMF. It should be noted that the IDF may not be the design flood loading for all features.

The concept of an IDF is more straightforward when a single flood hydrograph is provided for each return period flood. In this case, the largest of the hydrographs that can be passed is the IDF, and the IDF can be defined by the return period associated with that flood. If the largest flood that can be safely passed is somewhere between two available hydrographs (e.g., the 50,000- and 100,000-year event) for the spillway arrangement and dam crest elevation that provide the desired risk reduction, judgment can be used to approximate the return period of the IDF.

If a more comprehensive flood study is conducted, and hundreds or thousands of hydrographs are generated for each return period flood (through a Monte Carlo simulation), the IDF may become more difficult to define. Judgment may be required to select a representative return period or range of return periods that defines the IDF. Approaches might include selecting or estimating the return period for which all generated hydrographs can be safely passed, or selecting a return period for which a set percentage (e.g., 80 or 90 percent) of the generated hydrographs can be safely passed. Also, rather than identifying a flood return period, a maximum RWS and associated maximum discharge could define the design level. Once the IDF or a design maximum RWS with a design maximum discharge is selected, additional freeboard (robustness study) is evaluated and selected to establish the top of the dam elevation or the top of the parapet wall elevation.

2.2.1.3 Probable Maximum Floods

The PMF is defined as the flood hydrograph that results from the maximum runoff condition due to the most severe combination of hydrologic and meteorological conditions that are considered reasonably possible for the drainage basin under study [6]. The PMF is considered the largest flood event that can reasonably occur at a given site. The PMF is used as the upper limit for extrapolation of hydrologic hazard relationships [11] and is the largest flood that would be considered for design purposes [3]. Refer to figure 2.2.1.3-1 for an illustration of a Hydrometeorological Report (HMR) series that is used to estimate Probable Maximum Precipitation (PMP) and subsequent PMFs for many

Reclamation (and other) dams in the United States. It should be noted that more than one type of PMF can occur at a given dam site, such as rain-on-snow, thunderstorm, etc. (see figure 2.2.1-5), which leads to an important concept: the critical PMF. This flood event is defined as the PMF that would typically result in the highest maximum reservoir water surface (RWS) above other PMF-induced maximum RWSs, for evaluating overtopping or other high reservoir-induced potential failure modes (PFMs). There is no return period associated with the PMF; however, Reclamation estimates frequency floods that have a similar size (peak inflow and/or volume) to the PMF size. Such a frequency flood is used in quantitative risk analysis.

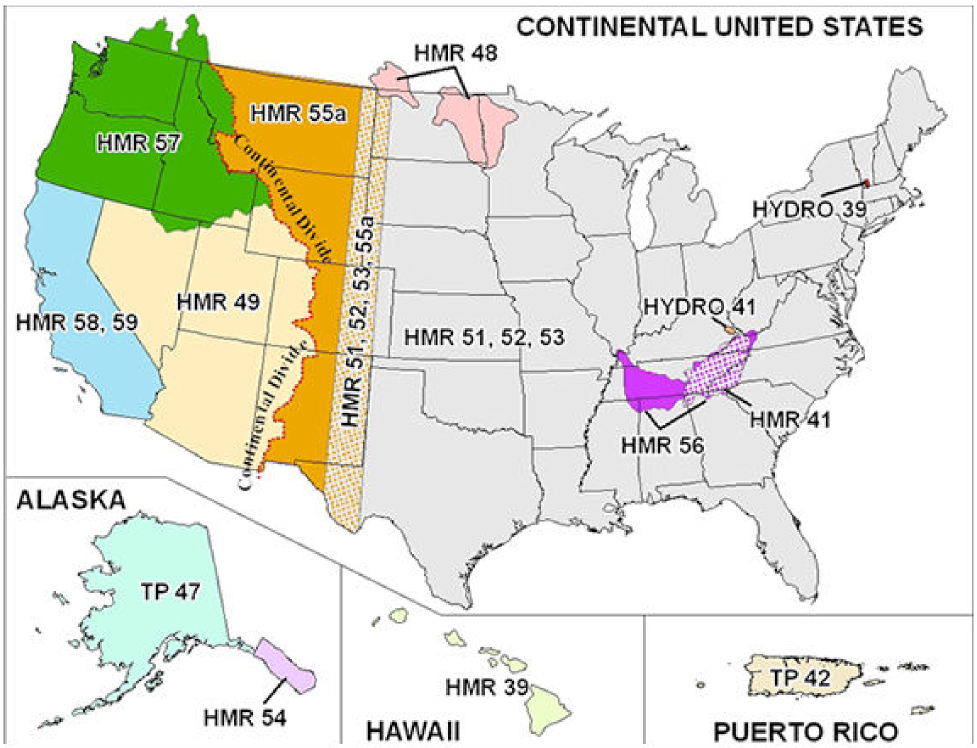

Figure 2.2.1.3-1. Generalized HMR series typically used to estimate PMP for Federal projects within the United States.

2.2.1.4 Other Design Floods

Design (frequency) flood hydrographs that are typically smaller and more frequent than the PMF and/or the IDF may be associated with a specific purpose or condition of a given reservoir, such as flood damage reduction, maximum safe downstream releases, etc. Several examples of more frequent design floods include:

- **Design floods for flood damage reduction**. At a number of Reclamation facilities, flood control is an important purpose; therefore, coordination with the U.S. Army Corps of Engineers (USACE) is necessary for flood damage reduction. As an example, for Reclamation's Folsom Dam, a design flood with a 0.005 AEP (200-year return period) has been selected for the Joint Federal Project. In this case, the USACE's flood damage reduction requirements apply up to a specified RWS elevation (associated with the maximum RWS elevation for the 200-year event), then above this RWS to the IDF-induced maximum RWS, Reclamation's dam safety considerations control operations.

- **Operational floods.** In some cases, multiple spillways and other hydraulic structures are employed to pass flood events and are triggered by different flood levels (flood return periods). For Reclamation's Gibson Dam (concrete gravity-arch), the service spillway (gated morning glory control structure) will pass up to the 100-year flood event before the auxiliary spillway (dam crest) begins to operate and augments discharges for flood return periods greater than 100 years.

- **Standard project flood (SPF).** A number of Reclamation facilities were designed and constructed by the USACE. For these facilities, the SPF may have been applied and is defined as a flood that results from the most severe combination of meteorological and hydrologic conditions that are considered reasonably characteristic of the region in which the study basin is located [5]. The SPF is intended as a practicable expression of the degree of protection to be considered for situations where protection of human life and high-valued property is required, such as for urban levees and/or floodwalls. It also provides a basis of comparison with the recommended protection for a given project. Although a specific frequency cannot be assigned to the SPF, a return period of a few hundred to a few thousand years is commonly associated with it. The SPF flood discharges are generally in the range of 40 to 60 percent of the PMF [5]. For all floods less than or equal to the SPF, releases from the dam are controlled to the point where downstream flood damages are limited. For floods more remote than the SPF, releases from the dam are increased, with the goal of preventing overtopping or other hydrologic PFMs for the dam.

- **Antecedent flood.** A flood that reflects meteorological and hydrological conditions prior to or coincident with a design flood (frequency flood, IDF, or PMF). An antecedent flood may be the result of rainfall-runoff or snowmelt-runoff and is usually much smaller in magnitude than the design flood. Antecedent floods are typically provided in conjunction with design flood hydrographs, so that flood routings are performed, using the design hydrographs that includes such assumptions and conditions. Antecedent flood methods vary, and they generally depend on the watershed of interest, purpose of flood study, and design needs. A frequency method is normally used to determine antecedent floods for Reclamation projects, with specific criteria for PMF estimates [6].

2.2.1.5 Construction Diversion Floods

Construction diversion floods are defined as floods that might occur during construction activities in and around a stream or river. These activities include constructing a new or modified dam and/or appurtenant structure. The term "construction diversion flood" refers to the flood level that can safely be passed through or around the construction site, typically relying on temporary cofferdams for some storage capability and low level outlet works, channels, flumes, culverts, or other hydraulic structures for discharging flows through or around the construction site for new dam construction. For modified dams, the method of passing floods during construction may be similar or identical to that prior to construction (i.e., through the existing spillway(s) and/or existing outlet works in combination with reservoir storage at the existing dam, although the spillway, outlet works, and dam may be altered during construction). For this document, the term "construction diversion flood" will refer to the level of flood that can safely be passed during construction, regardless of whether the construction is for a new dam or for the modification of an existing dam. Construction diversion floods are typically not the maximum hydrologic event that could occur on a given stream or river but are smaller, more frequent floods. These more frequent floods can be determined on an annual or seasonal basis. The maximum design flood level that can be safely accommodated during construction reflects a balance between cost of accommodating floods (i.e., cost associated with material and time, and construction sequencing) and risk of larger floods occurring, which could result in adverse consequences (i.e., impacts to construction, downstream damages, and potential life loss).

2.2.1.6 Flood Variables, Load Ranges, and Initial Reservoir Levels

In order to evaluate specific hydrologic-related PFMs, flood frequency information is needed on variables including flood peaks, volumes, durations, and elevations (e.g., maximum RWS). An example maximum RWS frequency curve (with uncertainty) is shown in figure 2.2.1.6-1. These relationships integrate variations in frequency flood hydrograph ranges, load uncertainties, and initial reservoir levels.

Load ranges[3] from both inflow and outflow frequency flood hydrographs are needed in many situations. Figure 2.2.1.6-2 shows an example inflow and outflow hydrograph. Outflow peaks are usually smaller than inflow peaks, and outflow hydrograph durations for some flow levels may be lengthened in the routing process. This difference between outflow and inflow may be due to a number of factors such as: (1) the discharge capacity is less than the inflow, which results in surcharging the reservoir (i.e., temporarily storing a portion of the flood); and/or (2) the initial RWS is below the minimum release elevation of one or more appurtenant structures (e.g., the initial RWS is below the spillway crest elevation),

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[3] In this case, load ranges are groupings of frequency floods (such as less than 1,000-year flood event, between 1,000- and 10,000-year flood events, and greater than 10,000-year flood event) that cover the full range of flood events.

which results in limited releases until the flood event fills the portion of the reservoir between the initial RWS and the minimum release elevation.

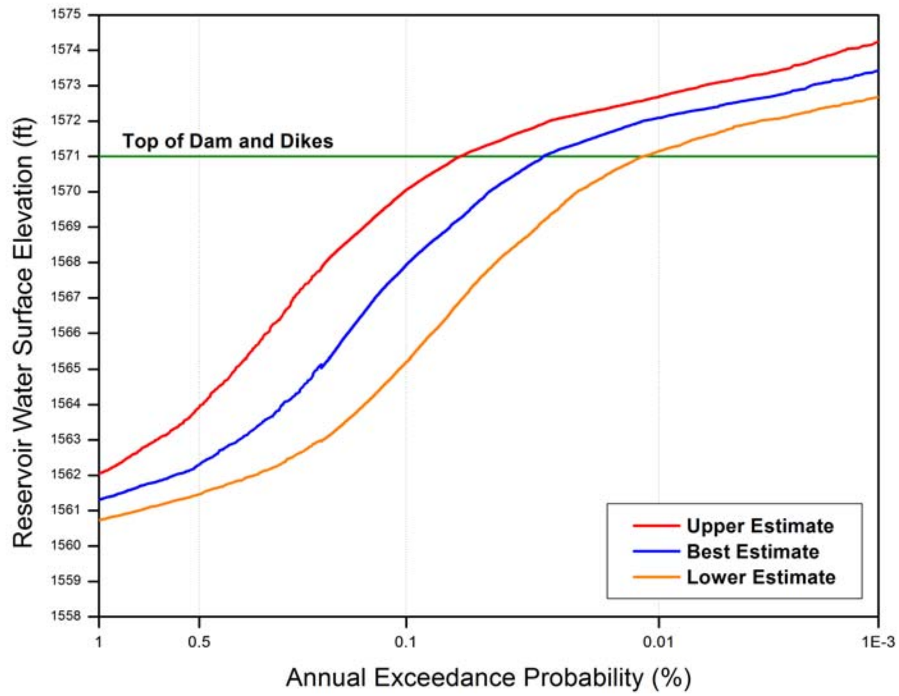


Figure 2.2.1.6-1. Example: Maximum RWS elevation frequency curve with uncertainty.

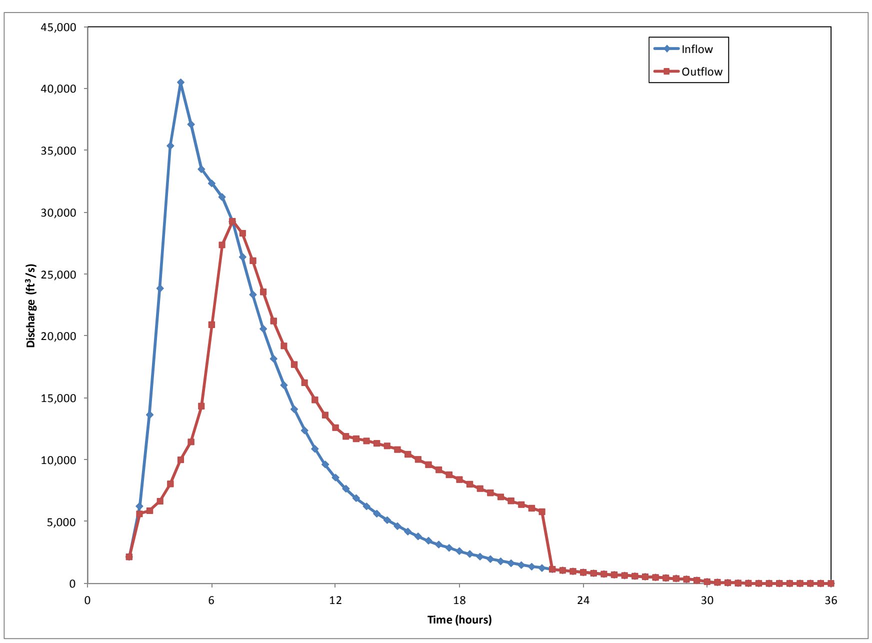


Figure 2.2.1.6-2. Example: Inflow and outflow hydrographs from reservoir routing.

Outflow hydrograph durations are important for estimating failure probabilities (for example, the duration of embankment overtopping or spillway chute wall overtopping). Response probabilities for hydrologic PFMs may need to be updated or revised as loads change or are revised. Evaluation of multiple flood variables (both thunderstorms and general storms) may be needed for certain facilities. For example, longer-duration general storm hydrographs might be critical for evaluating stagnation pressure related PFMs of spillway floor slabs, whereas shorter-duration, higher peak thunderstorm floods might be controlling for overtopping PFMs if reservoir storage is limited. Further details on flood variables and load ranges for hydrologic-related PFMs are provided in Chapter 3, "Hydrologic Hazard Analysis," of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

Initial reservoir levels are also an important consideration for evaluating hydrologic risks and design considerations. Often, the level of the reservoir at the onset of the flood is a key loading parameter for evaluating a PFM. Ranges of initial reservoir levels and their probabilities of occurrence, rather than fixed maximum levels (such as top of active conservation or spillway crest), need to be considered. Example reservoir exceedance relationships that vary by month are shown in figure 2.2.1.6-3.

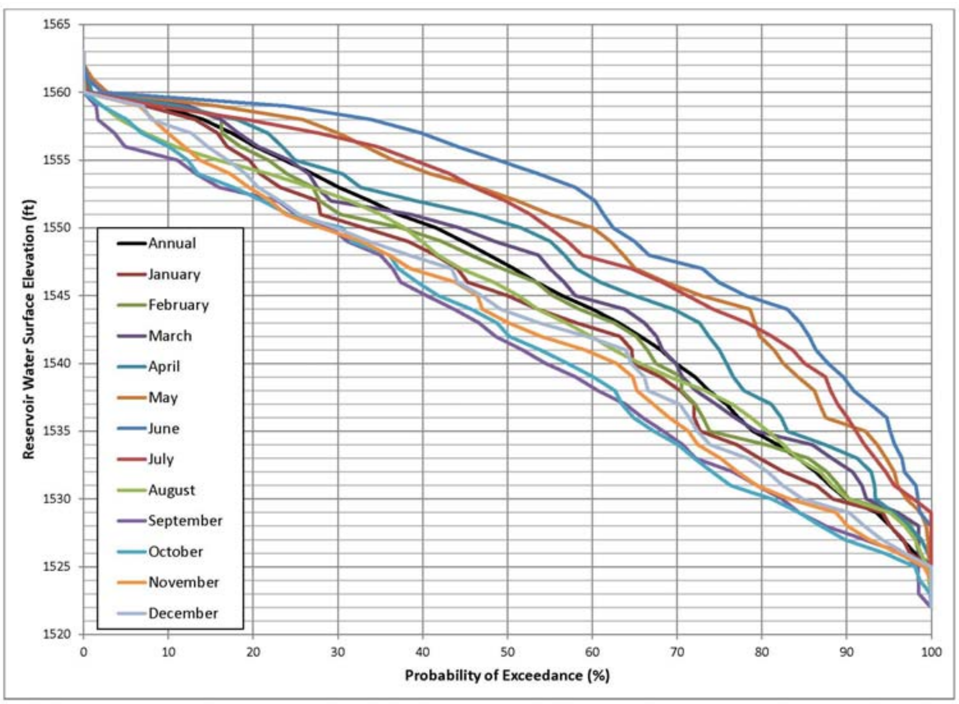


Figure 2.2.1.6-3. Example: Reservoir exceedance curves for each month, based on historical daily reservoir data, as initial reservoir water surface elevation ranges for frequency flood routings.

For existing structures, an evaluation of past reservoir operations data and project performance is made to estimate ranges of initial reservoir levels for each flood loading, as part of the hydrologic risk analysis. Also, there could be reservoir operations that would establish initial reservoir levels during certain times of the year (i.e., reservoir could be held at a lower elevation over the winter in anticipation of spring runoff), which should be considered and are typically documented in operations manuals such as Reclamation's Standing Operating Procedures (SOP) for a given dam. For new dams, reviewing available stream gauge data and considering planned project operations should be evaluated to estimate ranges of initial reservoir levels for each flood loading. Additional details are provided in Chapter 4, "Reservoir Level Exceedance Curves," of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

2.2.2 Quantitative Risk Analysis

A key aspect of identifying and selecting construction diversion floods and selecting the IDF is employing quantitative risk analysis methodology. This chapter relies on key references [3, 7, 8, and 9] for detailed guidance on applying quantitative risk analysis methodology. The following definitions and concepts touch on some of the aspects that apply to identifying and selecting construction diversion floods and selecting the IDF in a risk framework.

2.2.2.1 Potential Failure Mode

A PFM is a physically plausible (credible) process (or series of steps) leading to dam failure and uncontrolled release of the reservoir, which results from an existing inadequacy or defect related to a natural foundation or loading condition, the dam or appurtenant structure design, the construction, the operations and maintenance, the aging process, or a combination of these conditions [8].

2.2.2.2 Consequences

Consequences of dam failure can include economic losses due to property damage, loss of benefits, and ripple effects through the economy; environmental damages as a result of large downstream flows and releases of reservoir sediment; damages to cultural resources; and socioeconomic damages to the affected communities. Although these consequences can be considered in the decisionmaking process, the primary consequences considered in Reclamation's dam safety program are human fatalities (life loss). It should be noted that incremental consequences are used when evaluating hydrologic-induced PFMs. Incremental consequences represent the difference in life loss estimates for dam failure and life loss estimates for maximum nondam failure releases [8]. For additional details, see Chapter 5, "Consequences of Dam Failure," of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual.*

2.2.2.3 Event Tree

An event tree is a decomposition of a PFM into a series of steps representing the initiation, development, and completion of a PFM, including the consequences of failure, for determination of risks (including the annualized failure probability and the annualized life loss). Refer to Chapter 6, "Event Trees," of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7] for further information and examples of event trees.

2.2.2.4 f-N Chart

Reclamation's approach to portraying risks employs an f-N "event" chart which is composed of individual f-N pairs, where each pair typically represents one PFM (or in the case of total risk, the summation of all PFMs). On the f-N chart, "f" represents the annualized failure probability over all loading ranges. "N" represents the estimated life loss or number of fatalities associated with an individual PFM, or the weighted equivalent number of fatalities associated with the summation of the PFMs. Refer to Chapter 30, "Public Risk Tolerance and Risk Guidelines," of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7] for further information and examples of f-N charts. Also, refer to figure 2.4.1-3 for an illustration of Reclamation's f-N chart.

2.2.2.5 Confidence

Confidence is defined as a qualitative measure of belief that an engineering analysis, risk estimate, or recommended action is correct. Confidence is used to describe how sure the estimator(s) is about the general location of a risk estimate on an f-N chart [8].

2.2.2.6 Risk Terms from Reclamation's Public Protection Guidelines

A number of terms and associated concepts found in Reclamation's *Interim Dam Safety Public Protection Guidelines* [8] include:

- **Annualized failure probability (AFP).** The probability of dam failure occurring in any given year. It is the product of the probability of the load and the probability of dam failure given the load.

- **Annualized life loss (ALL).** The product of the AFP and the life loss that is expected to result from failure.

- **Risk.** The probability of adverse consequences. It can be measured in two ways – AFP and ALL.

- **Total risk.** The sum of the ALL for all PFMs and sum of the AFP for all PFMs, which are considered separately. If the sum of the ALL is greater than or equal to 1.0E-3, there is increasing justification to take action to reduce risk. If the sum of the ALL is less than 1.0E-3, there is decreasing justification to take action to reduce risk. If the sum of the AFP is greater than or equal to 1.0E-4, there is increasing justification to take action to reduce risk. If the sum of the AFP is less than 1.0E-4, there is decreasing justification to take action to reduce risk.

- **As low as reasonably practicable (ALARP).** ALARP considerations provide a way to address efficiency in reducing risks. The concept for the use of ALARP considerations is that risk reduction beyond a certain level may not be justified if further risk reduction is impracticable or if the cost is grossly disproportionate to the risk reduction. ALARP only has meaning in evaluating the justification for, or comparison of, risk reduction measures; it cannot be applied to an existing risk without considering the options to reduce the risk. Reclamation has chosen to apply ALARP principles when risks plot in the lower right region on an f-N chart that is associated with risk estimates that combine very high consequences (life loss estimated to be greater than 1,000 people) with very low annualized failure probabilities (less than 1.0E-6).

2.2.2.7 Typical Hydrologic Potential Failure Modes

The following flood-induced PFMs and the associated risks could influence the identification and/or selection of construction diversion floods and the selection of the IDF for existing and new dams and their appurtenant structures. For more details, see various chapters of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

- **Dam overtopping.** Overtopping of a dam, dike, and/or low portion (saddle) on the reservoir rim occurs when a flood event overwhelms reservoir flood surcharge storage and discharge capacity associated with available appurtenant structures (typically spillways and outlet works). For an embankment dam, dike, or saddle on the reservoir rim, if the depth and duration of overtopping is sufficient, erosion will result, which could lead to breach and an uncontrolled release of the reservoir. For a concrete dam, if the depth and duration of the overtopping is sufficient to erode abutments and/or foundation, leading to the undermining and destabilizing of the dam, breaching (due to downstream displacement) of the dam could occur, resulting in uncontrolled release of the reservoir. An additional consideration for embankment dam and dike overtopping is the potential concentration of flow along the groins (abutments contact between the dam/dike and the foundation).

- **Elevated RWS (nonovertopping of dam) resulting in internal erosion.** Flood-induced internal erosion of an embankment dam and/or dike would result from the RWS being substantially elevated above what the dam and/or dike may have historically experienced (i.e., first filling conditions). The elevated RWS would typically be above the maximum normal RWS (either top of active conservation or top of joint use storage, whichever is higher in elevation). Once the reservoir is above the maximum normal RWS, seepage flows could increase through flaws or discontinuities in the dam and/or dike, in the foundation, or a combination. Seepage velocities could be sufficient to move soil material, enlarging the discontinuities until a continuous conduit/pipe develops. Internal erosion would continue, eventually leading to a collapse of the conduit/pipe, erosion of the dam and/or dike crest, and an uncontrolled release of the reservoir.

- **Chute wall overtopping.** Flood-induced discharge that exceeds the maximum design discharge may result in overtopping of chute walls, leading to erosion of adjacent fill material and undermining and failing of a portion of the chute. With extended operation, additional erosion could lead to headcutting (and undermining of the control structure) upstream to the reservoir and an uncontrolled release of the reservoir.

- **Conduit/tunnel pressurization.** Flood-induced discharge that exceeds the maximum design discharge may result in pressurizing a conduit/tunnel that was designed for free-flow conditions. This pressurization could lead to two potential failure paths: (1) the conduit/tunnel lining is overloaded and collapses, and/or (2) with extended operation, injecting high-pressure flow through conduit/tunnel joints and/or cracks into the surrounding foundation material, causing erosion adjacent to the conduit/tunnel, which could destabilize portions of the conduit/tunnel lining. Once the conduit/tunnel lining has failed, the result could be extensive internal erosion (if foundation consists of soil materials) extending to the upstream reservoir and an uncontrolled release of the reservoir.

- **Cavitation of chute and/or conduit/tunnel.** Discharge through a concrete-lined (and in some cases, steel-lined) chute or conduit/tunnel with flow surface offsets at joints and/or other irregularities, such as cracks, may create separation of high velocity flow at the flow surface, which results in low pressure zones (vapor bubbles and/or voids form). These bubbles and/or voids rapidly collapse as they move into higher pressure zones, which issue high-pressure shock waves. Swarms of collapsing bubbles and/or voids can lead to fatigue and erosion of the flow surface material, such as concrete or steel liner. Cavitation damage is cumulative and may not occur upon first operation, but damage potential increases with operation time. With this in mind, extended operation and the erosion of the concrete or steel liner and foundation could lead to erosional headcutting upstream to the reservoir and an uncontrolled release of the reservoir.

- **Stagnation pressure of chute and/or conduit/tunnel.** Discharge through a concrete-lined chute or conduit/tunnel leads to introduction of high-velocity, high-pressure flow through open flow surface joints or cracks, which can result in structural damage or failure of the concrete lining due to uplift pressures (hydraulic jacking) and/or erosion of the foundation. Displacement of portions of the concrete-lined chute or conduit/tunnel can expose the foundation to further erosion. With continued operation, erosion of the foundation could lead to additional erosional headcutting (and undermining of the structure) upstream to the reservoir and an uncontrolled release of the reservoir. Stagnation pressure damage may occur during a single operation or may be cumulative.

- **Other.** This list of PFMs is not intended to be comprehensive. Other PFMs may apply, based on the specific conditions at an individual dam. For additional information and guidance, refer to various chapters of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

2.2.2.8 Uncertainty

Uncertainty is a qualitative or quantitative measure of the range or spread of reasonable outcomes of a risk estimate. Uncertainty is used to portray variability or range of values for loads, consequences, and risk estimates, rather than relying solely on single point estimates. Uncertainty is portrayed on the f-N chart by ranges of life loss and AFPs [8].

2.2.3 Downstream Hazard Potential Classification

An important purpose of any classification system is to help identify and select appropriate design guidelines. Such is the downstream hazard classification, which categorizes dams based on the probable loss of human life and the impacts on economic, environmental, and lifeline consequences[4] in the event of a dam failure and uncontrolled release of the reservoir. Three downstream hazard classification levels have been adopted: LOW, SIGNIFICANT, and HIGH, listed in order of increasing adverse incremental consequences. The downstream hazard classification levels build on each other (i.e., the higher order classification levels add to the list of consequences for the lower classification levels) [10].

2.2.3.1 Low Hazard

Dams assigned the low hazard classification are structures where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

2.2.3.2 Significant Hazard

Dams assigned the significant hazard classification are structures where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline interests, or can impact other

[4] Lifeline consequences include loss of communication and power, water and sewer services, and food supply.

concerns. Significant hazard classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.

2.2.3.3 High Hazard

Dams assigned the high hazard classification are structures where failure or misoperation will probably cause loss of human life. Of note, economic, environmental, and/or lifeline losses are not necessary for this classification if there is probable loss of human life.

2.3 Hydrologic Hazards and Design Floods

Historically, dam design and analysis methods have focused on selecting a level of protection based on evaluation of spillway flood loadings. Traditionally, the protection level has been the PMF. Currently, Reclamation uses quantitative risk analysis to assess the safety of dams, recommend safety improvements, and prioritize expenditures. Quantitative risk analysis, from a hydrologic perspective, requires an evaluation of a full range of hydrologic loading conditions and possible dam failure mechanisms tied to consequences of a failure. This risk approach is in contrast to the traditional approach of only using single upper bound, maximum events such as the PMF for design. However, for both existing and new dams and their appurtenant structures, there may be compelling nonrisk factors (such as minimal cost increases and maintaining public trust) to design for the PMF even if risk considerations support designing for a flood event less than the PMF.

The flood loading input to a dam safety risk analysis prepared by Reclamation is a Hydrologic Hazard Curve (HHC) that is developed from a Hydrologic Hazard Analysis (HHA). HHCs are peak flow and volume probability relationships [6, 11]. These HHCs are presented as graphs and tables of peak flow and volume (for specified durations) versus AEP. The range of AEPs that is displayed on these graphs is intended to be sufficient to support the decisionmaking needs of the organization. Presently, suites of frequency flood hydrographs ranging from 0.01 AEP (1/100-year return period) to approximately 0.00001 AEP (1/100,000-year return period) are developed [11] to help size existing modifications to dams and their appurtenant structures, along with sizing new dams and their appurtenant structures. Frequency flood hydrographs for AEPs less than (more remote than) this range are developed as needed for specific PFMs or for particular hydrologic risk evaluations. The maximum frequency flood peak and/or volume should not exceed the current critical PMF peak and/or volume. Frequency flood hydrographs ranging from 2 years to 100 years are typically developed to help evaluate diversion requirements (capabilities) during construction. Additional frequency flood hydrographs (such as seasonal flood events) are developed as needed for specific construction risk evaluations.

2.3.1 Identifying Hydrologic Data Needs

Hydrologic hazard data needs are determined on a case-by-case basis. Key hydrologic hazard data requirements include: streamflow data, historical flood and paleoflood data, and precipitation and extreme storm data. These data are described in various sources, including Reclamation's *Guidelines for Evaluating Hydrologic Hazards* [11] and Chapter 3, "Hydrologic Hazard Analysis," of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

Developing HHCs for risk analysis involves using the length of record and type of data to determine the extrapolation limits for hydrologic hazards. The length of record is often 50 to 100 years at a particular site (i.e., at-site). At-site data are defined as data that are measured or obtained within the watershed upstream of the dam or dam site of interest. Data sets are significantly expanded by using regional information and using space-for-time substitution concepts. Regional data (or regional analysis) consist of pooling streamflow, paleoflood, and precipitation data from many sites around the location of interest to substantially increase the information on extreme floods that are used to estimate HHCs. Paleoflood data are estimates of extreme floods, or limits on floods that usually have occurred over the past hundreds to thousands of years, determined from geologic evidence along rivers or streams. Details on data sources, data types, and extrapolation ranges for hydrologic hazards are listed in Reclamation's *Guidelines for Evaluating Hydrologic Hazards* [11].

Hydrologic hazard data needs vary and depend on the available information, hydrologic hazard method, level of study, and decision being made. The flood hydrologist works with the designer of record and design team to determine appropriate hydrologic hazard data needs commensurate with the HHA method and design level of study.

2.3.2 Hydrologic Hazard Levels of Study for Design

There are three hydrologic hazard levels of study that correspond to appraisal, feasibility, and final design levels. For the Reclamation Dam Safety Program, the typical study levels are Comprehensive Review (CR), Issue Evaluation (IE), and Corrective Action Study (CAS). These levels are approximately equivalent to appraisals for CRs and some IEs and feasibilities for some IEs and CASs [12]. It should be noted that dam safety studies do not involve design until the CAS takes place.

The initial CR-level and some IE-level (appraisal) approaches are to conduct a HHA with peak flow using statistical techniques and paleoflood data using reconnaissance-level studies. Hydrographs are then typically estimated based on scaling an observed flood event or a design event (such as the PMF) to the peak-flow frequency curve (peak-flow scaling) at various AEPs. In some cases, a rainfall-runoff model with available precipitation frequency information may be used for a CR-level and some IE-level studies. There may be considerable uncertainty with these flood estimates, which may provide approximate results (i.e., results may vary from conservative to unconservative) and require engineering judgment to determine if further action is needed. Flood estimates from these initial approaches are not used for IDF selection, but they are replaced by results from final-design level hydrologic hazard studies, depending on project needs.

Based on hydrologic risk results from CRs and/or some IEs (appraisal level analysis), some IEs and/or CASs (feasibility level study), or final designs may be completed to better define the hydrologic risk or design problem, reduce uncertainties, develop design solutions, or make decisions. These additional studies typically involve precipitation and extreme storm frequency analysis and modeling and more in-depth paleoflood studies, as well as the use of rainfall-runoff models with Monte Carlo approaches. Typical HHC and frequency flood estimates from these studies are ranges that include variations in peaks, volumes, hydrographs, and initial reservoir levels, and they include uncertainty. Figure 2.3.2-1 shows example ranges of hydrograph shapes and variations in peak flows (six hydrographs) that have the same 1/10,000 AEP flood volume. Maximum RWS elevations are also caused by combinations of peak, volume, and initial reservoir level, as shown in table 2.3.2-1.

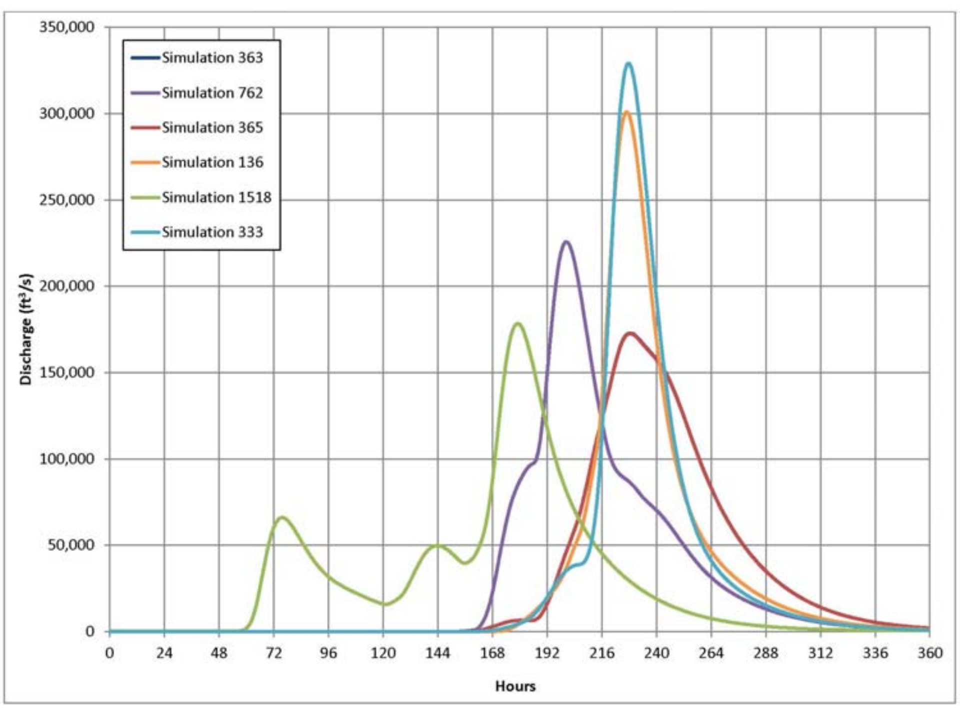


Figure 2.3.2-1. Example: Reservoir inflow frequency hydrograph variations based on a 1/10,000 AEP volume.

Table 2.3.2-1. Example: variations in peak inflow and initial reservoir level for a maximum RWS

Return period	AEP (%)	Max RWS (feet)	Initial RWS (feet)	Inflow peak (ft^3/s)	Volume (acre-feet)
21,900	4.56E-03	1572.98	1533.47	324,600	1,547,000
18,000	5.56E-03	1572.93	1549.97	320,100	859,000
15,200	6.56E-03	1572.93	1558.93	318,700	1,608,000

Note: ft^3/s = cubic feet per second

Details on the IE and CAS hydrologic hazard methods, including some of their strengths and limitations, are described in Reclamation's *Guidelines for Evaluating Hydrologic Hazards* [11] and in Chapter 3, "Hydrologic Hazard Analysis," of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7]. Since each study site is different, no single approach can be identified to address all hydrologic issues. The methods chosen consider climatic and hydrologic parameters, drainage area size and type, amount of upstream regulation, data availability and regional information, design level, and level of confidence needed in the results.

No single HHA approach is capable of providing the needed characterization of extreme floods over the full range of AEPs required for quantitative risk analysis. Results from several methods and sources of data should be weighted and combined to yield a single HHC. In ideal situations, Reclamation uses multiple methods to estimate HHCs due to the significant extrapolation of the flood frequency relationships and the uncertainties involved in the analysis. In practice, this means (for CAS and final designs) the use of two hazard methods for a dam: peak-flow frequency with paleoflood data, and a stochastic rainfall-runoff model. These estimates are then combined into a single recommended hazard curve. For more information about the multiple methods used by Reclamation, refer to the *Guidelines for Evaluating Hydrologic Hazards* [11].

The HHCs that are provided for design need to also include uncertainty estimates. Some examples of HHCs with uncertainty estimates are shown for peak flows in figure 2.2.1.1-1 and figure 2.2.1.6-1 for reservoir elevation. These uncertainty estimates may be considerable, depending on the data, period of record, methods utilized, and amount of extrapolation. It is important to quantify the HHC uncertainties and include them in the hydrologic risk analysis.

2.4 Inflow Design Flood

The following text provides a historical perspective (background) of IDFs and the current process for selecting IDFs. It should be noted that the current process is applicable to high and significant hazard dams and associated appurtenant structures (including most Reclamation storage and multipurpose dams, along with some diversion and detention dams). The current process can also be considered for low hazard dams (mostly diversion and detention dams). It should also be noted that if the current process is not used for a low hazard dam, an economic analysis is typically used to select the IDF. This economic analysis could be based on a similar approach used to select construction diversion floods. For more details, the reader is directed to Section 2.5.1.2, "Probabilistic Approach," and Example 2 in appendix B of this chapter. These approximate the USACE's method of balancing initial costs of the structure associated with specified flood return periods and the damage costs that are due to floods larger than the specified flood return periods.

2.4.1 Background

Starting in the early 1940s, Reclamation developed various deterministic approaches to estimate design floods based on the PMF or its variants [37, 38]. The IDFs for dams constructed before the early 1940s were based on a combination of flood frequency curve extrapolations, envelope curves, and maximum flood ratios (e.g., 50 percent greater than the flood of record). Prior to 1980, an equivalent deterministic approach was used, where the Maximum Probable Flood (MPF) was typically selected as the IDF for most storage and multipurpose dams. The MPF is roughly equivalent to the PMF, except that site-specific PMP rainfall information was used, rather than PMP estimates from the HMR series [38] (see figure 2.2.1.3-1). Reclamation subsequently adopted PMF nomenclature in the early 1980s, along with most Federal agencies [6, 38]. During the early 1980s and into the mid-1990s, IDF selection criteria were used which were based on downstream hazard potential classification and potential loss of project operations [37, 41]. With the adoption of quantitative risk analysis methodology in the mid-1990s, the selection process has changed in the following ways:

- The PMF is considered the maximum hydrologic loading (i.e., IDF will be equal to or less than the current critical PMF).
- Frequency flood events may be estimated up to the current critical PMF size (i.e., peak and volume). Figure 2.4.1-1 illustrates the relationship between the PMF and frequency floods.

- The Interim Guidelines for Addressing the Risk of Extreme Hydrologic Events [3] defines the general concepts between frequency floods and the PMF.

Figure 2.4.1-2 is a generalized HHC showing the relationship between the PMF and frequency floods.

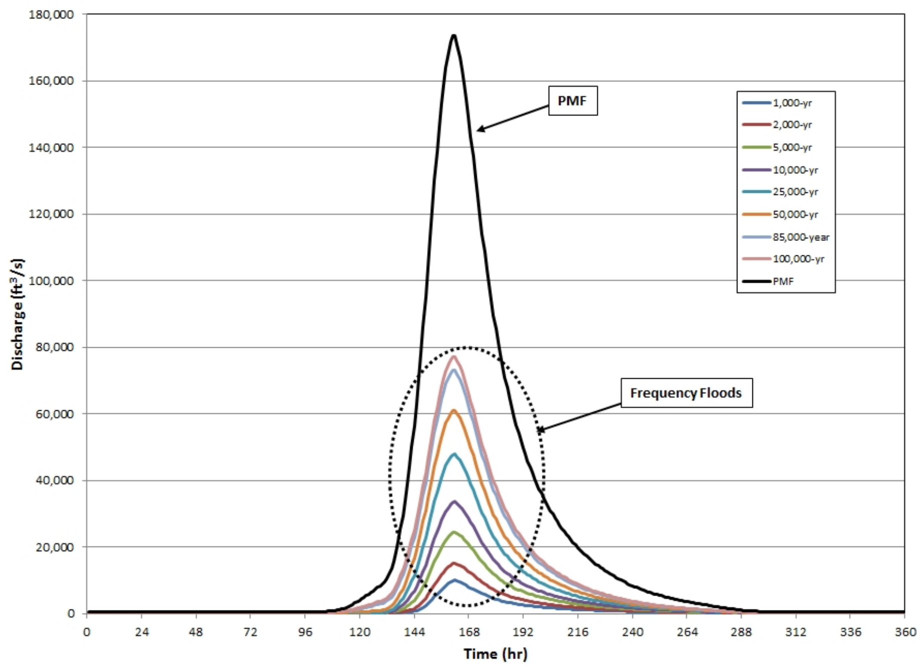


Figure 2.4.1-1. Example frequency flood hydrographs and relationship to the PMF

For both modifying existing and constructing new dams, along with appurtenant structures, the following generalized selection process is used:

- Clearly identify credible PFMs for all loading conditions (static, seismic, and hydrologic).

- Estimate baseline total risks for all credible PFMs involving all loading conditions (includes loads, responses, and consequences).

- Establish initial design loading conditions:

    - For modifications to an existing dam and appurtenant structures, the IDF is equal to or less than the current critical PMF, and the design earthquake event is typically equal to or less than the 50,000-year return period (for more information see Chapter 3, "General Spillway Design Considerations," and Chapter 4, "General Outlet Works Design

Considerations," of this design standard). Initial flood and earthquake loadings are selected to approximate estimated threshold events (i.e., loadings larger than the selected design events would likely result in initiation of PFMs).

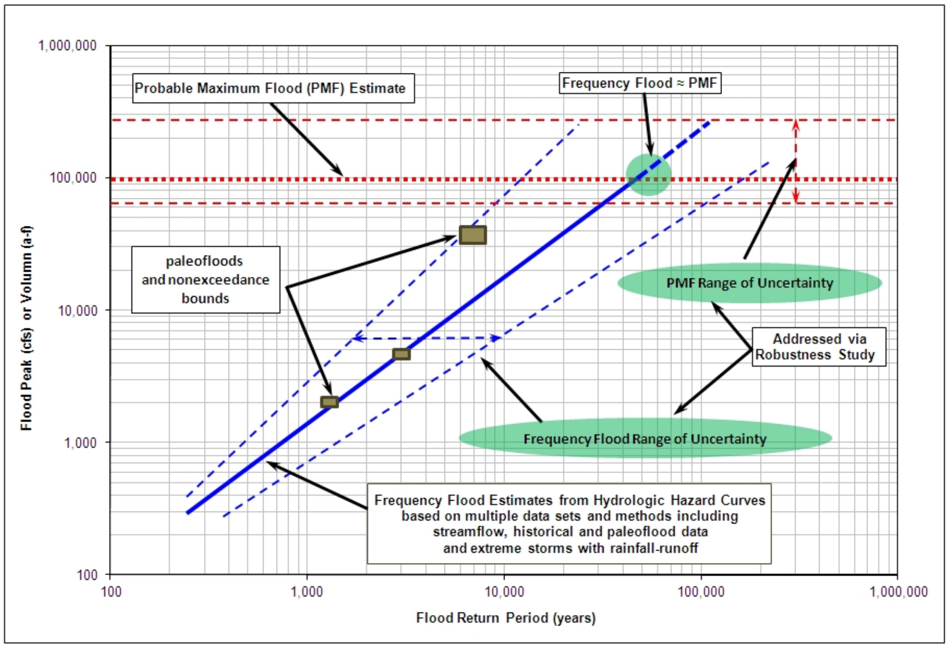


Figure 2.4.1-2. Generalized HHC schematic (relationship between the PMF and frequency floods).

- For designing a new dam and appurtenant structures, target design flood and earthquake may have similar ranges as noted for modifications to an existing dam and appurtenant structures. For new dams, the maximum practical frequency flood event (including the current critical PMF) as the IDF, and the maximum practical frequency earthquake event should be selected for design. For new dams, there may be great cost efficiency in mitigating more remote risks than there would be for modifying existing dams. Initial flood and earthquake loadings are selected to maintain total risks to a level of decreasing justification to take action (i.e., less than Reclamation's risk guidelines).

- For both modifications to an existing dam and designing a new dam, where stochastic modeling has resulted in hundreds to hundreds of thousands of frequency hydrographs, initial design maximum RWSs with initial design maximum discharges are selected (rather than initial flood return periods).

- Confirm or revise initial flood (or initial maximum RWS with initial maximum discharge) and earthquake design levels. If the initial frequency flood level results in total risk plotting in an area of the f-N chart indicating decreasing justification to take action to reduce risks, the frequency flood level (or a maximum RWS with a maximum discharge) may be acceptable. It should be highlighted that whether or not a flood level (or a maximum RWS with a maximum discharge) is judged as acceptable will be unique to each condition/situation and will be recommended by the designer of record and concurred with by Reclamation management (decisionmakers). Also, risks below Reclamation guidelines do not ensure that a chosen frequency flood level (or a maximum RWS with a maximum discharge) is acceptable. Other risk and nonrisk factors such as uncertainty, confidence, cost, physical constraints, etc., will have a bearing on identifying acceptable risks. If the total risk is unacceptably high, a more remote frequency flood event should be assumed, and total risks should be re-estimated until the total risks are in an area of the f-N chart indicating decreasing justification to take action to reduce risks. Total risk could range from just below, to more than one order of magnitude below Reclamation guidelines [8].5 Figure 2.4.1-3 illustrates this concept.

- Select freeboard above the design maximum RWS elevation based on robustness study (see section 2.4.2.3).

[5] Designs may be acceptable if the estimated sum (total) of the AFP and the ALL for all credible PFMs is in an area of the f-N chart below Reclamation guidelines (1E-4 or a 1 in 10,000 chance during a given year for AFP; and 1E-3 or a 1 in 1,000 chance during a given year for ALL); however, it should be noted that risks below Reclamation guidelines do not ensure that the design is acceptable. Other risk and nonrisk factors such as uncertainty, confidence, ALARP, cost, physical constraints, etc., will have a bearing on identifying acceptable designs. If total risks are not in an area of the f-N chart below Reclamation guidelines, additional design considerations/features will likely be necessary to lower the estimated total AFP and ALL for the modified or new dam, spillway, and/or outlet works.

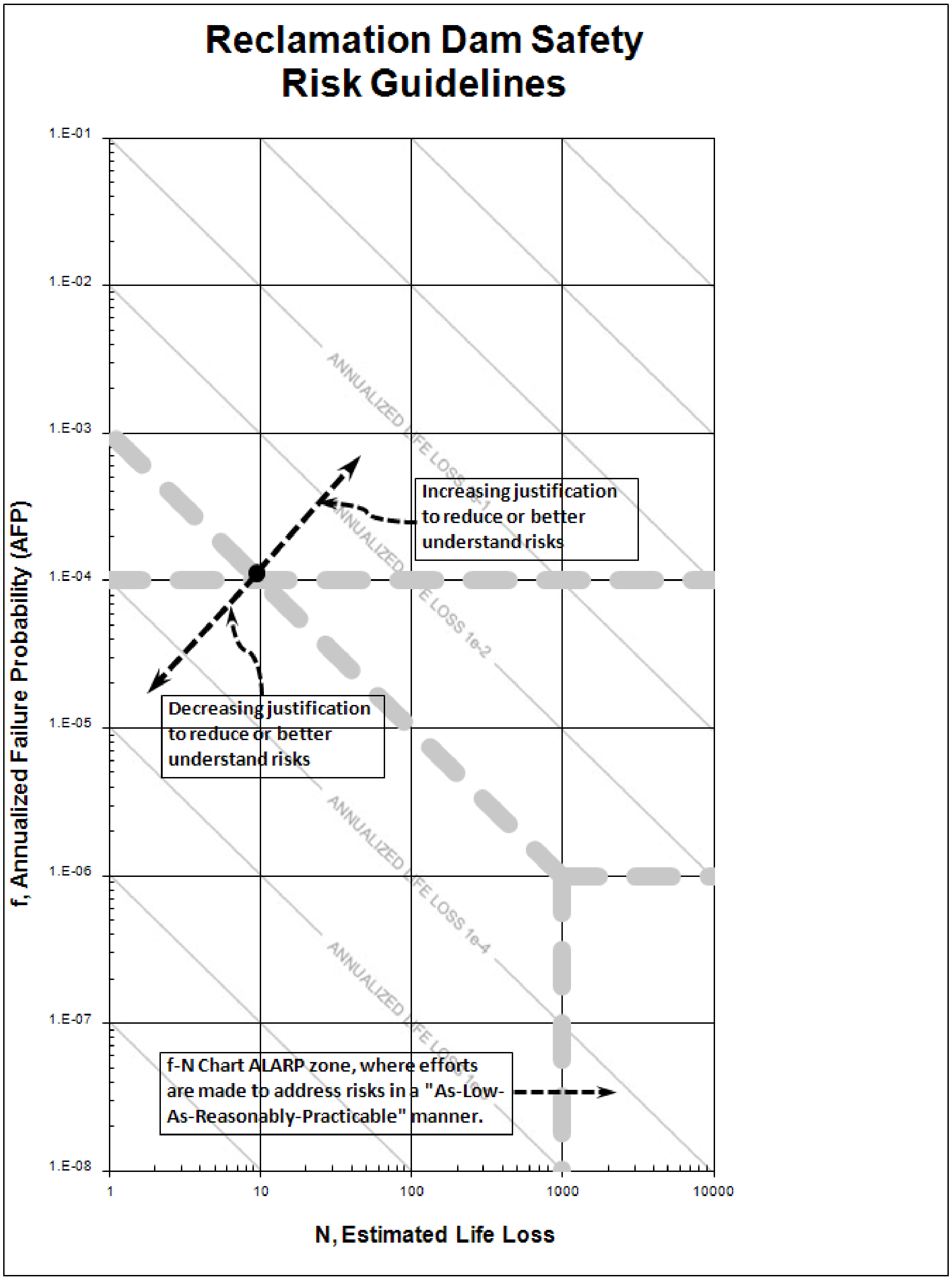
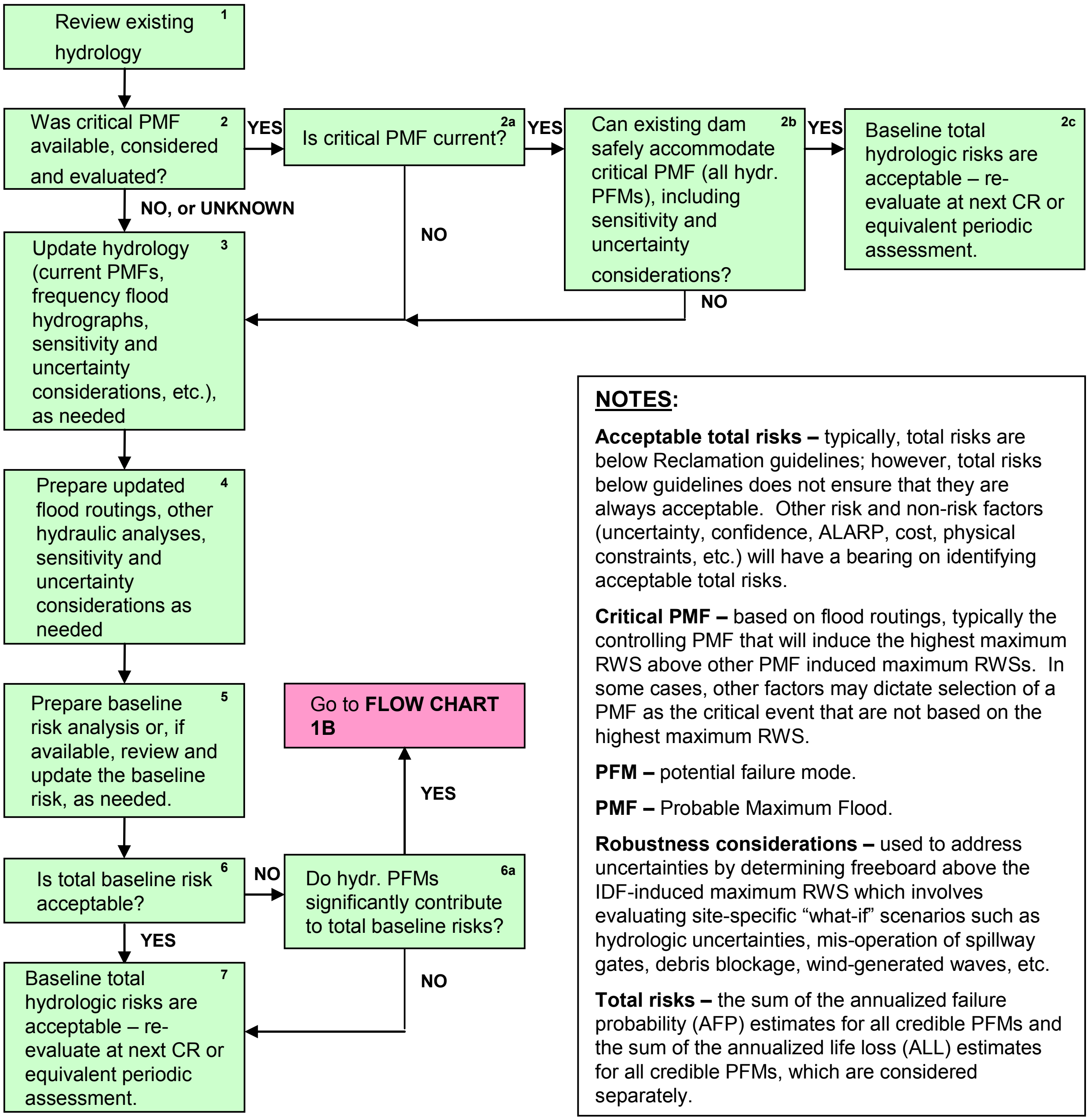


Figure 2.4.1-3. Reclamation's f-N chart.

Flow Chart 1A – Significant and High Hazard Existing Dams:

Process for Evaluating Baseline Risks (Hydrologic Loading)



NOTES:

Acceptable total risks – typically, total risks are below Reclamation guidelines; however, total risks below guidelines does not ensure that they are always acceptable. Other risk and non-risk factors (uncertainty, confidence, ALARP, cost, physical constraints, etc.) will have a bearing on identifying acceptable total risks.

Critical PMF – based on flood routings, typically the controlling PMF that will induce the highest maximum RWS above other PMF induced maximum RWSs. In some cases, other factors may dictate selection of a PMF as the critical event that are not based on the highest maximum RWS.

PFM – potential failure mode.

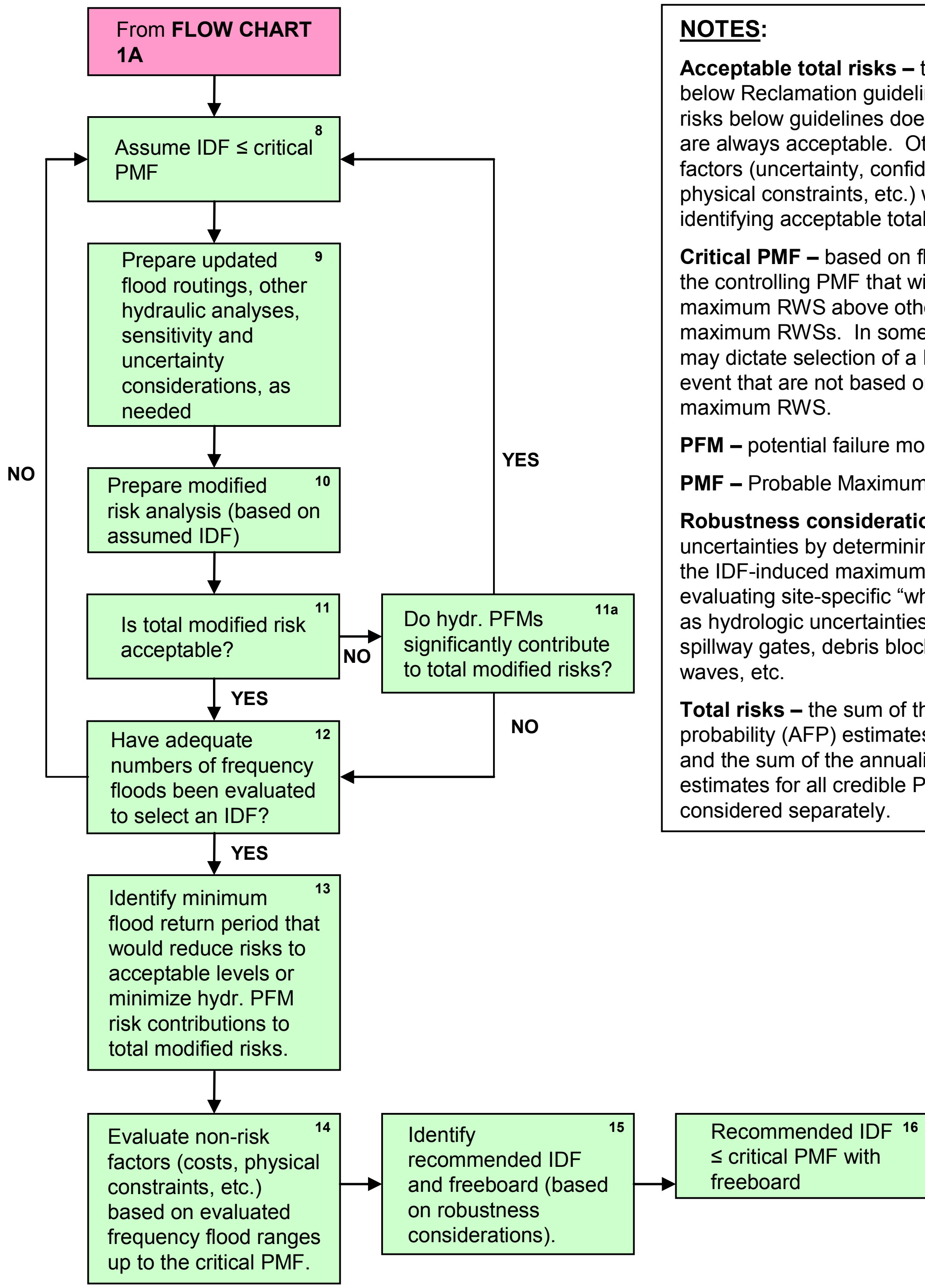
PMF – Probable Maximum Flood.

Robustness considerations – used to address uncertainties by determining freeboard above the IDF-induced maximum RWS which involves evaluating site-specific "what-if" scenarios such as hydrologic uncertainties, mis-operation of spillway gates, debris blockage, wind-generated waves, etc.

Total risks – the sum of the annualized failure probability (AFP) estimates for all credible PFMs and the sum of the annualized life loss (ALL) estimates for all credible PFMs, which are considered separately.

Flow Chart 1B – Significant and High Hazard <u>Existing Dams</u>:

Process for Selecting the Inflow Design Flood (IDF)



<u>NOTES</u>:

Acceptable total risks – typically, total risks are below Reclamation guidelines; however, total risks below guidelines does not ensure that they are always acceptable. Other risk and non-risk factors (uncertainty, confidence, ALARP, cost, physical constraints, etc.) will have a bearing on identifying acceptable total risks.

Critical PMF – based on flood routings, typically the controlling PMF that will induce the highest maximum RWS above other PMF induced maximum RWSs. In some cases, other factors may dictate selection of a PMF as the critical event that are not based on the highest maximum RWS.

PFM – potential failure mode.

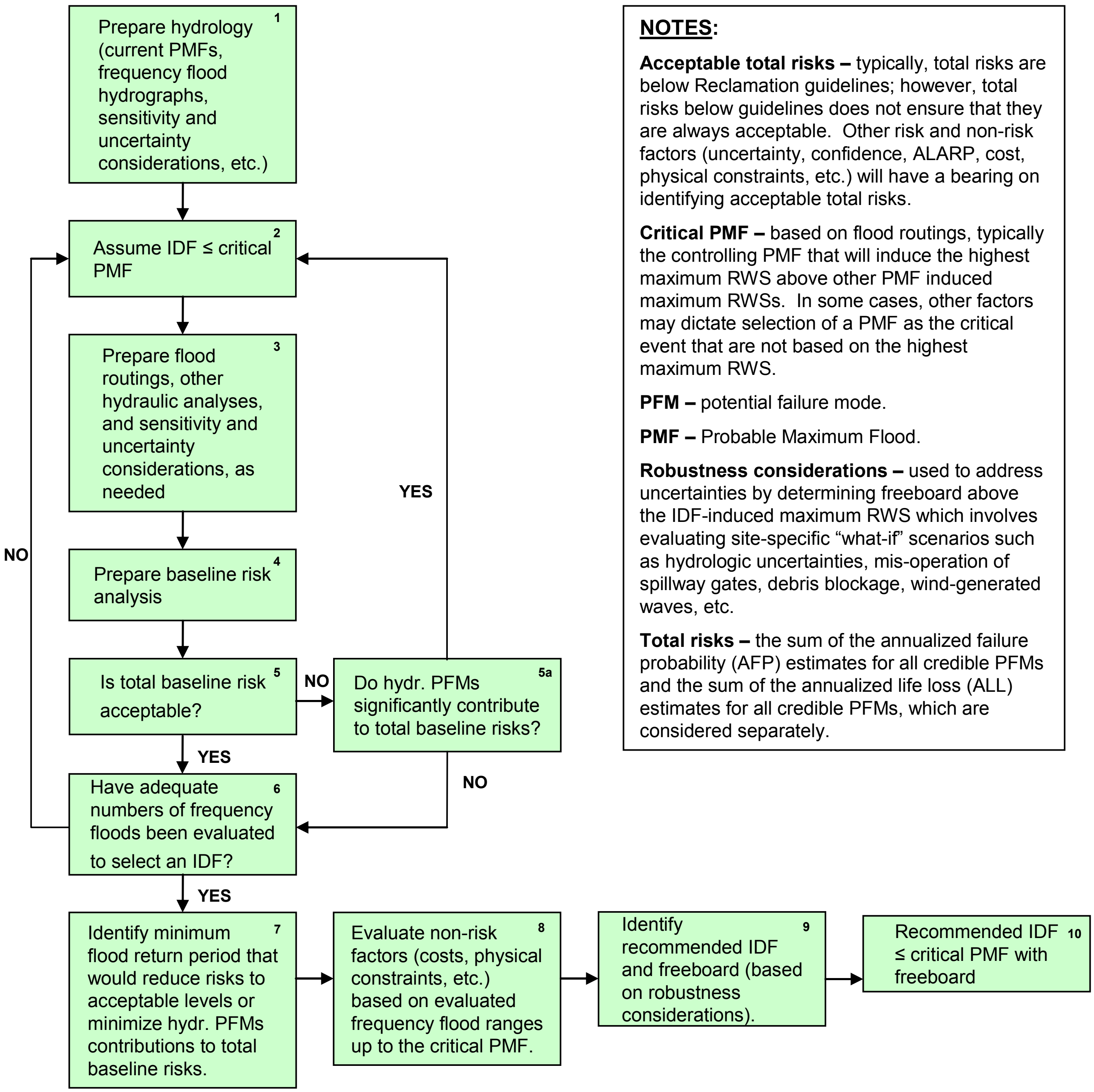
PMF – Probable Maximum Flood.

Robustness considerations – used to address uncertainties by determining freeboard above the IDF-induced maximum RWS which involves evaluating site-specific "what-if" scenarios such as hydrologic uncertainties, mis-operation of spillway gates, debris blockage, wind-generated waves, etc.

Total risks – the sum of the annualized failure probability (AFP) estimates for all credible PFMs and the sum of the annualized life loss (ALL) estimates for all credible PFMs, which are considered separately.

Flow Chart 2 – Significant and High Hazard <u>New Dams</u>:

Process for Selecting the Inflow Design Flood (IDF)



<u>NOTES</u>:

Acceptable total risks – typically, total risks are below Reclamation guidelines; however, total risks below guidelines does not ensure that they are always acceptable. Other risk and non-risk factors (uncertainty, confidence, ALARP, cost, physical constraints, etc.) will have a bearing on identifying acceptable total risks.

Critical PMF – based on flood routings, typically the controlling PMF that will induce the highest maximum RWS above other PMF induced maximum RWSs. In some cases, other factors may dictate selection of a PMF as the critical event that are not based on the highest maximum RWS.

PFM – potential failure mode.

PMF – Probable Maximum Flood.

Robustness considerations – used to address uncertainties by determining freeboard above the IDF-induced maximum RWS which involves evaluating site-specific "what-if" scenarios such as hydrologic uncertainties, mis-operation of spillway gates, debris blockage, wind-generated waves, etc.

Total risks – the sum of the annualized failure probability (AFP) estimates for all credible PFMs and the sum of the annualized life loss (ALL) estimates for all credible PFMs, which are considered separately.

2.4.2 Selection Process

The selection process for the IDF (or a design maximum RWS with a design maximum discharge) is defined by the steps previously noted and is further detailed in this section and summarized by Flow Charts 1A and 1B (existing dams and appurtenant structures) and Flow Chart 2 (new dams and appurtenant structures). Appendix A contains additional examples of selecting the IDF. The flow charts summarize the process on one page and are intended to be a quick reference once the user becomes familiar with the process.

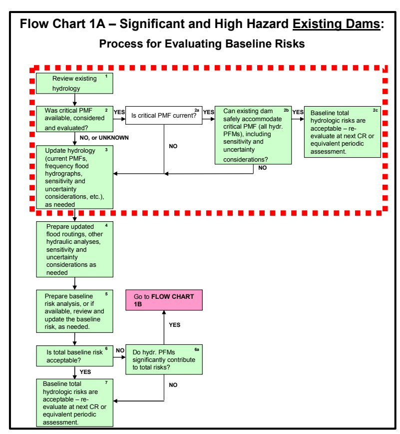
The sections below explain how to use the flow charts and supporting text when selecting an IDF (or a design maximum RWS with a design maximum discharge). Small pictures of the flow charts appear beside the text to provide the reader with a visual image of the portion of the flow chart under consideration in the text (highlighted by red dashed lines). The reader is directed back to the page-size flow charts for reading purposes.

2.4.2.1 Existing Dams

The following text discusses Flow Chart 1A (Significant and High Hazard <u>Existing Dams</u>: Process for Evaluating Baseline Risks) and 1B (Significant and High Hazard <u>Existing Dams</u>: Process for Selecting the Inflow Design Flood, IDF).

1. **Review and Evaluate Existing Hydrology/Hydraulics.** This discussion addresses **Flow Chart 1A, boxes 1, 2, 2a, 2b, 2c, and 3**.

   An initial screening/evaluation of existing hydrology is made **(box 1)**, which will help determine if the current critical PMF is available and was considered and evaluated for the existing dam **(boxes 2 and 2a)**.

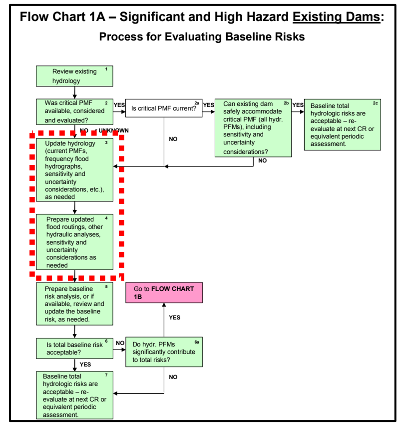


   If the current critical PMF is not available or is unknown, updated hydrology is needed **(box 3)**. This updated hydrology may or may not include the current critical PMF, but it will include sufficient updated hydrology (frequency floods) to select an IDF (or a design maximum RWS with a design maximum discharge) in a risk framework. The type and amount of updated hydrology will be site specific, and coordination with Reclamation's Technical Service Center's (TSC's) Flood Hydrology and Consequences Group will be required.

If the current critical PMF is available, a determination is made as to whether the dam can safely accommodate the current critical PMF, including analyses (typically flood routings and water surface profiles) needed to evaluate all hydrologic PFMs, such as dam overtopping, overtopping potential of chute walls, pressurizing of conduits/tunnels, cavitation, and stagnation pressure related PFMs. Also, uncertainties related to hydrology, reservoir operations, and/or future changes are addressed through sensitivity and uncertainty considerations **(box 2b)**. These sensitivity and uncertainty considerations may be similar to the robustness study (refer to Section 2.4.2.3, "Address Uncertainties with Freeboard (Robustness) Study, Existing and New Dam" in this chapter), but they are addressed prior to selecting an IDF (or a design maximum RWS with a design maximum discharge) and are integrated into evaluating the current critical PMF. It should be pointed out that a range of starting RWSs with the maximum starting RWS either at the top of active conservation or top of joint use storage, whichever is higher, should be used for the hydraulic analyses. The minimum starting RWS would typically reflect historical reservoir operations in terms of how low the reservoir might reasonably be during the time of year for the flood events. When establishing ranges of starting RWSs, consideration should be given to the event tree used to estimate risks for a given PFM. Refer to Chapter 6, "Event Tree," of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

If the existing dam cannot safely accommodate the current critical PMF, updated hydrology may be needed, which might include developing frequency floods based on the current critical PMF **(box 3)**. As previously noted, the type and amount of updated hydrology will be site specific, and coordination with the TSC's Flood Hydrology and Consequences Group will be required.

If the existing dam can safely accommodate the current critical PMF, including sensitivity and uncertainty considerations **(box 2c)**, baseline total hydrologic risks are acceptable, and further evaluation is not warranted until the next CR or equivalent period assessment. (Note: Reclamation considers the PMF as the maximum hydrologic loading condition possible at a given site.) It should be pointed out that further evaluation of risks associated with static and/or seismic PFMs may be needed to determine if total baseline risks are acceptable. Actions associated with a nonhydrologic evaluation are not part of this chapter. Guidance can be found in various chapters of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

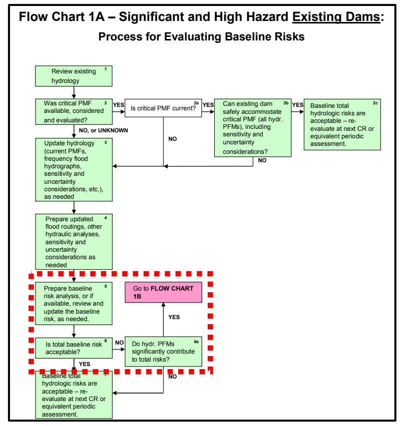


2. **Update Hydrology/Hydraulics.** This discussion addresses **Flow Chart 1A, boxes 3 and 4**.

   If it is determined that the current critical PMF has not been developed or routed through the existing dam or the existing dam cannot safely accommodate the current critical PMF (based on analyses needed to evaluate all hydrologic PFMs and sensitivity and uncertainty considerations have been evaluated), then some level of updated hydrology will likely be needed **(box 3)**. As previously mentioned, on a case-by-case basis, this could include preparing current PMFs along with frequency floods associated with the PMFs.

   Once the updated hydrology is available, flood routings would be performed to determine the response of the dam to a range of flood loadings up to and including the current critical PMF **(box 4)**. It should be noted that sensitivity and uncertainty considerations come into play. These sensitivity and uncertainty considerations may be similar to the robustness study (refer to section 2.4.2.3), but they are addressed prior to selecting an IDF (or a design maximum RWS with a design maximum discharge) and are integrated into the baseline risk analysis. Also, the flood routings would use a range of starting RWSs with the maximum starting RWS being either the top of active conservation or top of joint use storage, whichever is higher. As previously noted, the minimum starting RWS would typically reflect historical reservoir operations in terms of how low the reservoir might reasonably be during the time of the year for the flood event. When establishing ranges of starting RWSs, consideration should be given to the event tree used to estimate risks for a given PFM. Refer to Chapter 6, "Event Tree," of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7]. Other hydraulic analyses typically done include: water surface profiles to assess potential overtopping of spillway chute walls and/or pressurizing of conduits/ tunnels; flow cavitation potential evaluation; and stagnation pressure (hydraulic jacking and/or spillway foundation erosion, if applicable) evaluation (refer to Chapter 3, "General Spillway Design Considerations," and Chapter 4, "General Outlet Works Design Considerations," of this design standard).

3. **Prepare Total Baseline Risks.** This discussion addresses **Flow Chart 1A, boxes 5, 6, and 6a**.

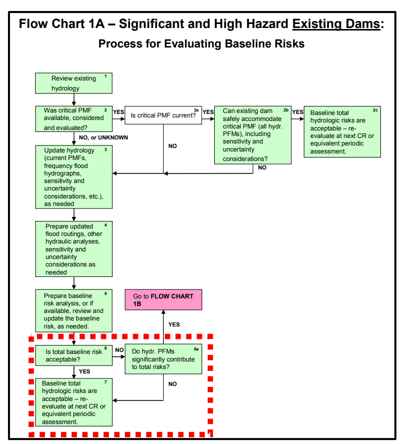


All credible PFMs for all loading conditions (static, seismic, and hydrologic) are identified and evaluated for the existing dam. A quantitative risk analysis using current risk methodology [3, 7, 8, and 9] is prepared or, if it already exists, is reviewed and updated, if needed (**box 5**).

If the total risks (the total annualized failure probability or the total annualized life loss) are not acceptable **(box 6)**, further evaluation of the hydraulic PFM risk contributions to the total risks is made **(box 6a)**. For the hydrologic PFMs, it should be highlighted that incremental consequences are used, which represent the difference of the life loss estimates for the dam failure and life loss estimates for maximum nondam failure releases. The term "not acceptable" typically means the total risk is in an area on the f-N chart indicating increasing justification to take action to reduce risks.

If the total risks are acceptable **(box 6)**, further evaluation of risks associated with static and seismic PFMs are not warranted until the next CR or equivalent periodic assessment **(box 7),** which is further discussed in the following step. As previously noted, an acceptable level of risk will typically be an area of the f-N chart indicating decreasing justification to take action.

4. **Actions if Total Baseline Risks are Unacceptable.** This discussion addresses **Flow Chart 1A, boxes 6, 6a, and 7**.

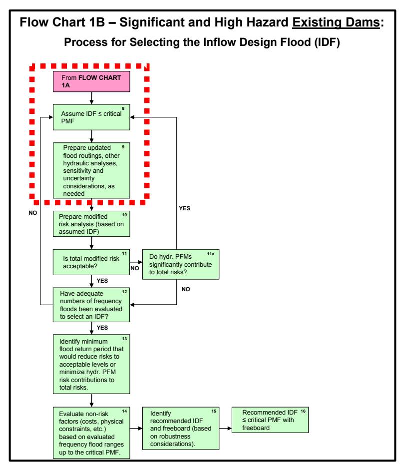


If it is determined that the hydrologic PFM risks do not significantly contribute to the total baseline risks **(box 6a)**, the process continues with concluding that no further evaluation of the hydrologic risks are needed until the next CR or equivalent periodic assessment **(box 7)**. It should be noted that further evaluation of risks associated with static and/or seismic PFMs would be needed to determine if total baseline risks are acceptable. Actions associated with this evaluation are not part of this Design

Standard, but guidance can be found in various chapters of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

If the hydrologic PFM risks are determined to significantly contribute to the total baseline risks **(box 6a)**, further evaluation of nonstructural and structural modifications that could reduce risks is made, which leads to the selection of an IDF (or a design maximum RWS with a design maximum discharge) (Go to **Flow Chart 1B**). This effort generally involves evaluating a suite of possible modifications, including but not limited to: reservoir restriction, breaching the dam, raising the dam, increasing the discharge capacity of existing appurtenant structures (spillways and/or outlet works), overtopping protection, and/or constructing new appurtenant structures (spillways and/or outlet works). The evaluation of these possible modifications would require appropriate analyses and designs, including flood routings up to and including the current critical PMF. Also, appropriate authorization[6] to evaluate modifications to the existing dam must be provided before the process is continued.

5. **Actions if Hydrologic PFMs Significantly Contribute to Total Baseline Risks.** This discussion addresses **Flow Chart 1B, boxes 8 and 9**.



To proceed with this portion of the process, a determination is made that modifications should be evaluated and authority has been given (from **Flow Chart 1A**).

As a starting point, the current critical PMF (determined from flood routings), or the maximum (most remote) frequency flood available is the assumed IDF, or the maximum RWS with a maximum discharge (associated with all floods evaluated in the HHC) is assumed as the design level **(box 8)**. This will be the maximum hydrologic loading condition and will set upper limits of modifications. As an alternative, in lieu of initially equating the IDF to the current critical PMF or selecting the maximum RWS with the maximum discharge (associated with all floods evaluated in the HHC) , the initial assumed IDF or the initial design maximum RWS with the design maximum discharge can be estimated by selecting a frequency flood equal to, or more remote

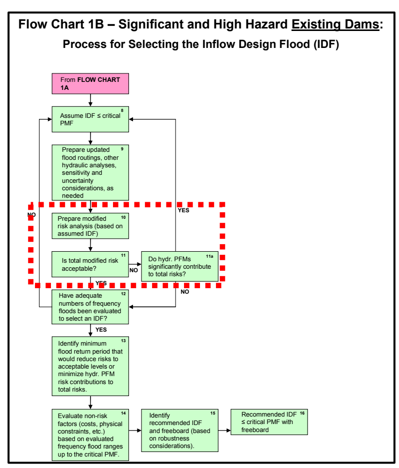
[6] Higher level studies (such as CAS, feasibility and/or final designs) must be approved by Reclamation management and, in some cases, authorized by Congress before this process can continue.

than, the result of dividing the mean annualized life loss by the mean incremental life loss for the hydrologic PFMs.

Updated flood routings and other hydraulic analyses would be performed to determine the response of the modified dam to the assumed IDF **(box 9)**. The flood routings and other hydraulic analyses are used to select the type, size, and location of the modified dam and/or appurtenant structures (spillways and/or outlet works). Additional guidance for selecting the type, size, and location of the modified or new appurtenant structures (spillways and/or outlet works) can be found in Chapter 3, "General Spillway Design Considerations," and Chapter 4, "General Outlet Works Design Considerations," of this design standard. It should be noted that sensitivity and uncertainty considerations may be similar to the robustness study (see Step 8 and refer to Section 2.4.2.3, "Address Uncertainties with Freeboard (Robustness) Study, Existing and New Dams," in this chapter), but they are addressed prior to selecting an IDF (or a design maximum RWS with a design maximum discharge) and are integrated into the baseline risk analysis. Also, the flood routings would use a range of starting RWSs with the maximum starting RWS either at the top of active conservation or top of joint use storage, whichever is higher. As previously noted, the minimum starting RWS would typically reflect historical reservoir operations in terms of how low the reservoir might reasonably be during the time of the year for the flood event. Other hydraulic analyses typically performed include water surface profiles to assess potential overtopping of spillway chute walls and/or pressurizing of conduits/tunnels, flow cavitation potential evaluation, and stagnation pressure (hydraulic jacking) evaluation. Additionally, downstream inundation mapping and associated estimated population at risk will be needed for subsequent steps of Flow Chart 1B.

6. **Prepare Total Modified Risks.** This discussion addresses **Flow Chart 1B, boxes 10, 11, and 11a**.

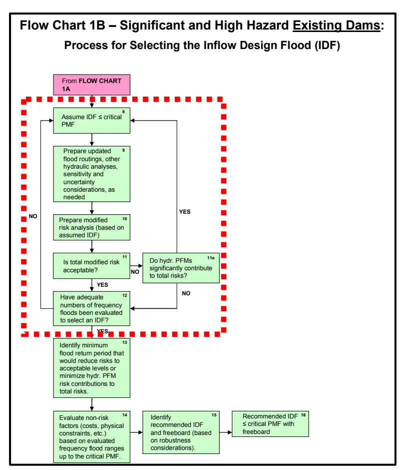
   All credible PFMs for all loading conditions are identified and evaluated for the modified dam and/or appurtenant structures. A modified quantitative risk analysis (based on the assumed IDF) using current risk methodology [3, 7, 8, and 9] is prepared (**box 10**).



If the total modified risks (the total annualized failure probability or the total annualized life loss) are not acceptable **(box 11)**, further evaluation of the hydraulic PFM risk contributions to the total risks is made **(box 11a)**. For the hydrologic PFMs, it should be highlighted that incremental consequences should be used, which is the difference of the life loss estimates for the dam failure and life loss estimates for maximum nondam failure releases. The term "acceptable" typically means the total risk is in an area on the f-N chart indicating decreasing justification to take action to reduce risks. However, it should be noted that risks below Reclamation guidelines are not necessarily acceptable. Other risk and nonrisk factors such as uncertainty, confidence, cost, physical constraints, etc., will have a bearing on identifying acceptable risks.

If the total modified risks are acceptable **(box 11)**, a determination is made as to whether adequate numbers of frequency floods (or numbers of maximum RWSs with maximum discharges) have been evaluated to select an IDF (or a design maximum RWS with a design maximum discharge) **(box 12),** which is discussed in more detail in the next step. As previously noted, an acceptable level of risk will typically be an area of the f-N chart indicating decreasing justification to take action. There may be situations where the total modified risk is in an area of the f-N chart indicating decreasing justification to reduce risk, but a decision is made that further risk reduction is appropriate (for example, a modest increase in modification cost would significantly reduce total risks, or perhaps there is a potential for future increased downstream consequences, which would increase risks to unacceptable levels). The designer of record can recommend an acceptable level of risk, but Reclamation management (decisionmakers) must concur with the recommendation. Consideration will include the level of uncertainty and confidence associated with estimates and future conditions that could affect the estimates.

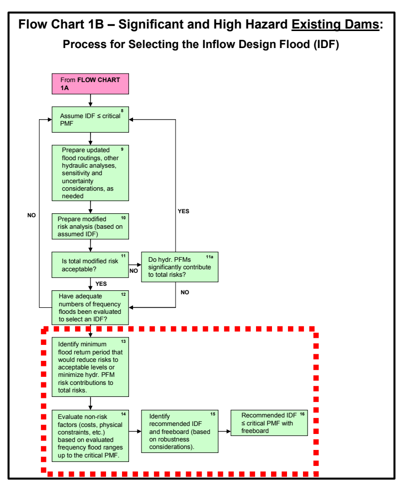
7. **Actions Related to Total Modified Risks; Hydrologic PFMs Risk Contribution to Total Modified Risks; and Whether Sufficient Information to Select an IDF Exists.** This discussion addresses **Flow Chart 1B boxes 11a and 12,** and recycling through **boxes 8 through 12**.



If it is determined that the hydrologic PFM risks do not significantly contribute to the total modified risks **(box 11a)**, a decision is made as to whether adequate representation of frequency floods have been evaluated to select an IDF (or a design maximum RWS with a design maximum discharge) **(box 12)**. Multiple assumed dam/spillway configuration scenarios, each represented by an IDF or a maximum RWS with maximum discharge, will be used to estimate associated modified risks. This effort is needed to identify total risk trends, which are used to determine acceptable risks. If there is not enough information to identify acceptable risks, part of Flow Chart 1B (**boxes 8 through 12**) is repeated until sufficient information to select an IDF (or a design maximum RWS with a design maximum discharge) has been developed. Refer to Appendix A, Examples 1 and 3, for further information about evaluating multiple assumed IDFs. Repeating part of Flow Chart 1B will involve reassessing and redesigning the nonstructural and structural features. This effort generally involves evaluating a suite of possible redesigns, including but not limited to: changes in reservoir operations, raising the existing dam, increasing the discharge capacity of existing appurtenant structures, or constructing new appurtenant structures. The evaluation of these possible redesigns would include appropriate analyses and designs (typically associated with CAS, feasibility, and/or final design level studies), including flood routings of flood events up to and including the current critical PMF.

If there is sufficient information to select an IDF (**box 12**), then the minimum flood return period (or the maximum RWS with the maximum discharge) that would reduce risks to acceptable levels is identified (**box 13**), which is discussed in more detail in the next step. Additionally, under this condition, other credible PFM risks due to static and/or seismic loadings may be significant contributors to the total modified risks. Other actions (not associated with selecting the IDF [or a design maximum RWS with a design maximum discharge]) to address static and/or seismic risks may be warranted but are not part of this chapter. Guidance can be found in various chapters of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

8. **Actions if There is Sufficient Information to Select an IDF.** This discussion addresses **Flow Chart 1B, boxes 13 through 16.**



From the previous effort (**box 12**), the minimum flood return period (or the lowest maximum RWS with the maximum discharge) that would reduce total modified risks to acceptable levels is identified **(box 13)**. It should be highlighted that evaluation of the static and seismic risk contributions will be needed to identify the minimum flood return period (or the lowest maximum RWS with the maximum discharge) that would reduce total modified risks to acceptable levels.

Prior to recommending an IDF (or a design maximum RWS with a design maximum discharge), further evaluation takes place to assess the nonrisk factors such as cost, physical constraints associated with the dam, policies, etc. **(box 14)**. These factors could change (reduce or increase) the identified minimum flood return period (or the lowest maximum RWS with maximum discharge) that would reduce risks to acceptable levels or minimize hydraulic PFM risk contributions to total risks.

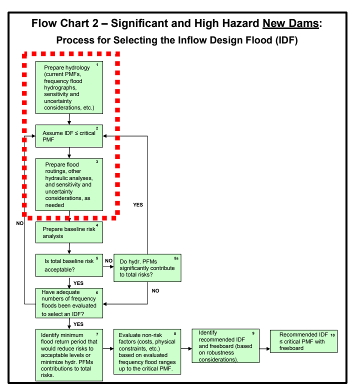
Using the frequency flood level identified as the recommended IDF (or a design maximum RWS with a design maximum discharge) (**box 14**), a robustness study will be conducted to evaluate the uncertainties associated with the flood, operations, future events, etc. (refer to section 2.4.2.3 for more details) **(box 15)**. The existing dam with modifications and robustness considerations (which is used to determine freeboard requirements) should safely accommodate a frequency flood equal to or less than the current critical PMF (or the PMF-induced maximum RWS with the maximum discharge). This frequency flood is the recommended IDF (or a design maximum RWS with a design maximum discharge) **(box 16)**. As previously noted, the recommended IDF (or a design maximum RWS with a design maximum discharge) must be concurred with by Reclamation management (decisionmakers), which will involve appropriate documentation[7] and presentation.[8]

2.4.2.2 New Dams

The following text discusses Flow Chart 2 (Significant and High Hazard New Dams: Process for Selecting the Inflow Design Flood, IDF).

1. **Prepare Hydrology/Hydraulics.** This discussion addresses **Flow Chart 2, boxes 1, 2 and 3**.

   Unlike an existing dam, where maximum loading conditions are determined, analyses and designs are based on targeted (selected) design loading conditions for a new dam.



[7] Typically including decision document and technical report of findings, along with supporting technical memoranda.

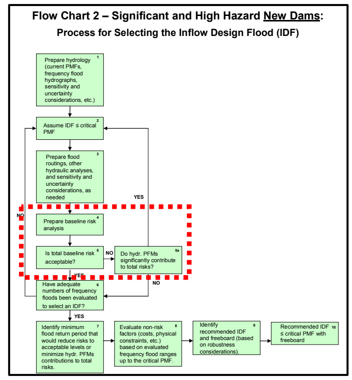
[8] Typically involves a Dam Safety Advisory Team (DSAT) meeting where recommendations are presented, discussed, and concurred with or rejected.

To begin the process, hydrology is prepared for the new damsite **(box 1)**. This could include preparing current PMFs along with frequency floods associated with the PMFs. Hydrologic sensitivity and uncertainty considerations should be evaluated, which may be similar to the robustness study (see Section 2.4.2.3, "Address Uncertainties with Freeboard (Robustness) Study, Existing and New Dams" in this chapter), but they are addressed prior to selecting an IDF (or a design maximum RWS with a design maximum discharge) and are integrated into the baseline risk analysis. The type and amount of updated hydrology will be site specific, and coordination with the TSC's Flood Hydrology and Consequences Group will be required.

As a starting point, the current critical PMF (determined from flood routings) or the maximum (most remote) frequency flood available is the initial design hydrologic loading condition **(box 2)**.

Once the hydrology is available, flood routings and other hydraulic analyses would be performed to determine the response of the new dam **(box 3)**. The flood routing and other hydraulic analyses are used to select the type, size, and location of the new dam and appurtenant structure (spillway and/or outlet works). Additional guidance for selecting the type, size, and location of the modified or new appurtenant structure can be found in Chapter 3, "General Spillway Design Considerations," and Chapter 4, "General Outlet Works Design Considerations," of this design standard. It should be noted that sensitivity and uncertainty considerations come into play. These sensitivity and uncertainty considerations may be similar to the robustness study (see Step 4 of Flow Chart 2 and refer to Section 2.4.2.3, "Address Uncertainties with Freeboard (Robustness) Study, Existing and New Dams" in this chapter), but they are addressed prior to selecting an IDF (or a design maximum RWS with a design maximum discharge) and are integrated into the baseline risk analysis. Also, the flood routings would use a range of starting RWSs with the maximum starting RWS either at the top of active conservation or top of joint use storage, whichever is higher. The minimum starting RWS would typically reflect planned reservoir operations in terms of how low the reservoir might reasonably be during the time of year for the flood event. Other hydraulic analyses typically performed include water surface profiles to assess: potential overtopping of spillway chute walls and/or pressurizing of conduits/tunnels; flow cavitation potential; and stagnation pressure (hydraulic jacking) potential. Additionally, downstream inundation mapping and associated estimated population at risk will be needed for subsequent steps of Flow Chart 2. The dam and appurtenant structures are initially sized based on assumed targeted design loading conditions.

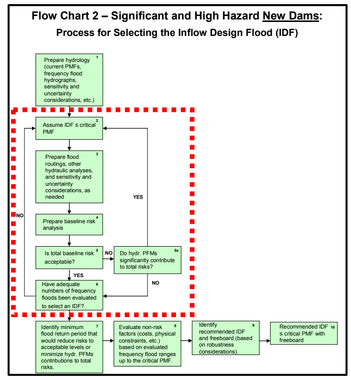
2. **Prepare Initial Total Baseline Risks.** This discussion addresses **Flow Chart 2, boxes 4, 5, and 5a**.



All credible PFMs for all loading conditions are identified and evaluated for the new dam and appurtenant structures. A baseline quantitative risk analysis using current risk methodology [3, 7, 8, and 9] is prepared (**box 4**).

If the total baseline risks (the total annualized failure probability and/or the total annualized life loss) are not acceptable **(box 5)**, further evaluation of the hydraulic PFM risk contributions to the total baseline risks is made **(box 5a)**. Note: For the hydrologic PFMs, incremental consequences should be used, which refer to the difference of the life loss estimates for the dam failure and life loss estimates for maximum nondam failure releases. The term "acceptable" typically means that the total baseline risk is in an area on the f-N chart indicating decreasing justification to take action to reduce risks. However, it should be noted that risks below Reclamation guidelines are not necessarily acceptable. Other risk and nonrisk factors such as uncertainty, confidence, cost, physical constraints, etc., will have a bearing on identifying acceptable risks.

If the total baseline risks are acceptable **(box 5)**, a determination is made as to whether adequate representation of frequency floods (or numbers of maximum RWSs with maximum discharges) has been evaluated to select an IDF (or a design maximum RWS with a design maximum discharge) **(box 6),** which is discussed in more detail in the next step. As previously noted, an acceptable level of risk will typically be an area of the f-N chart indicating decreasing justification to take action. There may be situations where the total baseline risk is in an area of the f-N chart indicating decreasing justification to reduce risk, but a decision is made that further risk reduction is appropriate (for example, a modest increase in cost would significantly reduce total risks, or there is a potential for future increased downstream consequences to increase risks to unacceptable levels). The designer of record can recommend an acceptable level of risk, but Reclamation management (decisionmakers) must concur with the recommendation. Consideration will include the level of uncertainty and confidence associated with estimates and future conditions that could affect the estimates. Also, for new dams, the maximum practical frequency flood event (including the current critical PMF) as the IDF should be selected for design. There may be great cost efficiency in achieving more remote risks than there would be for modifying existing dams.

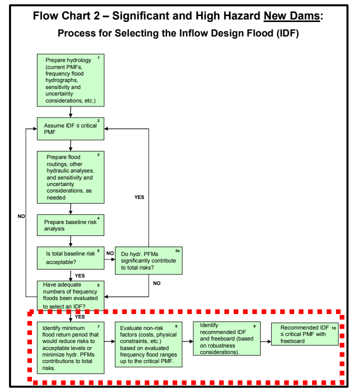


3. **Actions Related to Total Baseline Risks; Hydrologic PFMs Contribution to Total Baseline Risks; and Whether Sufficient Information to Select an IDF Exists.** This discussion addresses **Flow Chart 2, boxes 5a and 6,** and recycling through **boxes 2 through 6**.

   If it is determined that the hydrologic PFM risks do not significantly contribute to the total baseline risks **(box 5a)**, a determination is made as to whether adequate representation of frequency floods has been evaluated to select an IDF (or a design maximum RWS with a design maximum discharge) **(box 6)**. Multiple assumed IDFs equal to a range of frequency floods (or multiple maximum RWSs with maximum discharges) will be used to estimate associated baseline risks. This effort is needed to identify total risk trends, which are used to determine acceptable risks. If there is not enough information to identify acceptable risks, part of Flow Chart 2 (**boxes 2 through 6**) is repeated until sufficient information to select an IDF (or a design maximum RWS with a design maximum discharge) has been developed. Refer to Appendix A, Examples 1 and 3, for further information about evaluating multiple assumed IDFs. Repeating part of Flow Chart 2 will involve redesigning the nonstructural and structural features. This effort generally involves evaluating a suite of possible redesigns, including but not limited to: changes in reservoir operations, raising the new dam, increasing the discharge capacity of new appurtenant structures, or constructing additional appurtenant structures. The evaluation of these possible redesigns would include appropriate analyses and designs, including flood routings of flood events up to and including the current critical PMF.

   If there is sufficient information to select an IDF (or a design maximum RWS with a design maximum discharge) (**box 6**), then the minimum flood return period (or lowest maximum RWS with maximum discharge) that would reduce risks to acceptable levels is identified (**box 7**), which is discussed in more detail in the next step. Additionally, under this condition, other credible PFM risks due to static and/or seismic loadings may be significant contributors to the total risks. Other actions (not associated with selecting the IDF [or a design maximum RWS with a design maximum discharge]) to address static and/or seismic risks may be warranted, but they are not part of this chapter. Guidance can be found in various chapters of Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7].

4. Actions if There is Sufficient Information to Select an IDF. This discussion addresses Flow Chart 2, boxes 7 through 10.



From the previous effort (**box 6**), the minimum flood return period (or the lowest maximum RWS with maximum discharge) that would reduce total baseline risks to acceptable levels is identified **(box 7)**. It should be highlighted that evaluation of the static and seismic risk contributions will be needed to identify the minimum flood return period (or the lowest maximum RWS with maximum discharge) that would reduce total baseline risks to acceptable levels. As previously noted, for new dams, the maximum practical frequency flood event (including the current critical PMF) as the IDF should be selected for design. There may be great cost efficiency in achieving more remote risks than there would be for modifying existing dams.

Prior to recommending an IDF (or a design maximum RWS with a design maximum discharge), further evaluation is done to assess the nonrisk factors such as cost, physical constraints associated with the dam, policies, etc. **(box 8)**. These factors could change (reduce or increase) the identified minimum flood return (or the lowest maximum RWS with maximum discharge) period that would reduce risks to acceptable levels or minimize hydraulic PFM risks contributions to total risks.

Using the frequency flood level identified as the recommended IDF (or a design maximum RWS with a design maximum discharge) (**box 8**), a robustness study will be conducted to evaluate the uncertainties associated with the flood, operations, future events, etc. (refer to Section 2.4.2.3, "Address Uncertainties with Freeboard (Robustness) Study, Existing and New Dam," in this chapter) **(box 9)**. The new dam with redesigns and robustness considerations should safely accommodate a frequency flood equal to or less than the current critical PMF. (Note: For new dams and their appurtenant structures, there may be compelling nonrisk factors, such as minimal cost increases, to design for the PMF, even if risk considerations support designing for a flood event less than the PMF). This frequency flood (or the maximum RWS with the maximum discharge) is the recommended IDF (or a design maximum RWS with a design maximum discharge) **(box 10)**. As previously noted, the recommended IDF (or a design maximum RWS with a design maximum discharge) must be concurred with by Reclamation management (decisionmakers), which will involve appropriate documentation[9] and presentation.[10]

2.4.2.3 Address Uncertainties with Freeboard (Robustness) Study (Existing and New Dams)

Once an IDF (or a design maximum RWS with a design maximum discharge) has been selected, uncertainties are incorporated into the design process, which is referred to as a "robustness study." These uncertainties may be related to the method of estimating the IDF return period (or a design maximum RWS with a design maximum discharge) and the size and shape of the IDF hydrograph, reservoir and dam operations, gated spillway operations, reduction of spillway discharge capacity due to debris, and other mechanisms and future events associated with upstream and downstream developments. To account for these uncertainties, plausible "what-if" scenarios are evaluated. It should be highlighted that only what-if scenarios that have not been considered and evaluated as part of the risk analysis will be part of this study. These scenarios could create an elevated RWS above the design maximum RWS and might include:

- **Misoperation of gated spillways.** This what-if scenario relates to human error associated with how gates are opened and closed. Examples might include gate operations that are not in accordance with operating documents such as the Standing Operating Procedures (SOP). To account for this what-if scenario, it is typically assumed that gates are either not opened enough or there is a delayed operation which can be approximated by reducing the discharge capacity. Based on judgment and considering site-specific conditions, the rate of discharge change can be simulated by slowing the rate of gate opening. As an example, the rate of gate opening might be changed from 5:1 to 2:1 (ratio of gate opening to reservoir rise). The level of reduced discharge capacity is based on judgment which accounts for site-specific conditions. A delay in operations could occur for dams with a flood control pool and restricted spillway releases within this pool. Once the reservoir water surface exceeds the top of the flood control pool, restricted releases are no longer required, and spillway flows can typically be increased up to the level of flood inflows. It is possible that the dam operator will be reluctant to exceed the restricted inflows at the allowable RWS elevation, especially if downstream residents are already being impacted by spillway releases. This can be evaluated by applying a delay in resuming unrestricted releases, either by assuming a

[9] Typically including decision document and technical report of findings, along with supporting technical memoranda.

[10] Typically involves a DSAT meeting where recommendations are presented, discussed, and concurred with or rejected.

time delay (such as 1 to 3 hours) or assuming that the switch to unrestricted releases occurs at a higher elevation (such as 1 to 2 feet above the top of the flood control pool).

- **Mechanical/electrical malfunction of gated spillways.** This what-if scenario relates to potential malfunction of gates that is not associated with human error. Even with regular maintenance of equipment and appropriate attendance by an operator, mechanical and/or electrical failure could occur. Some of the considerations include: reliability of gate operations from actual experience and/or historical performance of this type of gate; size and complexity of the mechanical/electrical features; number of gates; reliability of commercial and auxiliary power supply; and availability of emergency equipment and materials to open spillway gates without relying on the existing hoist system. To account for this what-if scenario, it is typically assumed that one or more gates remain closed or not opened enough, which can be approximated by reducing the discharge capacity. The level of reduced discharge capacity is based on judgment accounting for site-specific conditions. General guidelines would initially assume a 10- to 50-percent reduction in total discharge capacity.

- **Change in hydrology (methodology, watershed changes, and climate changes).** This what-if scenario relates to potential future changes in the hydrologic data and methods used to develop flood loadings. To account for this what-if scenario, the size (inflow peak and/or volume) of the IDF may be increased by 5 to 10 percent and routed through the reservoir and dam. Percentages should be verified in consultation with a hydrologist, and other percentages could be considered. Uncertainty of the hydrologic loadings also needs to be considered.

- **Debris blockage of hydraulic structures (spillways, outlet works, etc.).** This what-if scenario relates to potential blockage of the spillway and/or outlet works by debris during a flood event, which can result in reduced discharge capacity. Assuming that there are debris sources within the watershed, this what-if scenario would be more likely to affect surface structures such as those associated with a spillway; in particular, gated control structures. Also, uncontrolled control structures such as ogee crests with widths of less than 40 feet (less than 60 feet in the Pacific Northwest) [7] and morning glory control structures tend to be susceptible to debris blockage. General guidelines would initially assume a 10- to 50-percent reduction in total discharge capacity, with possible adjustments based on spillway geometries and potential debris loads.

- **Failure of upstream dams.** This what-if scenario relates to evaluating the potential failure of upstream dams during a flood event. This is typically limited to upstream Reclamation dam failure impacts on the dam being evaluated. However, on a case-by-case basis, upstream non-Reclamation dam failure impacts on the dam being evaluated may be considered. General guidelines would be to conduct a flood routing through the upstream dam and, if it is likely to fail, develop a breach hydrograph which would be routed through the reservoir impounded by the dam being evaluated.

- **Change in upstream and/or downstream consequences.** This what-if scenario relates to potential future increases in upstream and/or downstream population at risk and subsequent increase in economic damage and potential life loss. Specifically, if additional future developments occur, the population at risk could increase. This could lead to encroachment into the reservoir area and/or into the downstream flood plain. To evaluate this what-if scenario, assumed limitations on flood-induced reservoir rise (which could result in increased releases during the flood) and/or reduced releases (which could result in increased reservoir rise during the flood) are evaluated. The amount of reduced reservoir rise and/or reduced releases is site specific.

- **Flood events that differ from the original basis for the IDF (i.e., series of storms versus one storm).** This what-if scenario relates to potential future changes in the hydrologic data and methods used to develop flood loadings. To account for this what-if scenario, different types of flood hydrographs (such as scaled historical events, thunderstorm events, series of storms, etc.) with different shapes (such as front-end loaded, with/without antecedent event, etc.) as those used for the IDF could be evaluated.

- **Wind-generated waves (runup and setup).** This what-if scenario relates to assessing the potential wave sizes that could be generated by winds occurring over the normal, maximum, and intermediate water surface elevations. For details, the reader is directed to Design Standard No. 13, Embankment Dams, Chapter 6, " Freeboard." For new embankment dams, this reference includes checks to determine if there is adequate minimum and normal freeboard for typical (10-percent hourly exceedance) and extreme (100-mile-per-hour) wind events, respectively. For an existing embankment dam, this reference includes two similar checks to determine if there is enough existing freeboard to prevent wind-generated wave loads from washing over the dam from the normal and maximum RWSs.

- **Security concerns.** This what-if scenario relates to assessing deliberate attempts to breach a dam and/or hydraulic structures that would lead to uncontrolled release of part of or the entire reservoir. Security concerns should be considered when freeboard is being determined for a dam. Reclamation's Safety, Security, and Law Enforcement (SSLE) Office can provide guidance.

Results of the robustness study will be considered when recommending freeboard requirements for either an existing or new storage or multipurpose dam, along with maximum discharge potential for hydraulic structures such as spillways. In some cases (such as changes in hydrology), the maximum discharge potential could exceed the maximum design discharge, which could lead to potential hydraulic and/or structural issues that should be further evaluated as part of the design. The robustness study will indicate estimated maximum RWS elevations for a number of different scenarios. This result should be used to guide, but not dictate, the recommended freeboard. To maintain reasonableness with the freeboard study (given that some results associated with extreme hydrologic events are very unlikely to occur), the recommended freeboard may not have to accommodate the maximum RWS elevations from all what-if scenarios. If the maximum RWS elevations from a few what-if scenarios exceed the level of all other what-if scenarios, and the outlier scenarios are judged to be less likely than the other what-if scenarios, there may be justification for recommending freeboard that accommodates all except the outlier what-if scenarios. If a number of what-if scenarios are evaluated, and all but one what-if scenario has maximum RWS elevations that are tightly clustered, it may be reasonable to set aside the outlier what-if scenario. Appendix A includes examples of robustness studies.

Other considerations that will influence the final freeboard requirements associated with the design maximum RWS for the modification of an existing dam or for design of a new dam include:

- **For a new embankment dam,** the minimum freeboard (above the design maximum RWS) should be 3 feet or a larger value based on considerations from the robustness study. Also, parapet walls should not be used to impound a portion of the reservoir and should only provide freeboard associated with wind-generated wave runup (for more details, see *Design Standard No. 13, Embankment Dams,* Chapter 6, "Freeboard"). If parapet walls are used to address wind-generated waves, the following safeguards must be met:

    - The parapet wall should be adequately tied into the impervious zone and the abutments of the dam.

    - Proper zones should be provided around and beneath the parapet wall to prevent undercutting and piping.

  - Future foundation and dam settlement that would adversely affect the structural integrity of the parapet wall must be provided for during design and construction.
  - Hydrostatic and hydrodynamic (wave) loads must be accounted for.
  - Drainage off the dam crest around or through the parapet wall must be accounted for.
  - Connecting and sealing the parapet wall sections together with each other and each end of the dam must be addressed.
  - Safety and security must be addressed.
  - Maintenance, snow and ice removal, sight lines, and aesthetics must be addressed.

- **For an existing embankment dam being modified,** a minimum freeboard (above the design maximum RWS) should be recommended based on the robustness study results with consideration given to the cost for various levels of protection, sensitivity to various and critical uncertainties, and other relevant factors. Use of parapet walls to provide freeboard may be considered on a case-by-case basis. Again, parapet walls typically should not be used to impound a portion of the reservoir and only provide freeboard associated with wind-generated wave runup (for more details, see *Design Standard No. 13, Embankment Dams,* Chapter 6, "Freeboard"). However, in some rare cases, parapet walls can be used to impound a portion of the reservoir, and must meet similar safeguards as noted by the previous bullet.

- **For a new concrete dam or an existing concrete dam being modified,** a minimum freeboard (above the design maximum RWS) should be recommended based on the robustness study results. If overtopping failure of a concrete dam is judged to be remote, minimal or no freeboard may be appropriate (however, if the reservoir is normally kept near the top of dam, additional freeboard may be considered to prevent waves from lapping over the dam on a regular basis). Use of parapet walls to provide freeboard may be considered on a case-by-case basis, including impounding a portion of the reservoir. If parapet walls are used to address freeboard needs, the following safeguards must be met:

  - Hydrostatic and hydrodynamic (wave) loads must be accounted for.
  - Drainage off the dam crest around or through the parapet wall must be accounted for.

- Connecting and sealing the parapet wall sections together with each other and each end of the dam must be addressed.
- Safety and security must be addressed.
- Maintenance, snow and ice removal, sight lines, and aesthetics must be addressed.

2.4.2.4 Technical References

Technical references by Reclamation associated with selection of the inflow design flood include:

- *Interim Guidelines for Addressing the Risk of Extreme Hydrologic Events* [3]
- *Design of Small Dams*, third edition [4]
- *Dam Safety Risk Analysis Best Practices Training Manual* [7]
- *Interim Dam Safety Public Protection Guideline* [8]
- DSO-99-06 – *A Procedure for Estimating Loss of Life Caused by Dam Failure* [9]
- *Guidelines for Evaluating Hydrologic Hazards* [11]
- EM No. 9 – *Discharge Coefficients for Irregular Overfall Spillways* [19]
- EM No. 25 – *Hydraulic Design of Stilling Basins and Energy Dissipators* [20]
- REC-ERC-88-3 – *Overtopping Flow on Low Embankment Dams – Summary Report of Model Test* [21]
- REC-ERC-78-8 – *Low Froude Number Stilling Basin Design* [22]
- Assistant Commissioner – Engineering and Research (ACER) Technical Memorandum (TM) No. 10 – *Guidelines for Using Fuseplug Embankments in Auxiliary Spillways* [23]
- *Hydraulic and Excavation Tables*, 11th edition [24]
- *Computing Degradation and Local Scour* [25]

- *Guide for Computing Water Surface Profiles* [26]
- *Plastic Pipe Used In Embankment Dams: Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [27]
- *Outlet Works Energy Dissipators: Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [28]
- EM No. 14 – *Beggs Deformeter-Stress Analysis of Single-Barrel Conduits* [29]
- EM No. 14 Supplement – *Beggs Deformeter-Analysis of Additional Shapes* [30]
- EM No. 27 – *Moments and Reactions for Rectangular Plates* [31]
- EM No. 34 – *Control of Cracking in Mass Concrete Structures* [32]
- *Concrete Manual,* eighth edition [33]
- *Reinforced Concrete Design and Analysis Guidelines* – working draft [34]
- *Design Criteria for Retaining Walls* [35]
- *Roller-Compacted Concrete: Design and Construction Considerations for Hydraulic Structures* [36]
- EM No. 42 – *Cavitation in Chutes and Spillways* [40]
- REC-ERC-73-5 – *Hydraulic Model Studies of Chute Offsets, Air Slots, and Deflectors for High-Velocity Jets* [42]
- DSO-07-07 – *Uplift and Crack Flow Resulting from High Velocity Discharge Over Offset Joints* [43]

2.4.2.5 Examples

Appendix A provides examples with additional details for selecting the IDF. These examples include:

- **Example No. 1. – Dam G1 Modification Final Design.** Presents the process of selecting an IDF less than the current critical PMF for an existing embankment dam, existing multiple embankment dikes, existing service spillway, and existing outlet works.

- **Example No. 2. – Dam F.** Presents the process of selecting an IDF equal to the current critical PMF for an existing composite dam, existing multiple dikes, existing auxiliary embankment dam, existing service and emergency spillway, and existing outlet works.

- **Example No. 3. – Dam W.** Presents the process of selecting an IDF equal to the current critical PMF for a new embankment dam, new service spillway, and new outlet works.

2.5 Construction Diversion Floods

The following text provides a historical perspective (background) and the current process for identifying and/or selecting construction diversion floods used by Reclamation. It should be noted that the current process is applicable to high and significant hazard dams and associated appurtenant structures, including most Reclamation storage and multipurpose dams, along with some diversion and detention dams (see Section 2.5.1.4, "Current Approach (Reclamation)," in this chapter). The current process can also be considered for low hazard dams (mostly diversion and detention dams). In some cases when dealing with low hazard dams and their appurtenant structures, a simplified approach can be used that is similar to Reclamation's historical approach (see Section 2.5.1.1, "Historical Approach (Reclamation)," in this chapter).

Both annual and seasonal frequency floods may be used for evaluating diversion during construction (see Section 2.2.1.5, "Construction Diversion Floods," in this chapter). Generally speaking, seasonal frequency floods (such as rain-on-snow events associated with a 4-month period or summer thunderstorm event associated with a 3-month period of the year) are used when a refined evaluation of the construction schedule is needed to avoid or limit critical construction activities during the portion of the year associated with the greatest flood potential. Annual frequency floods are used when there is not much change in flood potential during the year, or when the risk exposure period (construction season) exceeds 1 year. Also, a conservative approach that has been used is to assume annual frequency floods apply to the part of the year associated with the greatest flood potential.

It should be highlighted that this section only applies to the identification and/or selection of the construction diversion flood, not the diversion method or who will be responsible for the diversion method (either Reclamation or the contractor). Reclamation will decide who is responsible for the diversion method (which involves diversion and care of the stream or river during construction) prior to the start of a final design.

2.5.1 Background

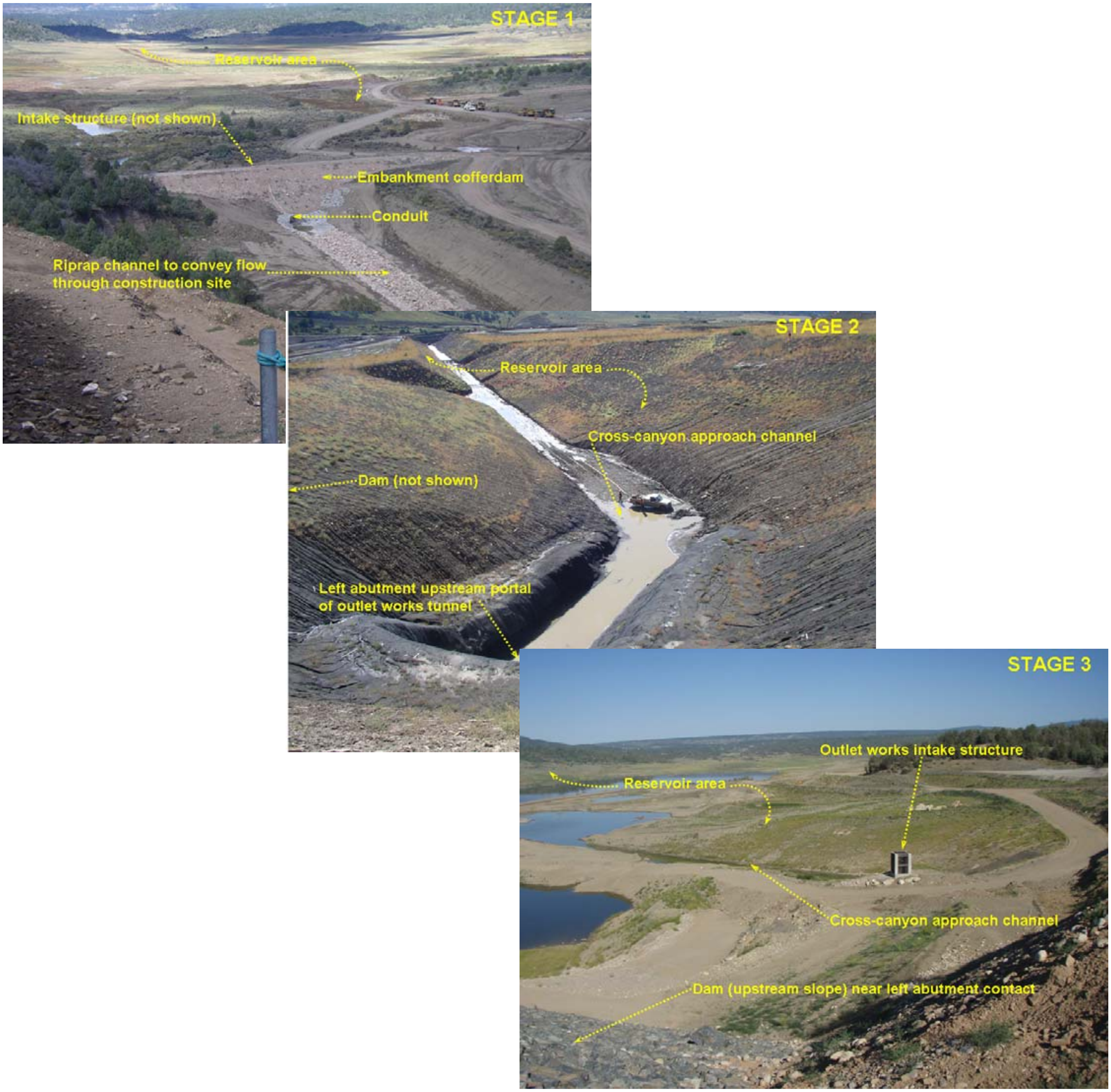
At any time construction activities occur in and/or around streams or rivers, consideration must be given to safely accommodating both normal streamflow and flood events during the construction period (i.e., diverting flows through and/or around the construction area with no or limited impacts to construction efforts and the downstream area). It is noted that for existing dams that are being modified, flows during construction are typically released through the permanent appurtenant structures, and operations may be the same as they were prior to construction. Diversion methods typically represent a balancing between cost of the diversion method and risk associated with a larger flood occurring (larger than the floods used to size the diversion features) [4, 13]. Refer to figures 2.5.1-1 through 2.5.1-3 for illustrations of diversion methods used by Reclamation and its contractors at some projects. The diversion method during construction accounts for the following considerations:

- historical streamflow;
- construction diversion flood type (rain-on-snow, thunderstorm, etc.), size (peak and volume), and frequency of occurrence (return period);
- site-specific conditions; and
- specifications requirements (i.e., diversion methods are either the contractor's or the designer's responsibility, or a shared responsibility).

2.5.1.1 Historical Approach (Reclamation)

Based on the previous considerations, Reclamation historically selected and/or identified the annual construction diversion flood types and frequencies for new dam construction as general storm events and used a rule-of-thumb of five times the construction period to select an annual construction flood return period. This rule-of-thumb typically resulted in selecting a 5-, 10- or 25-year annual frequency flood, which was used to select and size the diversion method, including cofferdam and/or conveyance feature type and size. This historical approach, which focused more on impacts to the ongoing construction, rather than on the safety of the downstream public, is infrequently used by Reclamation. If no other considerations apply (such as risk), this approach can be used to establish a minimum annual or seasonal construction diversion flood event. When using this method, consideration should be given to the following:

- **Minimum return period.** A minimum 5-year annual or seasonal flood event should be initially considered for construction periods less than or equal to 1 year. In some rare cases, a smaller more frequent annual or seasonal flood event may be considered where it is impractical to accommodate a 5-year flood event.



Figures 2.5.1-1. Example: Construction diversion method. Stage 1 - upper photograph: right abutment upstream embankment cofferdam, intake structure (not shown), conduit through the embankment cofferdam, and riprap open channel through the construction site used prior to construction of the outlet works through the left abutment. Stage 2 – center photograph: cross-canyon approach channel from right abutment to left abutment outlet works tunnel prior to construction of the intake structure. Stage 3 – lower photograph: cross-canyon approach channel directing flow to the left abutment outlet work intake structure and tunnel. Ridges Basin Dam site, Colorado.

Figures 2.5.1-2. Example: Two-stage construction diversion method involving the first stage (season 1). Upper photograph: construction area immediately upstream of the right half of the existing concrete dam is isolated from the reservoir via sheetpile crib wall and cellular cofferdam. Center photograph: unwatered/dewatered construction area in foreground and combination sheetpile crib wall and cellular cofferdam. Lower photograph: construction area in foreground and the left half of the existing dam outlets. Jackson Lake Dam, Wyoming.

Figures 2.5.1-3. Example: Construction diversion method. Upper photograph: excavated rock channel without cofferdam through the construction site. Center photograph: access over the excavated rock channel. Lower photograph: close-up of the excavated rock channel. Upper Stillwater Dam site, Utah.

- **Risk level during construction.** A form of the Binomial distribution referred to as the Bernoulli's sequence or distribution (see Section 2.5.1.2, "Probabilistic Approach," in this chapter) provides a means to correlate the flood frequency selected (return period) for the diversion system design in this rule-of-thumb method. For example, a 5-year annual flood event for a construction period of 1 year would be associated with an exceedance probability of 20 percent. In other words, this simple historical approach would be equivalent to accepting a 20-percent chance of an annual flood event more remote and larger than a 5-year return period occurring during the 1-year construction period.

- **Risk exposure period.** Several factors regarding this historical approach must be considered. Part of the Reclamation design process will involve developing a construction schedule. In many cases, the risk exposure period is much less than the entire construction period. Such a risk exposure period might be associated with excavating the foundation for, and constructing, a spillway control structure. The foundation can be excavated, and the control structure constructed, so that a cofferdam or retention of a rock or earthen plug in the upstream approach channel can provide flood protection during this time period. Once the control structure is in place, it may be able to serve as the cofferdam and/or can operate during flood events. When this is possible, the risk exposure period may be considerably less than the entire construction period. For example, application of the special form of the Binomial distribution (Bernoulli distribution) demonstrates that the probability of exceeding a prescribed annual flood event is significantly reduced when a 5-year annual flood event is used for cofferdam design. If the risk exposure time is reduced to 3 months, the likelihood (probability) of the flood event occurring is about 5 percent within the risk exposure period of 3 months. For more details about calculating the probability of the flood event occurring (P_n), the reader is directed to Section 2.5.1.2, "Probabilistic Approach," in this chapter.

- **Risk level reduction during construction.** In many cases, it does not require significant cost (in terms of construction time or diversion system costs) to markedly reduce risk (i.e., reduce the exceedance probability) for flooding of the construction site. Such risk reduction may be accomplished by reducing the time of exposure to a flood event (as noted in the previous paragraph) or selecting a larger flood event (return period) for design of a diversion system. For example, the increase in the height of a cofferdam may be minimal to attain protection for the 10-year (0.10- or 10-percent chance of being exceeded in any year) or 25-year annual flood event (0.04- or 4-percent chance of being exceeded in any year). This should always be evaluated when considering construction flood protection.

2.5.1.2 Probabilistic Approach

Others [14, 15] have estimated construction diversion floods using the Binomial distribution and Geometric distribution (special form of the Binomial distribution that is referred to as the Bernoulli's sequence or distribution). The Binomial distribution is used in cases where the time period is more frequent or the probabilities are large ($P > 0.1$). The Binomial distribution is expressed by the following equation:

$$P_n = \binom{n}{r} P_a^{\,r} (1 - P_a)^{n-r} \; where: \binom{n}{r} = \left(\frac{n!}{r!(n-r)!} \right)$$

Where: P_a = annual exceedance probability of an event occurring (such as a flood).

P_n = probability that an event (such as a failure) occurs in n years (failure probability).

n = time period (in years) being considered, and must be integers (whole numbers) when dealing with factorial conditions ($n!$).

r = number of times that a given event (such as a flood) occurs in n years, and must be integers (whole numbers) when dealing with factorial conditions ($r!$).

For the case where an event (such as a flood) does not occur ($r = 0$) in n years, the Binomial distribution can be simplified and expressed by the following equation:

$$P_n = (1 - P_a)^n$$

Furthermore, for the case where one or more events (such as floods) occur in n years, the equation is:

$$P_n = 1 - (1 - P_a)^n$$

This expression is a special form of the Binomial distribution and is referred to as a Bernoulli sequence of events. In this case, the time to the first event (success or failure) results in a Geometric distribution which is a special case of the Binomial distribution. The Bernoulli sequence of event equation can be manipulated to solve directly for the annual exceedance probability (P_a) by the following equation:

$$P_a = 1 - (1 - P_n)^{\frac{1}{n}}$$

It is noted that Reclamation has taken license with applying integers and decimals to "*n*" when using this Bernoulli distribution (i.e., for a 3 month period, $n = 3/12 = 0.25$ years or for a 27 month period, $n = 2.25$ years). This application provides approximate results.

The reciprocal of the annual exccedance probability (P_a) for a flood event is the return period in years (T_a) and expressed by the following equation:

$$T_a = \frac{1}{P_a}$$

The formula for P_a can be used to easily determine the annual probability or return period T_a of the flood, given an assumed construction period n and failure probability P_n. The formula for P_n can be used to determine the annual failure probability (risk), given an assumed construction period n and the annual probability P_a or return period T_a of the flood.

Two different approaches using the special form of the Binomial and Geometric distributions are presented in the following text:

- The Institute of Civil Engineering cites an example where a dam is classified as a category A or B (similar to a high hazard dam). For this category of dam, a diversion system must safely pass annual floods which have only a 1-percent chance of being exceeded during the critical construction period. For the dam modification under consideration, it has been determined that the critical timeframe is during a 30-month period or 2.5 years. Using the special form of the Binomial distribution, an annual diversion flood with a return period of about 250 years would be necessary to meet the requirement of a 1-percent chance of exceedance during the critical construction period [16]. Appendix B presents an application of this approach (see Example 2: Dam R Feasibility Level Study). This method does not distinguish between dams with varying potential for loss of life; therefore, it is not a fully risk-informed approach.

- The USACE cites an example of a new lock and dam being constructed over a 3-year period on a river where navigation and normal flows will be maintained. The special form of the Binomial distribution was used to estimate a range of annual frequency floods occurring during the construction period (49-percent chance for a 5-year annual flood event to a 3-percent chance for a 100-year annual flood event). These estimates were then used to calculate a range of total probable flood costs, which are the product of a flood occurring during the 3-year construction period (results from the special form of the Binomial distribution), and the cost of work stoppage and cleanup due to flooding. Also, these probable flood costs are compared to diversion costs (cofferdam type and size) associated with an annual frequency flood event. The annual construction diversion flood event is identified where the incremental probable flood costs and incremental diversion costs are approximately the same. In this case, a 12-year annual event was selected as the construction diversion flood level [17]. Reclamation has employed some of these concepts in appendix B (see Example 2: Dam R Feasibility Level Study). This method is focused on impacts to the ongoing construction, and it does not account for impacts to the downstream population at risk; but it is a risk-informed approach based on economic considerations.

2.5.1.3 Risk-Based Approach

The New South Wales government does endorse quantitative risk analysis methodology in addressing construction risks [18], which is similar to Reclamation's approach. Its approach is summarized in the following paragraphs.

Flood capacity during construction of new dams is an area in which risk assessment is generally necessary and useful. For new embankment dams, if it is reasonably practicable to meet the Dam Safety Committee (DSC) public safety risk guidelines during construction of dams, they are to be met. If it is not reasonably practicable to meet the public safety risk guidelines, the DSC will accept a flood capacity, during those phases of construction when public safety is at risk, with an AEP range of 1 in 500 to 1 in 1,000 flood discharge on the basis of world practice, provided the risks are ALARP.[11]

For the modification of existing dams, the objective is to keep risks to public safety during construction no higher than the preexisting risks. If it is not reasonably practicable to meet that objective, the risks are to be reduced pursuant to ALARP requirements.

For risks during construction, the DSC will judge the ALARP requirement against the principles of prevention, control, and mitigation as follows:

- **Prevention.** Have reasonably practicable measures been taken to prevent failure of the partly completed dam? The measures include cofferdams, diversion tunnels or channels, and reinforced rockfill to allow substantial overflow.

- **Control.** There is limited scope to control flood failures, but there are steps that can be taken as a flood develops. For example, it is necessary to make

[11] DSC defines ALARP more broadly than Reclamation in that ALARP principles should be used so that construction risks do not exceed preexisting risks, regardless of the level of risks. Reclamation applies ALARP principles only if risk estimates (annualized failure probabilities) are less than 1E-6 and life loss estimates exceed 1,000.

the edges of partially completed lifts of reinforced rockfill safe against overflow. Having cranes, gabions, and men available for this work is a necessary control measure.

- **Mitigation.** The DSC requires a construction phase Dam Safety Emergency Plan that has an effective flood warning system, forecasting inundation levels in the event of dam failure, effective communication systems and protocols for interaction with the emergency authorities, and an effective evacuation and welfare plan to protect those at risk.

2.5.1.4 Current Approach (Reclamation)

Reclamation employs quantitative risk analysis methodology to evaluate the construction risks and identify and/or select annual or seasonal construction diversion floods that would reduce these construction risks to acceptable levels (i.e., acceptable levels of construction risks will be unique to each condition/situation and will be recommended by the designer of record and concurred by Reclamation management [decisionmakers]). On a case-by-case basis, acceptable construction risks could exceed baseline total risks and/or exceed Reclamation guidelines during portions of the construction schedule. Consideration will include the level of uncertainty and confidence associated with estimates and tradeoffs of increased costs relative to reduced construction risk or reduced exposure time. Exposure time is the duration of estimated risk during a given construction stage. Acceptable exposure time is determined on a case-by-case basis and is dependent on whether or not Reclamation decisionmakers concur with the level and duration of risk estimated for a given construction stage. The following sections lay out the process for identifying and/or selecting the construction diversion flood level in a risk framework.

2.5.2 Selection Process

This process is based on estimating risks during construction which are compared to baseline (existing) risks. It should be noted that recommended alternatives (modifications and/or new construction) need to be evaluated, both for long-term risk reduction (which typically have already been determined by the time annual or seasonal construction diversion floods are being considered) and for acceptable construction risks. Construction risks could influence the recommended modification of an existing structure or a new structure that is ultimately identified and/or selected as the preferred option. Although the focus of this chapter is the identification and/or selection of the annual or seasonal construction diversion flood level, this is only part of the overall process for estimating/evaluating construction risks. This process should also include consideration of credible PFMs from all loading conditions (static, hydrologic, and seismic). For additional details, see Chapter 27, "Construction Risks," in Reclamation's *Dam Safety Risk Analysis Best Practices Training Manual* [7]. Also, appendix B contains examples of identifying and/or selecting construction diversion floods.

For the unique case where no credible PFMs and associated risks have been determined, identification and/or selection of the annual or seasonal construction diversion flood levels are based on other factors such as economic considerations, environmental impacts, and/or engineering judgment. Historical and industry practice can be considered and might include some of the approaches previously noted in Section 2.5.1.1, "Historical Approach (Reclamation)," in this chapter.

2.5.2.1 Estimate Baseline Risks (Existing and New Dams)

For both existing and new dams, along with appurtenant structures, credible PFMs should be identified, and the risks should be estimated. This would initially take place during the feasibility or CAS and then updated/revised during the final design process. Steps include:

- Clearly identify credible PFMs for all loading conditions including static (normal), hydrologic (floods), and seismic (earthquake).

- Estimate baseline risks (AFP and ALL) for all credible PFMs for all loading conditions (including loads, responses, and consequences).

2.5.2.2 Initial Construction Schedule (Existing and New Dams)

For modifying existing dams, as well as constructing new dams and their appurtenant structures, develop an initial construction schedule during the feasibility or CAS and then update/revise it during the final design process. This construction schedule would include construction stages based on specific tasks (or activities), construction stage durations, and likely time of the year for each construction stage (all based on site-specific restrictions/considerations).

2.5.2.3 Construction Risks (Existing and New Dams)

For modifying existing dams, as well as constructing new dams and their appurtenant structures, review baseline PFMs to evaluate how construction activities may potentially increase their risks. Also, identify and evaluate new PFMs created by construction activities, and estimate associated risks. As part of the evaluation process, look at the major factors that affect these PFMs during each construction stage of each construction schedule (likely to require analysis, such as flood routings of annual or seasonal frequency floods, which include both flood and nonflood season events associated with the time of the year for a given construction stage). Considerations include:

1. RWS ranges at the time of specific construction stages (if reservoir operations do not change from baseline operations, historical operations data can be used, if available).

2. Dam/dike/cofferdam crest at time of construction stages.

3. Potential annual or seasonal flood events at the time of construction stages.

4. Maximum RWS elevations for different annual or seasonal frequency floods based on the combination of initial RWS and discharge capacity applicable to various construction stages.

Also, for all credible PFMs, estimate the risks (AFP and ALL) during each construction stage. (**Note:** Take care to manage this effort efficiently by grouping similar tasks and/or times of the year into construction stages). These construction risks are then compared to the baseline risks for each construction stage using a number of tools, such as:

1. Tables of AFP and/or ALL associated with construction stages, durations (calendar days), and times of the year (starting month to ending month).

2. Plots comparing ALL of the given construction stages to the time of the year.

3. Ratios of construction risks to baseline risks are estimated. These ratios could be estimated as the sum of the product of all AFPs and then normalizing this value for a 1-year duration.

As previously noted, based on annual or seasonal flood routings during various construction stages, the construction risk estimates are made for the initial construction schedule. These flood routings will identify the initial annual or seasonal construction diversion flood levels that can be safety accommodated and can be used to estimate the hydrologic risk for the various construction stages. If the construction risk and/or exposure time for these annual or seasonal flood levels are deemed unacceptable, additional efforts are needed to reduce the risks and/or exposure time. These efforts are further discussed in the following section.

2.5.2.4 Construction Risk Reduction (Existing and New Dams)

For modifying existing dams, as well as constructing new dams and their appurtenant structures, an evaluation to reduce construction risks may be needed if these risks are unacceptably high and/or the exposure times are unacceptably long. This evaluation would look at different construction schedules and/or modified or new structure options to reduce construction risks. If there is no way to reduce construction risks and/or shorten the exposure time, decisionmakers must either accept the modified structure or new structure option, as well as the high construction risks, or else not proceed. Some of the possible risk and exposure time reduction measures that could be (have been) considered include the following:

1. Change diversion plan to increase discharge capacity and/or flood surcharge space.

2. Change (re-sequence) construction stages to occur at different times of the year that coincide with reduced flood potential.

3. Change (re-sequence) construction activities to minimize reduction of existing discharge capacities for modifications to spillways and outlet works (such as only work on one gate or bay at a time).

4. Reduce (limit) construction stage durations by expediting work through the use of multiple work shifts.

Final identification and/or selection of the annual or seasonal construction diversion flood levels are made as described below. (**Note:** Although the designer of record will recommend the annual or seasonal construction diversion flood level, Reclamation management [decisionmakers] will make the final decision.) Flood routings are made for various construction stages, and construction risk estimates are updated. These flood routings will identify the final annual or seasonal construction diversion flood levels that can be safety accommodated (the annual or seasonal flood levels will likely change for the various construction stages). It should be stressed that, although it is likely that increased interim risk (greater than baseline risks) will result during certain construction stages, every effort should be made to minimize the level of increased risk and exposure time. Other factors that are important for identifying and/or selecting the annual or seasonal construction diversion flood levels include: diversion and care of stream costs, time impacts to construction schedule, site-specific physical limitations, and reservoir operations during construction.

2.5.2.5 Revised Construction Schedule (Existing and New Dams)

For both modifying existing dams, as well as constructing new dams and their appurtenant structures, revise the initial construction schedule(s) to balance costs and minimize construction risks and/or exposure time. These actions will reflect the evaluation made in the previous sections.

2.5.2.6 Technical References

Technical references by Reclamation associated with identification and selection of the construction diversion flood include:

- *Interim Guidelines for Addressing the Risk of Extreme Hydrologic Events* [3].

- *Design of Small Dams*, third edition [4].

- *Dam Safety Risk Analysis Best Practices Training Manual* [7].

- *Interim Dam Safety Public Protection Guidelines* [8].

- Dam Safety Office (DSO) 99-06 – *A Procedure for Estimating Loss of Life Caused by Dam Failure* [9].

- Engineering Monograph (EM) No. 9 – *Discharge Coefficients for Irregular Overfall Spillways* [19].

- EM No. 25 – *Hydraulic Design of Stilling Basins and Energy Dissipators* [20].

- Bureau of Reclamation - Engineering and Research Center (REC-ERC) 88-3 – *Overtopping Flow on Low Embankment Dams – Summary Report of Model Test* [21].

- REC-ERC-78-8 – *Low Froude Number Stilling Basin Design* [22].

- *Hydraulic and Excavation Tables*, 11th edition [24].

- *Computing Degradation and Local Scour* [25].

- *Guide for Computing Water Surface Profiles* [26].

- *Plastic Pipe Used In Embankment Dams: Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [27].

- *Outlet Works Energy Dissipators: Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair* [28]

- EM No. 14 – *Beggs Deformeter-Stress Analysis of Single-Barrel Conduits* [29].

- EM No. 14 Supplement – *Beggs Deformeter-Stress Analysis of Additional Shapes* [30].

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2.5.2.7 Examples

Appendix B provides additional details for selecting and/or identifying construction diversion floods. These examples include:

- **Example No. 1 – Dam S Modification Final Design.** Presents the process and level of detail used to identify and/or select construction diversion floods for modifications to an existing embankment dam, dike, and service spillway at a final design level elevation

- **Example No. 2 – Dam R Feasibility Level Study.** Presents the process and level of detail used to identify and/or select construction diversion floods for a new roller compacted concrete (RCC) dam, service spillway, and outlet works at a feasibility design level elevation

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Appendix A

Examples: Selecting Inflow Design Floods (IDFs)

Example No. 1. – Dam G1 Modification Final Design (Existing Dam, IDF Less than Critical PMF)

Example No. 2. – Dam F (Existing Dam, IDF Equated to Critical PMF)

Example No. 3. – Dam W (New Dam, IDF Equated to Critical PMF)

Example No. 1 – Dam G1 Modification Final Design (Existing Dam, IDF Less than Current Critical PMF)

Background

Dam G1 is located approximately 4 miles from nearest town in Wyoming and 16 miles upstream of Dam G2 (another Bureau of Reclamation [Reclamation] facility). The dam was completed in 1958 and provides a total storage capacity of 1,055,505 acre-feet at the original design maximum reservoir water surface (RWS) elevation 4669.0. The reservoir provides flood control, recreation, irrigation water, hydroelectric power, sedimentation retention, pollution abatement, wildlife conservation, and municipal and industrial water. The existing major features are summarized below:

- **The zoned embankment dam** has a structural height of 190 feet, a crest width of 35 feet, a crest length of 2,096 feet, and a crest elevation of 4675 feet.

- **Three earthfill dikes**, with a total crest length of 2,440 feet and maximum heights of 32, 45 and 75 feet, with a crest width of 25 feet at elevation 4675, which extend across low areas on the south reservoir rim.

- **The reinforced concrete service spillway** is located on the right abutment of the dam and consists of an uncontrolled ogee crest control structure, a chute and stilling basin. The spillway is designed to release up to 10,335 cubic feet per second (ft^3/s) at the original design maximum RWS elevation 4669.

- **The outlet works,** located through the south (right) reservoir rim, consists of a 21-foot-diameter concrete-lined conduit and tunnel with a penstock bifurcation near the powerplant. The outlet works has a design discharge capacity of 13,000 ft^3/s at original design maximum RWS elevation 4669.

- **A powerplant** is located near the termination of the outlet works approximately 4,000 feet south of the dam. Approximately 3,920 ft^3/s can be released through the powerplant.

It was determined that total baseline risks were unacceptably high, and there was increasing justification to reduce risks. Of note, the risks associated with the hydrologic Probable Maximum Floods (PFMs) were significant contributors to the total baseline risks. Flood routings identified that frequency flood return periods greater than 5,500 years, about 22 percent of the current critical PMF, would overtop the dam and dikes, which in turn could lead to failure of the dam and/or dikes and uncontrolled release of the reservoir. Such a failure could also lead to failure of Reclamation's Dam G2, a non-Federal dam, and significant flooding through eastern Wyoming and across Nebraska.

Inflow Design Flood Selection

Estimate of Baseline Risks

As previously noted, total baseline risks for Dam G1 were unacceptably high and there was increasing justification to reduce risks. The total estimated baseline risks included total annualized failure probability (AFP) of 4.7E-4 and total annualized life loss (ALL) of 1.2E-2, with the overtopping PFM risks making up 31 percent of total the AFP and 29 percent of the total ALL. Key factors in estimating baseline hydrologic risks are summarized below:

- All credible PFMs were clearly identified and defined, including all hydrologic PFMs.

- To evaluate PFMs, appropriate data were needed. In the case of hydrologic PFMs, a hydrologic hazard analysis was done, which developed frequency flood events (hydrographs) ranging from 5,000- to 1,000,000-year return periods [1].

- Flood routings were then done to identify operational responses in terms of maximum RWSs, maximum discharges, and freeboard or overtopping conditions. A conservative approach was employed by setting the initial RWS equal to the maximum normal RWS (top of active conservation) of 4635.0 feet. Based on historical operations, the top of active conservation would be equal to or exceeded about 6 percent of the time.

- Downstream consequences were estimated, then event trees were used to estimate baseline risks for each PFM.

Estimate of Modified Risks

To achieve sufficient risk reduction (from baseline conditions), potential modifications focused on increasing the discharge capacity, increasing the flood surcharge, or a combination. Various nonstructural[2] and structural alternatives were identified and evaluated through the following process:

---

[1] The 1,000,000-year frequency flood size (peak and volume) approximates 80 percent of the current critical PMF size.

[2] Nonstructural alternatives included reservoir restrictions, early warning systems, and no action.

- Each alternative was sized by assuming a range of frequency floods as the Inflow Design Flood (IDF). Specifically, a range of frequency floods with return periods ranging between 100,000 and 1,000,000 years was used to hydraulically size the alternatives. Alternatives considered included dam/dike breach and removal, dam/dike overtopping protection, raising the dam and dikes between 3 and 6 feet, modifying the existing service spillway to safely accommodate higher RWSs, constructing a new auxiliary spillway on the reservoir rim (either fuseplug or ogee crest structure), and nonstructural alternatives.

- Flood routings were then re-done to identify operational responses in terms of maximum RWSs, maximum discharges, and freeboard or overtopping conditions for each modification alternative. As previously noted, a conservative approach was used by setting the initial RWS equal to the top of active conservation of 4635.0 feet.

- Downstream consequences were re-estimated (where needed), then event trees were used to estimate modified risks for each PFM.

- A comparison of total baseline and total modified risks was made to assess the level of risk reduction. It should be noted that influencing risk reduction efforts for Dam G1 was the need to avoid or limit increasing risks at Reclamation's Dam G2, located 16 miles downstream (i.e., a system evaluation of the modified risks was applied). This situation led to evaluating possible nonstructural and structural modifications at Dam G2, which resulted in the decision to rehabilitate one of two existing spillways so that the original design discharge capacity could be re-established at Dam G2. Table 1 and figure 1 summarize the comparison of baseline and modified total risks. Of note is the range of IDF return periods (>97,000 years to 500,000 years) that would result in total risks being in an area of reduced justification to take action.

- Other nonrisk factors influencing which alternative was pursued included modification costs, constructability considerations, and potential changes to operations after modifications. Again, a system evaluation (Dam G1 and Dam G2) came into play. A key factor for the selection of the IDF was cost (see table 1). It was determined that significant cost increases resulted if an IDF with a return period greater than 100,000 years was selected. Additionally, significant reservoir operation changes (sizable annual reservoir drawdown would be required before spring runoff began) would result if an IDF greater than 100,000 year return period was selected. Such a reservoir operation change could result in reduced irrigation and M&I supply.

Table 1. Summary of dam G1-G2 alternatives risks, costs, and modifications

Flood return period (years)	Total risks			Alternative costs	Comment
	AFP	Life lost	ALL		
10,000	4.70E-04	26	1.22E-02	$0	Baseline (existing)
100,000	3.50E-05	26	9.10E-04	$36,000,000	G1 modifications: Small dam raise (upstream parapet wall); modify existing service spillway; construct new auxiliary spillway. G2 modifications: rehabilitate existing south spillway.
200,000	1.50E-05	26	3.90E-04	$45,000,000	G1 modifications: Moderate dam raise (embankment raise and upstream parapet wall); modify existing service spillway; construct new auxiliary spillway. G2 modifications: small dam raise and rehabilitate existing south spillway.
500,000	4.00E-06	26	1.04E-04	$75,000,000	G1 modifications: Significant dam raise (embankment raise and upstream parapet wall); modify existing service spillway; construct new auxiliary spillway. G2 modifications: Moderate dam raise and rehabilitate existing north and south spillways.

Based on the previous discussion, the 100,000-year flood event, about 39 percent of the current critical PMF, was selected as the IDF, and it will be safely accommodated once the following modifications are made at Dam G1:

- **Existing service spillway. –** Construct a headwall in the existing ogee crest control structure that would limit discharges to no more than the original design discharge of 10,375 ft^3/s for a maximum RWS of 4678 feet. This will prevent overtopping of the service spillway chute walls.

- **New auxiliary spillway. –** Construct an uncontrolled 540-foot-wide reinforced concrete and roller compacted concrete ogee crest spillway through the reservoir rim near the existing dikes. The ogee crest spillway (rather than the fuseplug spillway that was initially considered) provided the greatest risk reduction when Dam G1 and Dam G2 were considered as a system. The design discharge capacity would be 100,040 ft^3/s at a maximum RWS of 4678 feet.

- **Dam/dike raise. –** A 3-foot dam and dikes raise will be constructed that would include a upstream parapet wall that could serve as a water barrier structure. This raise addresses the robustness (freeboard) study that is further discussed in the following section.

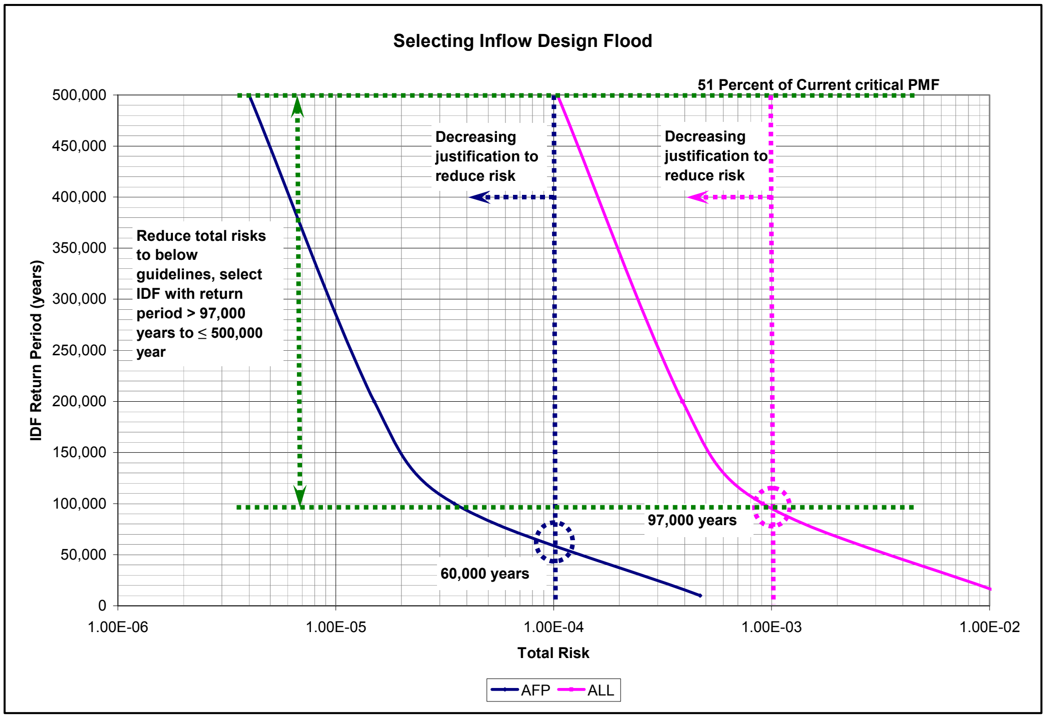

Figure 1. Dams G1-G2 IDF return period versus total risk.

It should be noted that only the modifications associated with mitigating the overtopping PFM are presented in this example. There are other modifications that will address nonovertopping PFM risks and further reduce the baseline total risks.

As previously noted, to minimize the potential of transferring and/or increasing risks at Dam G2, some modifications to Dam G2 were included that involved rehabilitation of the existing south spillway (including rehabilitation of one of the drum gates and replacement of the other drum gate with a fixed concrete weir). With both the south and north spillways almost fully functional at Dam G2 (i.e., the original total discharge capacity has been reestablished), a frequency flood with a return period of 96,000 years could be safety accommodated, which is almost three times more remote than the 31,100-year frequency flood that can be safely accommodated by the existing condition. Also, there are other modifications that will address nonovertopping PFM risks, but they are not presented in this example.

With these modifications in place, the total baseline system risks (ALL of 1.2E-2 and AFP of 4.7E-4) would be reduced to the total modified system risks (ALL of 9.1E-4 and AFP of 3.5E-5), which are below Reclamation's guidelines and are in an area of the f-N chart indicating decreasing justification to take action. The risk reduction due to system modifications is portrayed on figure 2.

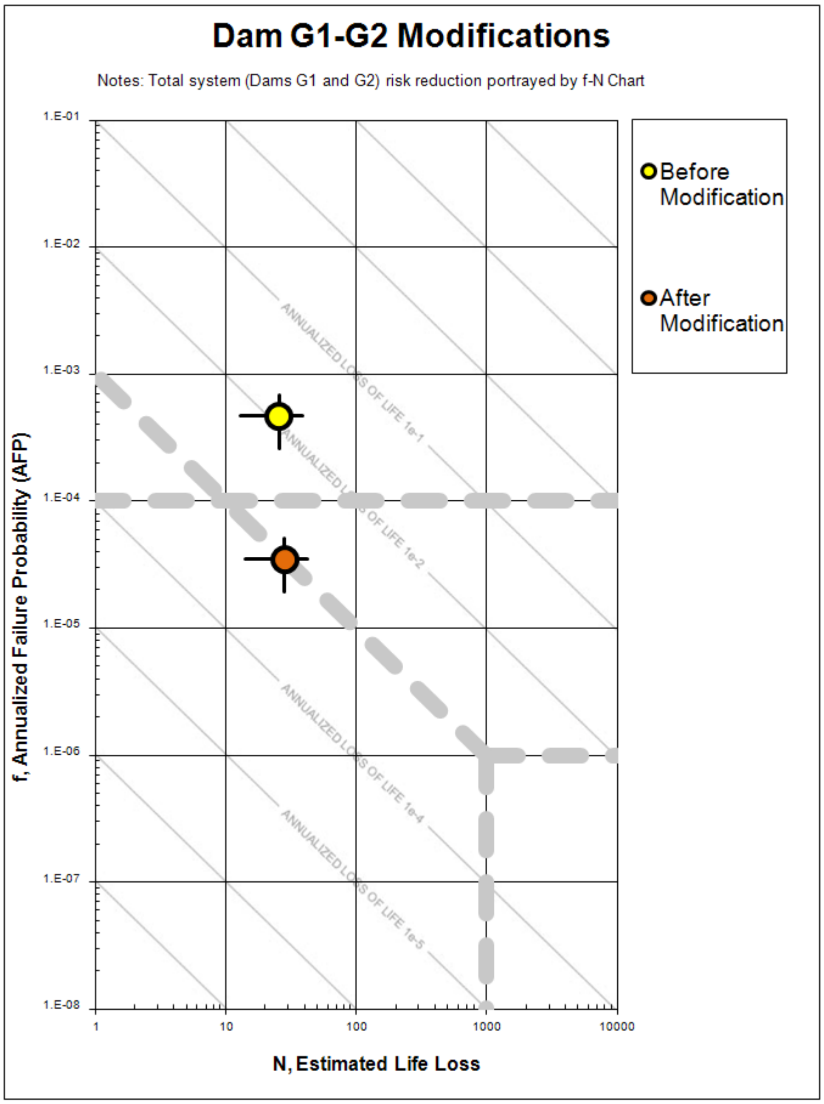

Figure 2. f-N chart portraying risk reduction for the Dam G1 and Dam G2 system modifications.

Robustness (Freeboard) Study

To address uncertainties associated with the method of estimating the IDF, reservoir and dam operations, and future events that could affect risk estimates, plausible "what-if" scenarios were evaluated and used to establish freeboard requirements. These scenarios could create elevated maximum RWSs above what would be expected for the IDF. For Dam G1, the what-if scenarios evaluated included:

- Partial (50 percent) debris plugging of both spillway and outlet works
- Future increase in hydrologic loading (500,000-year hydrograph was used as an upper-bound 100,000-year hydrograph to approximate future hydrologic loading changes)
- Potential future downstream consequence increases of about 50 percent
- Wind-generated waves

Figure 3 shows the results of the robustness study for Dam G1 modifications.

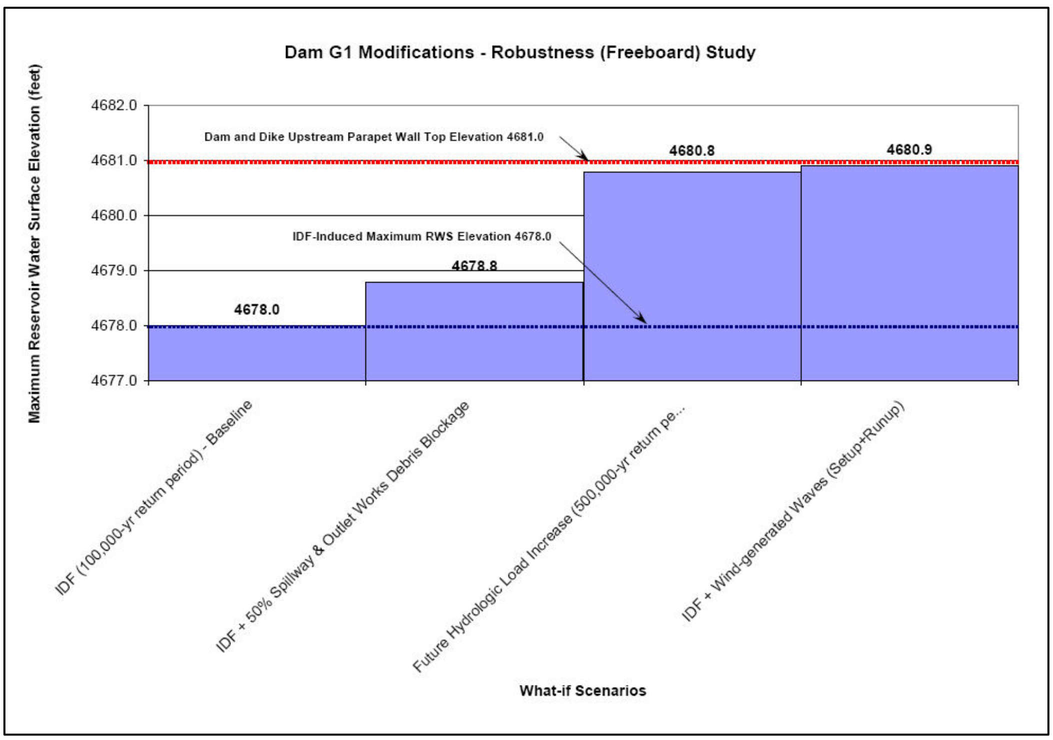

Figure 3. Robustness study results for Dam G1 modifications.

Findings

Based on flood routing results, 3 feet of freeboard would accommodate each of these scenarios (without overtopping of the dam and/or dikes). Of note, a frequency flood event with a return period of approximately 500,000 years (51 percent of the current critical PMF) would create a RWS 3 feet above the IDF-induced RWS (i.e., RWS equal to the top of the upstream parapet).

For Dam G1 modifications, the IDF has been equated to a 100,000-year frequency flood event. Additionally, 3 feet of freeboard above the maximum RWS associated with the IDF will be required to address uncertainties associated with the hydrology, modification, operations, and future events.

Example No. 2 – Dam F (Existing Dam, IDF Equated to Current Critical PMF)

Background

Dam F and reservoir are approximately 20 miles northeast and upstream of a very large metropolitan area in California. Construction of the dam was completed in 1956. The dam was designed and constructed by another Government agency and turned over to the Bureau of Reclamation (Reclamation) for operation and maintenance. The reservoir provides a total storage capacity of 1,084,780 acre-feet at the original design maximum RWS elevation 475.4. The Project provides water for irrigation, domestic, municipal, and industrial use, and power production. It also provides flood protection for the metropolitan area and helps maintain navigation along the lower reaches of the river. The existing major features include:

- **Twelve dams and dikes**, including:
  - A concrete dam having a structural height of 340 feet, a crest width of 30 feet, a crest length of 1,400 feet, and crest elevation of 480.5 feet
  - An embankment right wing dam having a structural height of 145 feet, a crest width of 30 feet, a crest length of 6,700 feet, and crest elevation of 480.5 feet.
  - An embankment left wing dam having a structural height of 145 feet, a crest width of 30 feet, a crest length of 2,100 feet, and crest elevation of 480.5 feet
  - Domney Auxiliary Dam (DAD) having a structural height of 165 feet, a crest width of 30 feet, a crest length of 4,820 feet, and crest elevation of 480.5 feet
  - Eight embankment dikes, ranging heights from 10 to 105 feet, crest widths of 30 feet, crest lengths of 740 to 2,060 feet, and crest elevations of 480.5 feet
- **A gated overflow spillway** with crest elevation of 418.0 feet is located in the center of the concrete dam and is divided into eight sections (bays) by piers. Flow through the spillway is controlled by five 42.0- by 50.0-foot service radial (tainter) gates, and three 42.0- by 53.0-foot emergency radial gates. The rated discharge capacity of the existing spillway is 468,000 ft^3/s at RWS elevation 475.4.

- **The outlet works** consists of eight 5- by 9-foot gated sluice outlet conduits located through the overflow section of the concrete dam. Each conduit has a hydraulically operated, 5- by 9-foot cast iron emergency gate and a hydraulically operated, 5-by 9-foot cast iron regulating slide gate. Four of the river outlet conduits are at elevation 280.0, and four are at elevation 210.0. The outlet works can pass 28,600 ft^3/s at reservoir elevation 427.0.

Other conveyance features include:

- Three 15.5-foot-diameter penstocks are located through the right nonoverflow section of the concrete dam to carry water to three generating units at Dam F Powerplant located approximately 500 feet downstream from the dam.

- One 7.0-foot-diameter intake conduit is located through the right nonoverflow section of the concrete dam to supply water to various local water districts.

A current critical PMF, which has a peak inflow of 906,000 ft^3/s and a 120-hour volume of 3.2 million acre-feet, was jointly developed by another Government agency and Reclamation. Also, frequency flood hydrographs were prepared by Reclamation. It was determined that a frequency flood with a return period of 7,100 years had a similar size (peak and volume) to the current critical PMF. It was also determined that the total baseline risks were unacceptably high, and there was increasing justification to reduce risks. Flood routings identified that frequency flood return periods greater than 5,000 years, about 92 percent of the current critical PMF, would overtop one or more of the dams and dikes, which in turn could lead to failure and uncontrolled release of the reservoir into a metropolitan area.

It should be noted that although the flood-induced overtopping potential failure mode (PFM) risk contributions to the total risks were significant, there are other PFMs which substantially contribute to the total risks and keep the total risks in an area of increasing justification to reduce risk. Therefore, in addition to flood risk reduction measures, both static and seismic risk reduction measures were incorporated into the overall modification so that the total risks were substantially reduced to an area of the f-N chart indicating decreasing justification to take action to reduce risks.

Inflow Design Flood Selection

Estimate of Baseline Risks

As previously noted, baseline risks for Dam F were unacceptably high, and there was increasing justification to reduce risks. The total estimated baseline risks included total AFP of 9.2E-4 and total ALL of 4.0E-1, with the overtopping PFM risks making up 43 percent of total AFP and 45 percent of total ALL. Keys to estimating baseline hydrologic risks are summarized below:

- All credible PFMs were clearly identified/defined, including all hydrologic PFMs. In the case of Dam F, there were over 50 credible PFMs associated with the 12 dams/dikes and the spillway.

- To evaluate PFMs, appropriate data were needed. In the case of hydrologic PFMs, a hydrologic hazard analysis was prepared, which developed frequency flood event peak inflows ranging from 500- to 7,100-year return periods and frequency hydrographs for 5,000- and 10,000-year return periods.[1] In addition to the frequency floods, scaled historic floods and off-season PMFs were prepared.

- Flood routings for a range of starting RWSs were then performed to identify operational responses in terms of maximum RWSs, maximum discharges, and freeboard or overtopping conditions. Two starting RWSs that bound the minimum and maximum limits of flood operations were used. These RWSs included the maximum flood control reservation elevation of 388.4 (which provided 670,000 acre-feet of flood control storage) and the maximum normal RWS (top of joint use storage) of 466.0 feet. Given that there is flood damage reduction requirements associated with reservoir operations, fairly complex flood routings were required. Modeling of another Government agency's flood control diagrams and emergency spillway release diagram was performed so that maximum operational releases of 115,000 ft^3/s would not adversely impact downstream levees for RWS $\leq$ 451.5 feet; flows were ramped up to 160,000 ft^3/s between RWS 451.5 feet and 470 feet (based on inflow projections), and then flows were ramped up to the design discharge of 468,000 ft^3/s for RWS > 470 feet.

- Downstream consequences were estimated; then, event trees were used to estimate baseline risks for each PFM.

Estimate of Modified Risks

To achieve sufficient risk reduction (from baseline conditions), potential modifications focused on increasing the discharge capacity, increasing the flood surcharge, or a combination. Various nonstructural[2] and structural alternatives were identified and evaluated through the following process:

---

[1] Since the current critical PMF size (peak and volume) is associated with a frequency flood having a return period of about 7,100 years, the 10,000-year frequency flood was not considered because it is larger than the current critical PMF.

[2] Nonstructural alternatives included reservoir restrictions, early warning systems, and no action.

- Each alternative was sized by assuming a range of frequency floods as the IDF. Specifically, a range of frequency floods with return periods ranging between 500 and 7,100 years was used to hydraulically size the alternatives for various flood operations. Alternatives considered included dam/dike breach and removal, dam/dike raise, constructing a new auxiliary spillway on the left wing dam left abutment (either fuseplug or gated auxiliary spillway), and nonstructural alternatives.

- Flood routings for the same range of starting RWSs as previously noted were then re-done to identify operational responses in terms of maximum RWSs, maximum discharges, and freeboard or overtopping conditions.

- Downstream consequences were re-estimated (where needed); then, event trees were used to estimate modified risks for each PFM.

- A comparison of total baseline and total modified risks was made to assess the level of risk reduction. A key consideration was unacceptably high residual hydrologic risk contributions to the total risk would remain if the current critical PMF was not safely accommodated (i.e., very high hydrologic risks would remain if the IDF was equated to a flood smaller than the current critical PMF). Also, as previously noted, influencing risk-reduction efforts for Dam F were multiple high-risk PFMs in addition to the flood-induced overtopping PFM. Because of this situation, risks reducing static, hydrologic, and seismic measures were part of the overall modifications. The nonflood risk reducing measures are only briefly discussed in this example.

- Other factors influencing which alternative was pursued included modification costs, constructability considerations, and potential changes to operations after modifications.

Based on the previous discussion, the current critical PMF was selected as the IDF, and it will be safely accommodated once the following modifications are made at Dam F:

- **Existing gated overflow spillway. –** Reinforce the existing radial gates and piers to resist seismic loads. Also, the existing discharge capacity is slightly larger (528,000 ft^3/s) due to a higher maximum RWS (477.5 feet). It is noted that gate control (orifice flow) will likely govern discharge.

- **New auxiliary spillway. –** Construct a new 6-bay top-seal radial gated auxiliary spillway on the left abutment of the left wing dam. The discharge capacity is 312,000 ft^3/s at RWS 477.5 feet.

- **Embankment dams and dikes.** – Although there is no raise of the crests, modify many of the dams and dikes to mitigate static- and seismic-induced PFMs. The risk reducing measures include placing downstream embankment overlays, including filter and drainage zones, along with excavating and replacing downstream materials.

Figure 4 shows the risk reduction for Dam F modifications.

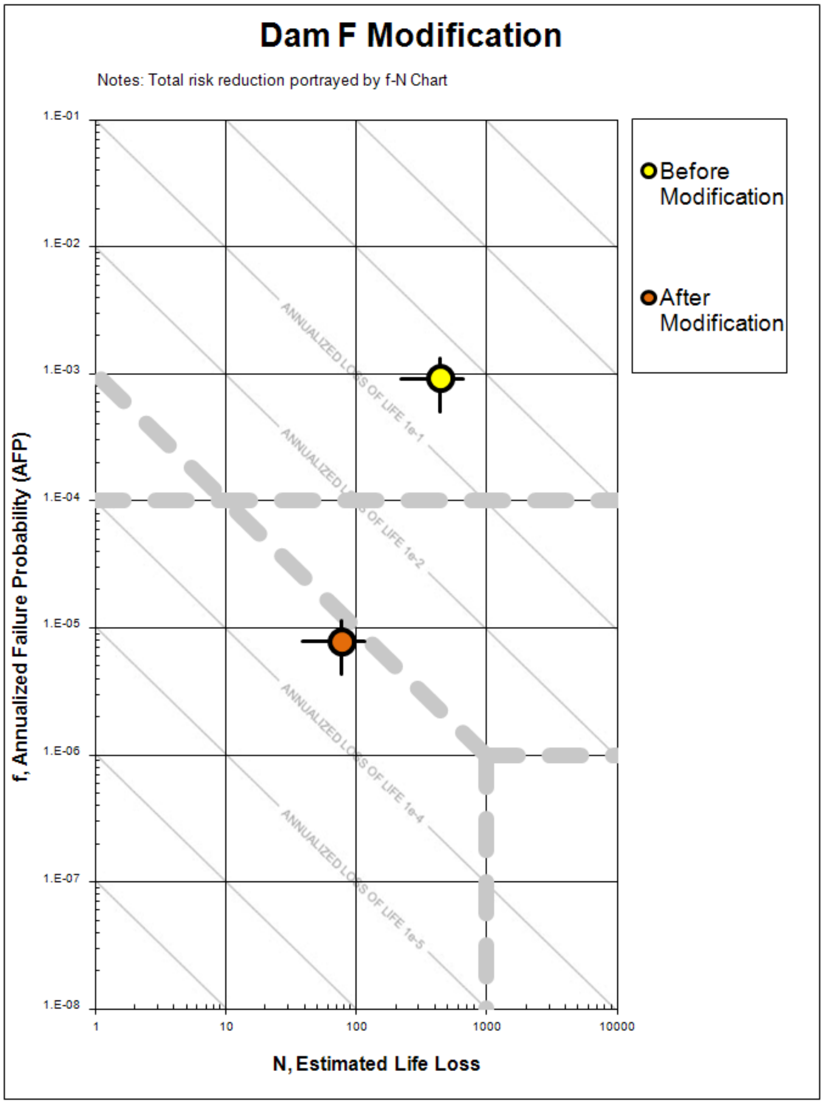

Figure 4. f-N chart portraying risk reduction for Dam F modifications.

With these modifications in place, the total baseline risks (ALL of 4.0E-1 and AFP of 9.2E-4) would be reduced to the total modified system risks (ALL of 6.1E-4 and AFP of 7.8E-6). Both the total AFP and total ALL are below Reclamation's guidelines and in an area of the f-N chart indicating decreasing justification to take action. It is noted that 18 "As-Low-As-Reasonably-Practicable" (ALARP) PFMs result in more than 70 percent of the total ALL. Careful monitoring is the anticipated action to manage the ALARP PFMs.

Robustness (Freeboard) Study

To address uncertainties associated with the method of estimating the IDF, reservoir and dam operations, and future events that could affect risk estimates, plausible "what-if" scenarios were evaluated and used to establish freeboard requirements between the IDF-induced maximum RWS and the dam/dike crests. These what-if scenarios could create elevated maximum RWSs above what would be expected for the IDF. These what-if scenarios represent plausible events that could occur, but events for which probabilities cannot be easily estimated. A robustness study is a way to account for deviations from the idealized assumptions made in flood routings in a qualitative manner. For Dam F, the what-if scenarios evaluated included:

- Scaled historic floods and off-season PMFs
- Inoperable existing spillway gates and/or new auxiliary spillway gates
- Misoperating in terms of delayed releases
- Future 10 percent increased hydrologic loading
- Wind-generated waves

Figure 5 shows the results of the robustness study for Dam F modifications.

Based on flood routing results, 3 feet of freeboard would accommodate most of these what-if scenarios (without overtopping of the dam and/or dikes). Of note (as can be seen from figure 5), 3 feet of freeboard would not accommodate all what-if scenarios - in particular, 110 percent of the current critical PMF, the current critical PMF plus two inoperable spillway gates (one existing and one new gate), and the current critical PMF in combination with misoperation by limiting releases to 160,000 ft^3/s until RWS elevation 475.5 is reached. Reclamation and another Government agency jointly decided that most of the uncertainties represented by these what-if scenarios would be addressed by 3 feet of freeboard, and three what-if scenarios that resulted in more than 3 feet above the IDF-induced maximum RWS were remote enough to not pursue raising the 11 embankment dams/dikes and the concrete dam. The other what-if scenario that exceeded 3 feet above the critical PMF-induced maximum RWS was the maximum wind-generated waves of 3.2 feet, which was associated with DAD. It was concluded that additional freeboard was unnecessary given that the waves would be intermittent and damage (if any) would be minimal.

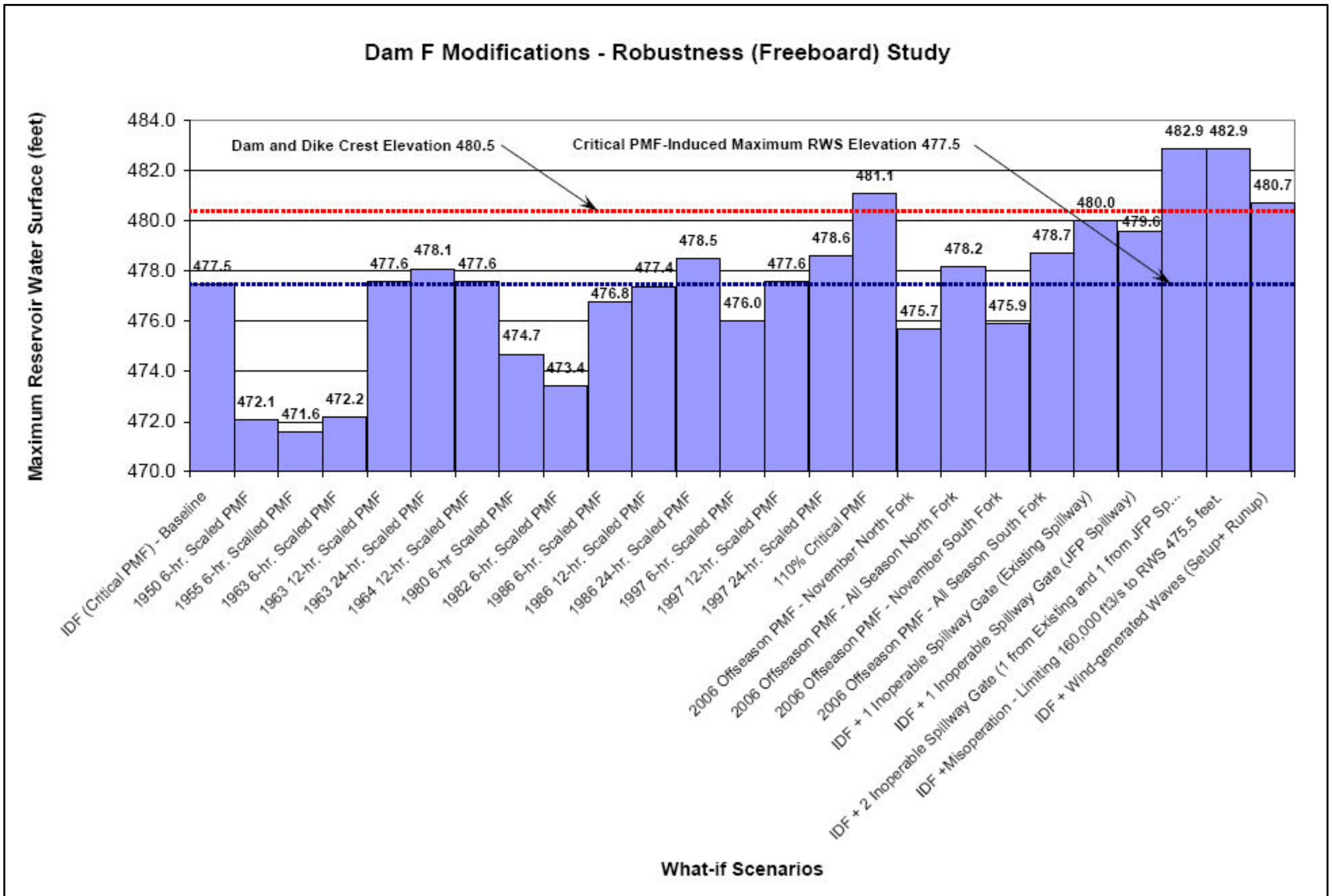

Figure 5. Robustness study results for Dam F modifications.

Findings

For Dam F modifications, the IDF has been equated to the current critical PMF. Additionally, 3 feet of freeboard above the maximum RWS associated with the IDF will be required to address uncertainties associated with the hydrology, modification, operations, and future events. Finally, an important consideration in equating the IDF to the maximum hydrologic loading (current critical PMF) and establishing 3 feet of freeboard above the IDF-induced maximum RWS was that ALARP considerations apply to 17 of the 56 PFMs, 2 of which are associated with hydrologic PFMs.

Example No. 3 – Dam W (New Dam, IDF Equated to Current Critical PMF)

Background

Dam W and reservoir will be located approximately 20 miles northeast and upstream of the closest metropolitan area in Nevada. Construction of the dam will be completed in 2015. The dam will be designed by Reclamation. It is planned that the reservoir will provide a total storage capacity of 400,000 acre-feet at the design maximum normal RWS elevation 6000.0 (top of joint use storage). The reservoir will provide flood control, recreation, supplemental irrigation water, and water for municipal and industrial use. The new major feature (initial) designs are summarized below:

- **A zoned earthfill dam** with an initial structural height of 275 feet, a crest width of 40 feet, a crest length of 1525 feet, and an initial crest elevation of 6030 feet.

- **A reinforced concrete service spillway** that will be located through the left abutment of the dam and is anticipated to consist of a radial gate controlled ogee crest structure, a chute varying from 15 feet wide at the top to 20 feet wide at the bottom, and a 20-foot-wide by 115.5-foot-long stilling basin. The spillway will be initially designed to release 3,100 ft^3/s at an assumed design maximum RWS elevation 6027.0.

- **An outlet works** that is anticipated to consist of a 12-foot-diameter reinforced concrete-lined tunnel through the right abutment. A 90-inch-diameter steel pipe will be supported within the downstream portion of the tunnel. The outlet works will have an initial design discharge capacity of 3,000 ft^3/s at an assumed design maximum RWS elevation 6027.0.

Flood hydrographs, including the current PMFs, were developed and routed through the initially sized dam and appurtenant structures. From these routings, it was determined that the current critical PMF was a rain-on-snow event with a peak inflow of 80,400 ft^3/s and a 72-hour volume of 157,300 acre-feet. Additional flood routings of frequency floods (scaled from the current critical PMF) were prepared for a risk analysis. It is noted that a frequency flood with a return period of 250,000 years has a similar size (peak and volume) to the current critical PMF.

Flood routings were based on two starting RWS elevations, including the top of active conservation, RWS elevation 5993.5, and top of joint use storage, RWS elevation 6000.0. Results indicate that frequency floods up to the size of the critical PMF can be passed without overtopping the dam (i.e., PMF-induced maximum RWS is elevation 6028 or 2 feet below the initial dam crest elevation of 6030.0 feet) with a starting RWS at the top of active conservation, elevation 5993.5. Flood routings for a starting RWS at the top of joint use storage were performed to evaluate uncertainties associated with reservoir operations and are further discussed in the following “Robustness (Freeboard) Study” section. Additionally, other hydraulic analyses were done to evaluate/verify chute wall heights, size the hydraulic-jump stilling basin, and evaluate cavitation potential and stagnation pressure (hydraulic jacking) potential.

Inflow Design Flood Selection

Estimate of Baseline Risks

A risk analysis was conducted that identified all credible PFMs, estimated associated downstream consequences, and estimated the risk contributions to the total risk for each credible PFM.

- For the IDF which was initially assumed to be equal to the current critical PMF, total mean AFP of 6.9E-7 and total mean ALL of 8.1E-5 were estimated. These risk estimates are in an area of the f-N chart indicating decreasing justification to take action to reduce risks.

- As a sensitivity study to evaluate the potential for equating the IDF to a frequency flood smaller than the critical PMF, 200,000-, 150,000-, and 100,000-year frequency floods were selected as the IDF, and the total risks were re-estimated:

    - For the IDF equaling the 100,000-year flood, the maximum RWS would be 6,024 feet (4 feet below the current critical PMF-induced maximum RWS with total mean AFP of 1.5E-4 and total mean ALL of 1.8E-2. Both total mean AFP and total mean ALL are in an area of increasing justification to take action.

    - For the IDF equaling the 150,000-year flood, the maximum RWS would be 6026 feet (2 feet below the current critical PMF-induced maximum RWS) with total mean AFP of 3.9E-5 and total mean ALL of 4.6E-3. Although the total mean AFP would be in an area of decreasing justification to take action, the total mean ALL is in an area of increasing justification to take action.

  - For the IDF equaling the 200,000-year flood, the maximum RWS would be 6027 feet (1 foot below the critical PMF-induced maximum RWS) with total mean AFP of 7.6E-6 and total mean ALL of 9.0E-4, which are in an area of the f-N chart indicating decreasing justification to take action to reduce risks.

  Table 2 and figure 6 summarize the previous discussion.

- Considering that total risk associated with the IDF equaling the current critical PMF and a frequency flood with a return period of 200,000 years are in areas of decreasing justification to take action, other factors come into play, including:

  - A nonrisk factor is the 1-foot difference between the maximum RWSs for an IDF equal to a 200,000-year frequency flood and IDF equal to the current critical PMF. The incremental cost for raising the dam 1 foot higher to accommodate the current critical PMF is very small compared to the total cost of the project (less than one-tenth of 1 percent of the total project cost, in the several $100,000 range).

  - ALARP consideration applies for the overtopping PFM (i.e., mean AFP is less than 1E-6, and potential mean life loss estimate is greater than 1,000 people). Given that Reclamation views the PMF as the largest hydrologic loading considered, equating the IDF to the critical PMF would minimize the overtopping PFM risk contributions to the total risk and be considered an ALARP action.

  - Future substantial population increases associated with the metropolitan area are anticipated, and the total risk could increase. Therefore, it would be prudent to minimize the hydrologic PFM risk contributions to the total risk by equating the IDF to the current critical PMF.

Based on the previous discussion, the IDF will be equated to the current critical PMF. The total baseline risks associated with 100,000-, 150,000-, 200,000- and 250,000-year (PMF) frequency floods equated to the IDF are portrayed on figure 7.

Table 2. Summary of dam W alternatives risks, costs, and modifications

Flood return period (years)	Total Risks			Alternative costs	Comment
	AFP	Life lost	ALL		
100,000	1.50E-04	118	1.77E-02	$22,000,000	New Dam W: Max RWS = 6024 feet
150,000	3.90E-05	118	4.60E-03	$27,500,000	New Dam W: Max RWS = 6026 feet
200,000	7.60E-06	118	8.97E-04	$28,000,000	New Dam W: Max RWS = 6027 feet
250,000	6.90E-06	118	8.14E-05	$28,200,000	New Dam W: Max RWS = 6028 feet

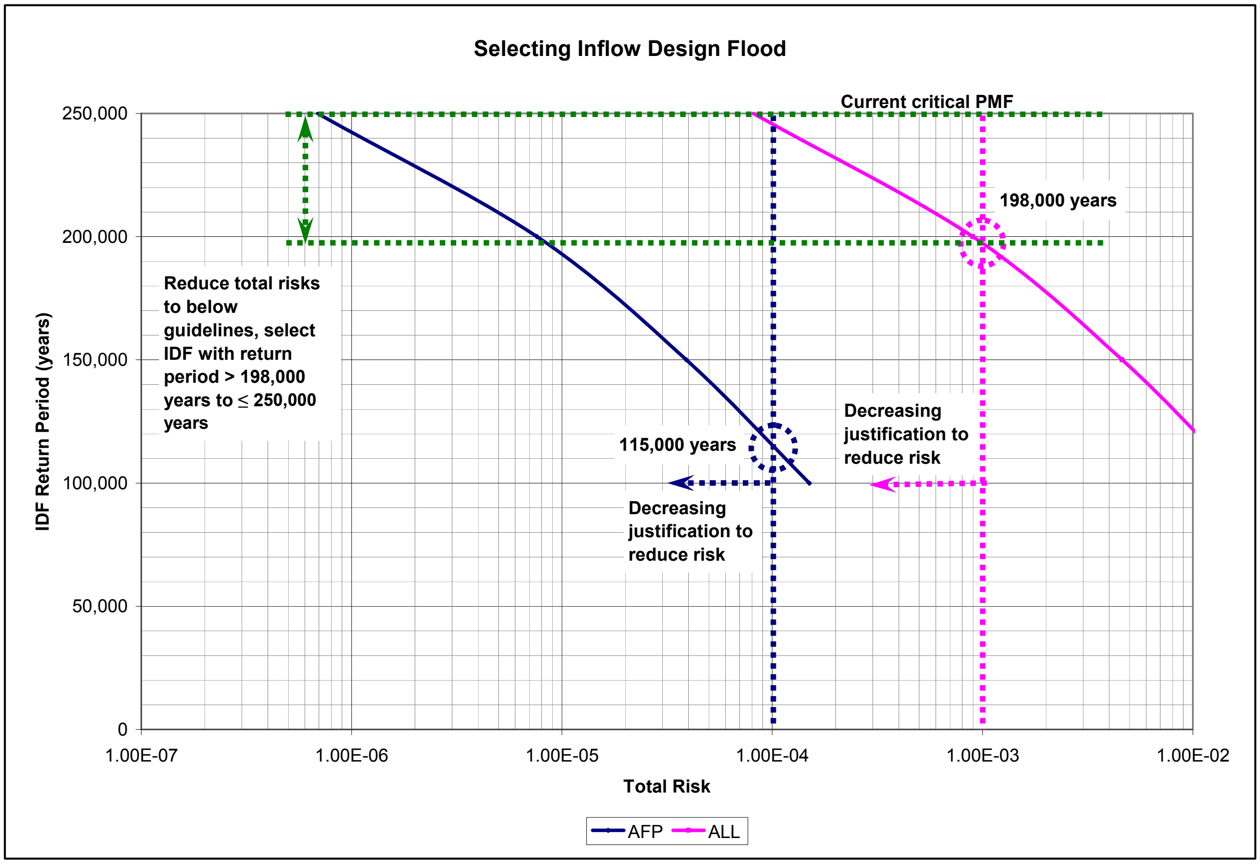

Figure 6. Dam W IDF return period versus total risks.

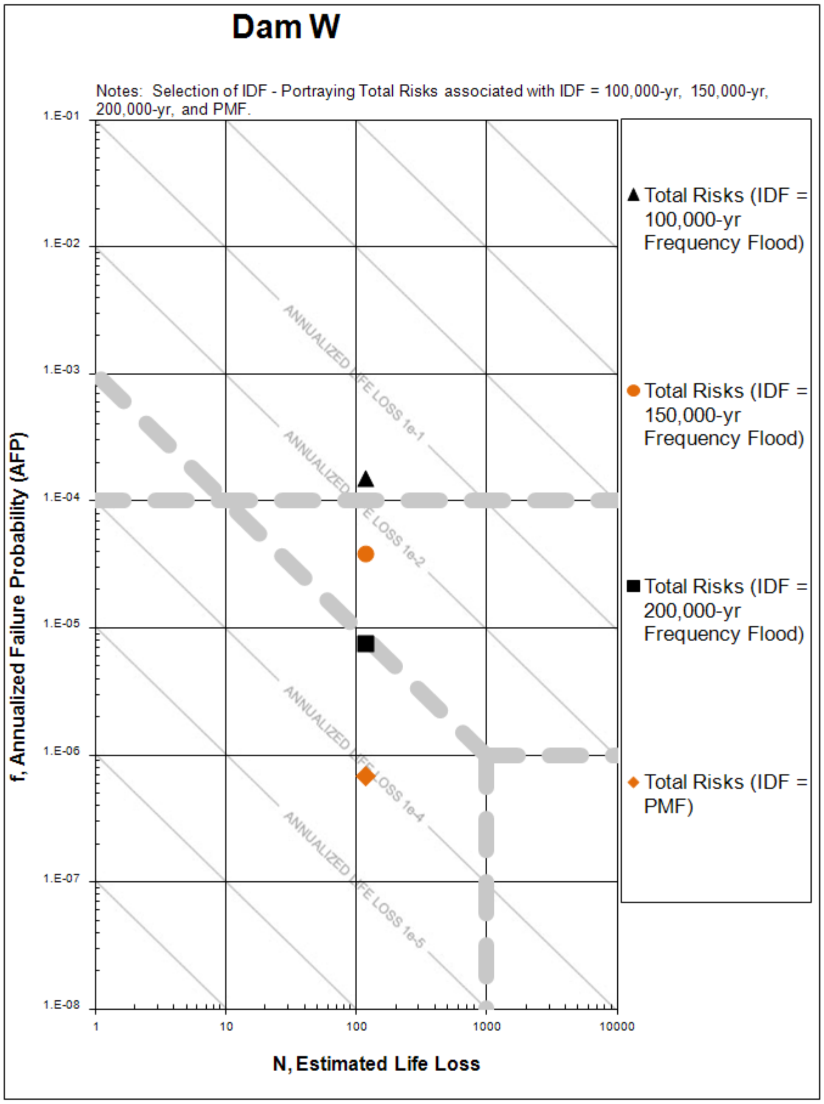

Figure 7. f-N chart portraying baseline risks assuming IDF = 100,000-, 150,000-, 200,000-year floods, and PMF for Dam W.

Robustness (Freeboard) Study

To address uncertainties associated with the method of estimating the IDF, reservoir and dam operations, and future events that could affect risk estimates, plausible “what-if” scenarios are evaluated and used to establish freeboard requirements. These what-if scenarios could create elevated maximum RWSs above what would be expected for the IDF. For Dam W, the what-if scenarios evaluated included:

- Debris blockage of 50, 75, and 100 percent of spillway
- Inoperable spillway radial gate (same as 100-percent debris blockage)
- Inoperable outlet works
- Inoperable outlet works and 50 percent debris blockage of spillway
- Future 5 and 10 percent increased hydrologic loading
- Starting RWS at top of joint use storage
- Wind-generated waves

Based on flood routing results, 4.0 feet of freeboard would accommodate all of these what-if scenarios (without overtopping of the dam). Figure 8 further illustrates the robustness study results.

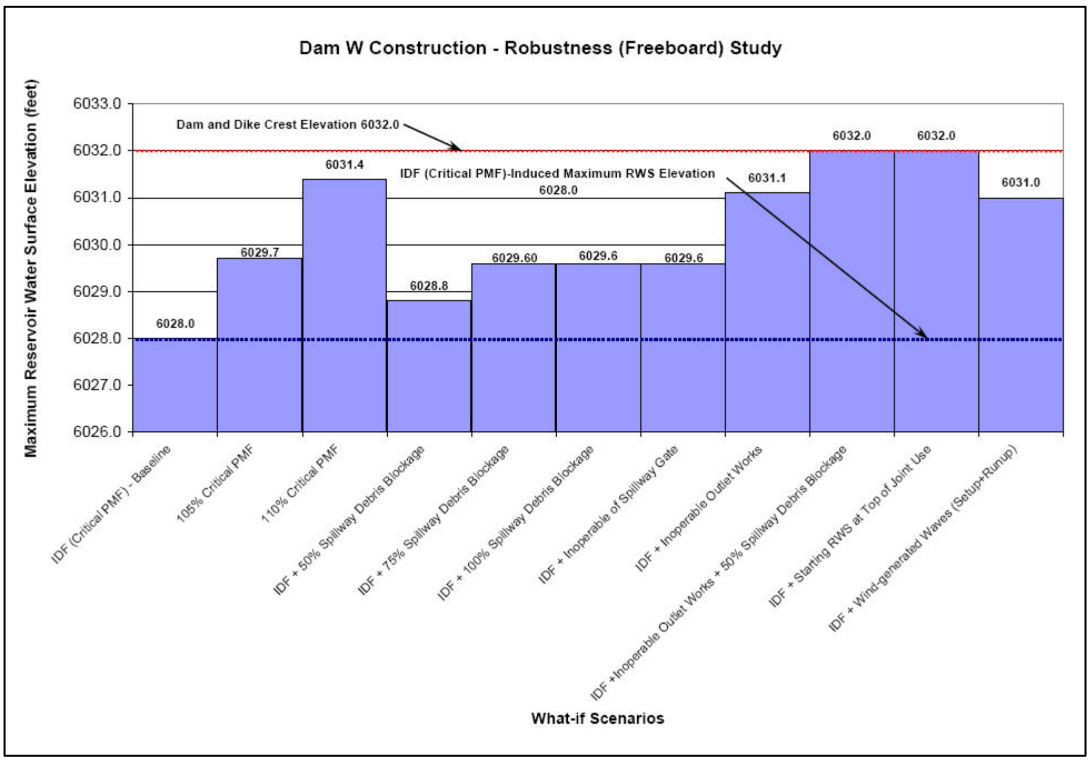

Figure 8. Robustness study results for Dam W.

Given the previous discussion, the new major feature (revised) designs include:

- **The zoned earthfill dam** will have a revised structural height from 275.0 to 277.5 feet, a crest width of 40 feet, a crest length of 1525 feet, and revised crest elevation from 6030 to 6032.5 feet (4.0 feet above the IDF-induced maximum RWS of 6028 feet).

- **The reinforced concrete service spillway** will be located through the left abutment of the dam and will consist of a radial gate controlled ogee crest structure, a chute varying from 15 feet wide at the top to 20 feet wide at the bottom, and a 20-foot-wide by 115.5-foot-long stilling basin. The spillway design discharge capacity changed from 3,100 to 3,250 ft^3/s at a revised design maximum RWS, which changed from elevation 6027.0 to elevation 6028.0.

- **The outlet works** will consist of a 12-foot-diameter reinforced concrete-lined tunnel through the right abutment. A 90-inch-diameter steel pipe will be supported within the downstream portion of the tunnel. The outlet works will have a revised design discharge capacity changed from 3,000 to 3,025 ft^3/s at a revised design maximum RWS, which changed from elevation 6027.0 to elevation 6028.0.

Findings

For Dam W, the IDF will be equated to the current critical PMF. Additionally, 4.0 feet of freeboard above the IDF-induced maximum RWS will be required to address uncertainties associated with the new dam hydrology, operations, and future events.

Appendix B

Examples: Selecting and/or Identifying Construction Diversion Floods

Example No. 1. – Dam S Modifications Final Design (Existing Dam)

Example No. 2. – Dam R Feasibility Level Study (New Dam)

Example No. 1 – Dam S Modifications Final Design (Selection and/or Identification of Construction Diversion Floods)

Background

Dam S is located approximately 11 miles northeast and upstream of a large metropolitan area in California. The dam was completed in 1970. Dam S Reservoir provides a total storage capacity of 280,200 acre-feet at the original design maximum reservoir water surface (RWS) elevation 5967.3. The reservoir provides flood control, recreation, supplemental irrigation water, water for downstream fisheries, and municipal and industrial water. The existing major features include:

- **The zoned earthfill dam,** which has a structural height of 239 feet, a crest width of 40 feet, a crest length of 1,511 feet, and a crest elevation of 5974.0 feet.

- **An earthfill dike,** approximately 1,449 feet long, which has a maximum height of 85 feet, a crest width of 40 feet at elevation 5974.0, and extends across a saddle on the south side of the reservoir.

- **The reinforced concrete service spillway,** located through the right abutment of the dam, which consists of an uncontrolled ogee crest structure, a chute varying from 15 feet wide at the top and 20 feet at the bottom, and a 20-foot-wide by 115.5-foot-long stilling basin. The spillway is designed to release 3,060 cubic feet per second (ft^3/s) of water at the original design maximum RWS elevation 5967.3.

- **The outlet works,** which consists of a 12-foot-diameter reinforced concrete-lined tunnel through the right abutment. A 90-inch-diameter steel pipe is supported within the downstream portion of the tunnel. The outlet works has a design discharge capacity of 2,740 ft^3/s at original design maximum RWS elevation 5967.3.

- **A powerplant,** which was constructed in 1987 and is adjacent to the outlet works exit channel. As a result, releases under normal conditions can be made through the same exit channel.

It was determined that the baseline total annualized life loss (ALL) was unacceptably high, and there was increasing justification to reduce risks. Over 70 percent of the total ALL was due to a single potential failure mode (PFM) associated with flood-induced overtopping of the dam and/or dike. A structural modification employing a mechanically stabilized earth (MSE) crest raise was pursued to reduce risks and consisted of:

- Constructing a dam and dike crest raise from the existing crest elevation 5974.4 to a modified crest elevation 5985.5. The dam and dike crest raise will consist of a MSE wall structure along the length of the existing main dam and dike and a traditional embankment section raise along the rock areas between the existing dam and dike.

- Demolishing and reconstructing the spillway crest structure to accommodate the MSE dam crest raise and to limit peak spillway discharges during the Inflow Design Flood (equivalent to the current critical Probable Maximum Flood [PMF]) to 3,000 ft^3/s. Peak spillway flows will be limited by constructing a new crest structure, which incorporates a concrete headwall that constricts flow (i.e., orifice control).

- Constructing two small saddle dikes in low areas along the south reservoir rim. The saddle dike structures will be homogenous embankments.

Construction Risk Evaluation

To achieve risk reduction modifications, temporary increased risks will likely result during construction. In an effort to balance costs and minimize these construction risks, an evaluation was done that focused on:

1. **Initial Construction Schedule.** Develop an initial detailed construction schedule for the final design modifications.

2. **Construction Risks.** Evaluate potential impacts to baseline risks during construction. This would include: (1) identifying baseline PFMs that could be adversely affected (i.e., increased risks), (2) identifying new PFMs created by construction activities, and (3) estimating associated risks.

3. **Construction Risk Reduction Considerations.** Identify and evaluate changes to the construction schedule that might reduce risks and/or reduce exposure time to these risks.

4. **Revised Construction Schedule.** Revise the initial construction schedule to balance costs and minimize construction risks and/or exposure time.

Working through this evaluation, the initial construction schedule (table 1) identified two potential impacts to the baseline risk during construction, which are addressed below.

Table 1. Initial construction schedule: Stages, sequencing and timeframes

Stages	Duration			
	Start	Finish	Stage time (days)[1]	Contract time (days)[2]
Initial site work	10/8/2012	3/28/2013	171	171
Reservoir saddle dikes	9/3/2013	9/30/2013	27	357
Spillway crest structure replacement	3/1/2013	10/1//2013	126	358
Dike raise (MSE wall)	3/28/2013	10/9/2013	195	366
• MSE to elevation 5974.4	3/28/2013	6/26/2013	90	261
• MSE to elevation 5985.5	6/26/2013	9/9/2013	75	336
• Complete	9/9/2013	10/9/2013	30	366
Demobilization, winter shutdown and mobilization.	10/9/2013	3/28/2014	170	536
Embankment connection	3/28/2014	5/5/2014	38	574
Dam raise (MSE wall)	3/28/2014	10/24/2014	210	746
• MSE to elevation 5974.4	3/28/2014	7/7/2014	101	637
• MSE to elevation 5985.5	7/7/2014	9/30/2014	85	722
• Complete	9/30/2014	10/24/2014	24	746
Miscellaneous	10/24/2014	10/31/2014	7	753
Complete modification	10/31/2014	10/31/2014	0	753

[1] Stage time refers to estimated time (calendar days) to complete a specific stage.
[2] Contract time refers to the estimated cumulative time (calendar days) from the start of work to the end of a given stage.

- **Increased overtopping potential.** Only the risk for the hydrologic PFM (dam and/or dike overtopping) would be adversely affected by construction activities—specifically, the dike raise (MSE crest raise) and the dam raise (MSE crest raise). During the construction of the MSE walls, the existing dam and dike crests would be temporarily lowered as much as 5 feet (from elevation 5974.4 to 5969.1) to expose the embankment core, providing a good foundation for the MSE crest raise structure. Because of this lower crest elevation, more frequent flood events could overtop and fail the dam and/or dike compared to the existing conditions.

- **Spillway modification.** During this review, it was identified that the spillway crest structure replacement construction stage could adversely affect overtopping potential at Dam B (a Reclamation facility located about 2 miles downstream from Dam S) by creating a large excavated section open to the reservoir. During a flood, significant releases could occur through the large excavated section that could overwhelm Dam B and reservoir.

Based on the two potential impacts previously discussed, it became evident that maximizing flood surcharge (flood retention) and maintaining as much of the existing and/or modified discharge capacity as possible would minimize potential impacts to the baseline risks during construction. This led to the decision to require a cofferdam to isolate the spillway construction area from the reservoir. The cofferdam would have a crest elevation approximating the existing dam crest elevation of 5974.4 feet. Having this size of cofferdam versus no cofferdam would increase the minimum threshold annual flood[16] (construction diversion flood) return period from less than 100 years to about 16,000 years. Also, increased risk transfer to Dam B would be significantly reduced. The diversion capacities through the construction are summarized in table 2 and on figure 1.

Table 2. Initial construction schedule versus diversion capacity

Stages	Duration		Minimum crest/structure elevation (feet)	Diversion capacity	
	Start	Finish		Available hydraulic structures	Threshold flood return periods (years)[1]
Initial site work	10/8/2012	3/28/2013	5974.4	Existing spillway; existing OW[3]	77,600 – 82,000
Dike raise (MSE wall)	3/28/2013	10/9/2013	Varies (see below)	Varies (see below)	Varies (see below)
• MSE to elevation 5974.4 – before spillway replacement	3/28/2013	5/1/2013	5969.1	Existing spillway; existing OW	19,600 – 21,600
• MSE to elevation 5974.4 – during spillway replacement (with spillway cofferdam)[2]	5/1/2013	6/26/2013	5969.1	Existing OW	16,000 – 17,800
• MSE to elevation 5985.5 – during spillway replacement (with spillway cofferdam)[2]	6/26/2013	9/9/2013	5974.4	Existing OW	48,500 – 48,600
• Completion – during spillway replacement (with spillway cofferdam)[2]	9/9/2013	10/1/2013	5974.4	Existing OW	48,500 – 48,600
• Completion – after spillway replacement	10/1/2013	10/9/2013	5974.4	Modified spillway; existing OW	62,200 – 65,000
Spillway crest structure replacement (with spillway cofferdam)[2]	5/1/2013	10/1/2013	5969.1	Existing OW	16,000 – 17,800
Reservoir saddle dikes (with spillway cofferdam)[2]	9/3/2013	9/30/2013	5974.4	Existing OW	48,500 – 48,600
Demobilization and winter shutdown	10/9/2013	3/28/2014	5974.4	Modified spillway; existing OW	62,200 – 65,000

[16] A conservative approach was used which assumed that annual frequency floods were similar (peak inflow and volume) to seasonal floods associated with the part of the year (March through mid-July) with the greatest flood potential.

Table 2. Initial construction schedule versus diversion capacity

Stages	Duration		Minimum crest/structure elevation (feet)	Diversion capacity	
	Start	Finish		Available hydraulic structures	Threshold flood return periods (years)[1]
Embankment connection	3/28/2014	5/5/2014	5969.1	Modified spillway; existing OW	18,400 – 19,600
Dam raise (MSE wall)	3/28/2014	10/24/2014	Varies (see below)	Varies (see below)	Varies (see below)
• MSE to elevation 5974.4	3/28/2014	7/7/2014	5969.1	Modified spillway; existing OW	18,400 – 19,600
• MSE to elevation 5985.5	7/7/2014	9/30/2014	5974.4	Modified spillway; existing OW	62,200 – 65,000
• Completion	9/30/2014	10/24/2014	5985.5	Modified spillway; existing OW	250,000 (PMF)
Miscellaneous	10/20/2014	12/10/2014	5985.5	Modified spillway; existing OW	250,000 (PMF)
Complete modification	12/10/2014	12/10/2014	5985.5	Modified spillway; existing OW	250,000 (PMF)

[1] All threshold flood results are based on an initial RWS elevation of 5946.1 (top of active conservation). Threshold flood ranges reflect the probable maximum precipitation and Reclamation hydrologic distributions.

[2] With no cofferdam to isolate the spillway crest structure construction site from the reservoir, which has a minimum excavation elevation of 5943.4 feet, the existing outlet works could only pass very frequent, very small flood events (< 100-year return period) without surcharging the reservoir (i.e., RWS will rise and flood the spillway construction area).

[3] The abbreviation “OW” represents “outlet works.”

Findings

The estimated maximum total mean risk for the initial construction schedule provides increasing justification to balance cost and minimize risks during construction. The high total mean risks are driven almost entirely by the hydrologic PFM associated with flood-induced overtopping of the main dam and/or dike during the anticipated 2-year construction period and winter shutdown. The overtopping potential is exacerbated by construction stages associated with MSE crest raise construction methods for the dam and dike along with flood-induced potential impacts to Dam B. To manage the very high construction risks, flood surcharge storage and discharge capacity will be maximized by the construction of a cofferdam to isolate the spillway construction from the reservoir and maintaining as much existing and/or modified discharge capacity as possible through the construction period. Minimum flood protection during construction will be an annual flood with a 16,000-year return period.

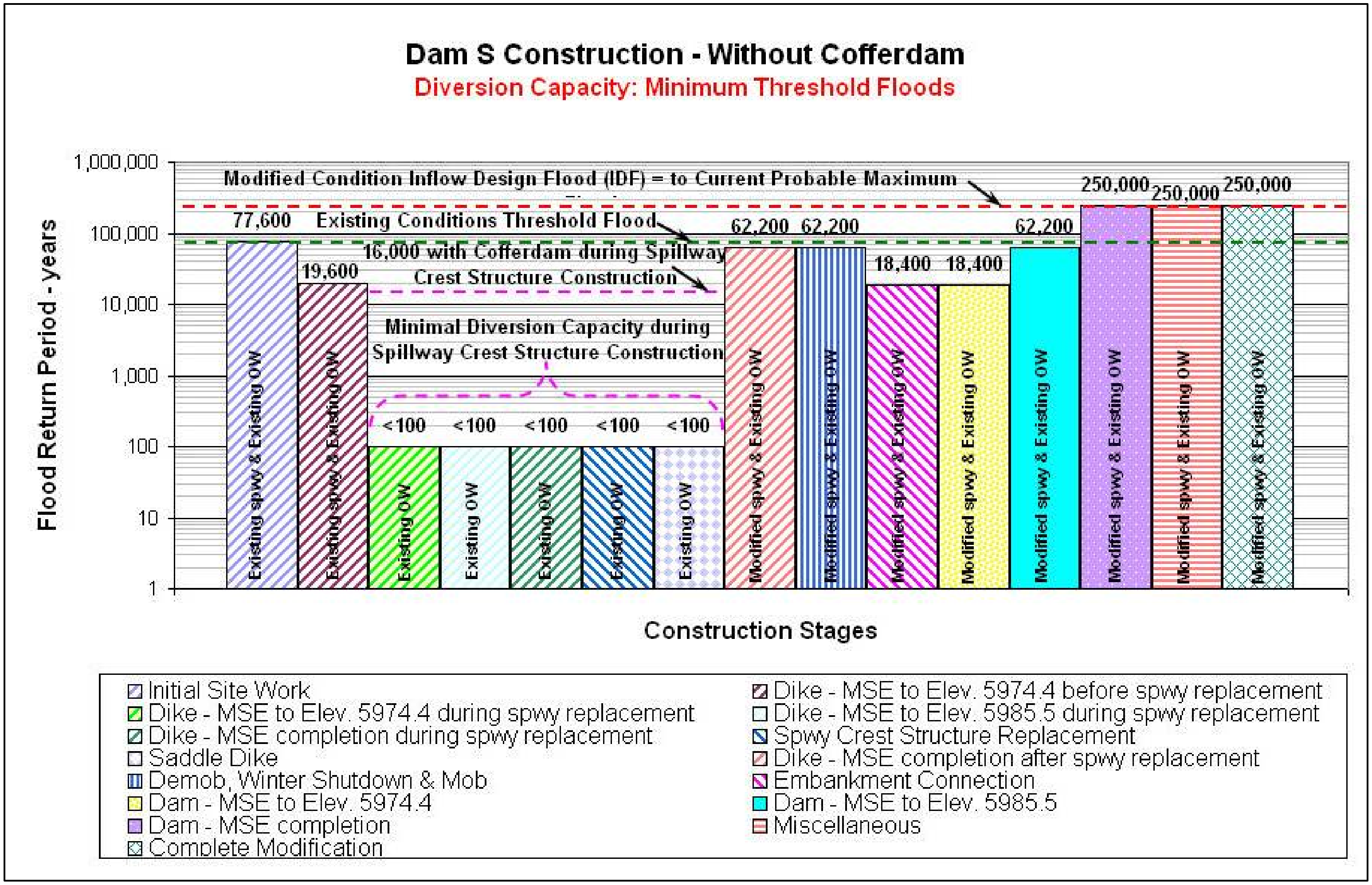

Figure 1. Diversion capacities during construction stages.

Example No. 2 – Dam R Feasibility Level Study (Selection and/or Identification of Construction Diversion Floods)

Background

First, it should be highlighted that since this is a feasibility design, it requires less detail than would be necessary for a final design. The selection and/or identification of construction diversion floods will be revisited and revised (as needed) during the final design process.

Dam R and reservoir will be located on a river approximately 30 miles northeast and upstream of the nearest metropolitan area in California. The dam will be located at the headwaters of the reservoir impounded by Dam T, an existing Reclamation facility. Reclamation will design the dam, and the new dam will be completed in 2025. The reservoir is planned to provide a total storage capacity of 1,260,000 acre-feet at design maximum normal RWS elevation 985.0 (top of active conservation). The reservoir will provide recreation, supplemental irrigation water, and water for municipal and industrial use. The new major feature (initial) designs are summarized below:

- **A roller compacted concrete (RCC) dam** is anticipated to have a structural height of 665 feet, a crest width of 30 feet, a crest length of 2,450 feet, and a crest elevation of 1005 feet.

- **A reinforced concrete service spillway** is anticipated to be integral to the dam and located near the midpoint of the dam. The spillway is anticipated to consist of a 500-foot-wide uncontrolled ogee crest, a 500-foot-wide stepped chute, and a 500-foot-wide flipbucket terminal structure, which is about 250 feet above the base of the dam. Spillway releases up to an operational design annual flood event with a return period of 500 years will be discharged into Dam T Reservoir, which will serve as a plunge pool to dissipate the kinetic energy. The spillway will be designed to release 134,400 ft^3/s of water at operational design maximum RWS elevation 1008.5, which is the top of the upstream parapet wall.

- **The dam crest** is anticipated to serve as an auxiliary spillway that will accommodate annual flood events with a return period greater than 500 years up to and including the current critical PMF, which is the Inflow Design Flood. Designs of dam crest and reinforced concrete abutment overtopping protection will permit safe releases of 521,800 ft^3/s into Dam T Reservoir at design maximum RWS elevation 1020.0.

- **The outlet works** is anticipated to be located through the left abutment and beneath the RCC dam foundation contact, will consist of an 800-foot-long trapezoidal excavated rock approach channel with a bottom elevation and width of 520 feet and 92 feet, respectively. The intake structure will be a reinforced concrete selective-level intake affixed to the upstream face of the RCC dam, which houses four emergency fixed-wheel gates over the entrance to four 15-foot-diameter steel pipes. The four steel pipes are embedded in mass concrete and extend beneath the dam footprint and downstream to a valve house. The valve house contains a 12-foot-diameter butterfly valve and an 8-foot-diameter regulating cone valve for each of the four 15-foot-diameter steel pipes. The outlet works will have a design total discharge capacity of 50,000 ft^3/s at design maximum RWS elevation 1008.5, which is the top of the upstream parapet wall.

Quantitative risk analysis methodology was used to develop the designs, which resulted in targeting design loadings so that the baseline total risks are in an area of the f-N chart where there is decreasing justification to take action to reduce risks. These risk estimates are acceptable to Reclamation management (decisionmakers).

Because there are flood control requirements associated with Dam T, the reservoir will be raised and lowered to meet these requirements and will typically fluctuate between the top of active conservation, elevation 470, and top of joint use storage, elevation 580, depending on runoff predictions. As will be discussed in the following section, this significant fluctuation of Dam T Reservoir is one of the challenges that must be considered when selecting and/or identifying construction diversion floods.

Construction Risk Evaluation

Temporary increased risks (over the baseline total risk estimates) will likely result during construction. In an effort to balance costs and minimize these construction risks, an evaluation was done that focused on:

1. **Initial Construction Schedule.** Develop an initial detailed construction schedule for the final design modifications.

2. **Construction Risks.** Evaluate potential impacts to baseline risks during construction. This would include: (1) identifying baseline PFMs that could be adversely affected (i.e., increased risks), (2) identifying new PFMs created by construction activities, and (3) estimating associated risks.

3. **Construction Risk Reduction Considerations.** Identify and evaluate changes to the construction schedule that might reduce risks and/or reduce exposure time to these risks.

4. **Revised Construction Schedule.** Revise the initial construction schedule to balance costs and minimize construction risks and/or exposure time.

Working through this evaluation, the initial construction schedule was based on a number of key considerations, including:

- Normal reservoir operations must be maintained for Dam T and reservoir. This would result in about 110 feet of annual reservoir fluctuations between top of active conservation, RWS elevation 470, and top of joint use storage, RWS elevation 580. Given that the lowest point of the foundation contact for Dam R is elevation 345, initial construction activities will involve constructing part of the diversion system, specifically placing upstream and downstream cofferdams in the wet (between 125 and 235 feet of water).

- The total construction duration is estimated to be 54 months (4.5 years) without any winter shutdown. Upfront work will include constructing the diversion system (see next bullet for details), which is expected to require 4 months. **Note:** the diversion system will be needed until the RCC dam can be raised to at least maximum normal RWS (580 feet) of Dam T. The duration is estimated to be 26 months and will include unwatering and dewatering of the construction site, foundation preparation, and placing RCC to elevation 580.

- The diversion system will include rockfill cofferdams located across the river at the upstream and downstream limits of the construction site. It was determined that the most economical way of conveying flow during construction would be to excavate the outlet works channel through the left abutment of the dam and use this channel to divert flows. The channel would be about 92 feet wide, with 1:10 side slopes, and it would have an invert elevation of 520 feet. To minimize construction impacts of placing the RCC dam, the area of the channel that is within and below the footprint of the dam will include four 15-foot-diameter steel pipes that will have an invert elevation of 530 feet and will be encased in mass concrete that extends up to the foundation contact of the dam, approximately elevation 550. The encased steel pipes will ultimately be part of the outlet works.

- Since the critical construction period of 26 months spans two flood seasons, the frequency hydrographs for sizing the diversion system are based on flood season conditions (rain-on-snow conditions), which are considered conservative (i.e., assumed to be annual rather than seasonal floods). During final design, evaluation of the diversion systems will be refined by consideration of both flood and nonflood season events.

- Flood routings of annual frequency floods identified maximum headwater behind the upstream cofferdam, maximum tailwater below the downstream cofferdam (i.e., RWS of Dam T Reservoir), and maximum diversion releases, which are summarized in table 3.

Table 3. Frequency flood routing results

Flood return period (year)	Headwater surface upstream cofferdam (feet)	Tailwater surface downstream cofferdam (feet)	Maximum discharge (ft^3/s)
2	541.3	470.0	5,550
10	552.3	480.0	15,910
25	556.4	500.0	21,980
50	581.5	530.0	27,140
100	623.6	550.0	37,640
200	667.9	580.6	46,720
500	684.4	580.6	48,780

- Based on previous flood routings for unregulated flow conditions (i.e., existing conditions in which there is no Dam R diversion system), it was determined that annual frequency floods in the range of 500- to 1,000-year return periods would raise the RWS to the top of the upstream parapet wall at Dam T (threshold flood conditions). This information was used to develop baseline risk estimates for Dam T, which were used to evaluate whether or not additional risk would be transferred to Dam T during the construction of Dam R.

One potential impact to the baseline risks for Dam T was identified and further evaluated. Specifically, the rockfill cofferdams could be overtopped and fail, which could release a breach hydrograph that might exceed the available reservoir surcharge and discharge capacity for Dam T. To evaluate this potential impact, breach hydrographs associated with annual frequency flood return periods were routed through Dam T. The results of this effort indicated that annual floods with return periods greater than 100 years could overwhelm Dam T and cause overtopping conditions. Since the existing Dam T can accommodate 500- to 1,000-year annual flood events without overtopping, additional risk could be transferred to Dam T if the diversion system was designed for an annual flood event larger than a 100-year return period.

To further evaluate and identify the annual construction diversion flood level, industry practice was reviewed:

- From a historical perspective, Reclamation has sized the construction diversion flood by equating the annual return period to five times the total construction period (4.5 years) or the critical construction period (26 months). In the case of Dam R, the critical construction period is used, which equals about a 10-year return period (2.17 x 5 = 10.9). This estimate could serve as a minimum return period for the construction diversion flood.

- Using the Institute of Civil Engineering approach (a special form of the Binomial distribution, $P_a=1-(1-P_n)^{1/n}$), a diversion system must safely pass annual floods that have only a 1-percent chance of being exceeded during the critical construction period (26 months in the case of Dam R) or about a 216-year return period ($1/(1-0.01)^{1/(2.17)} = 216$). This estimate exceeds the upper limit of a 100-year annual event (i.e., to avoid increased risk at Dam T due to failure of cofferdams designed for greater than a 100-year return period), so this approach would not be applicable for selecting and/or identifying construction diversion floods for Dam R.

- Using some of the concepts of the U.S. Army Corps of Engineers' approach (also using a special form of the Binomial distribution sometimes referred to the Bernoulli distribution), a diversion system is economically optimized by comparing the cost of the diversion system to the flood cost if the diversion system fails. Of note, a minimum return period of 10 years is established from Reclamation's historical practice of five times the critical construction period:

    - The total probable flooding cost for a given annual flood event occurring during a construction period is estimated by $C_F=P_n(DC_1+C_2)$, where C_F is the probable total flooding cost, P_n is probability of a given flood event occurring during the critical construction period, D is number of days the construction site is flooded before cleanup can begin (estimated to be 15 days), C_1 is investment losses per day while area is inaccessible (estimated to be $40,000 per day), and C_2 is fixed cost of cleanup (estimated to be $1,622,500). Table 4 summarizes the probable total flood costs, and table 5 summarizes the fixed cost due to flooding.

    - Probability of a given annual flood event occurring during a construction period is estimated by a special form of the Binomial distribution, Pn=1-(1-Pa)n, and is summarized in table 6.

Table 4. Diversion system cost estimates

Flood return period (years)	Costs			
	Diversion channel	Cofferdams	Total cost (C_C)	Total difference above 2-year flood (C_D)
2	$6,000,000	$5,000,000	$11,000,000	$0
10	$6,000,000	$5,100,000	$11,100,000	$100,000
25	$6,000,000	$5,300,000	$11,300,000	$300,000
50	$6,000,000	$5,700,000	$11,700,000	$700,000
100	$6,000,000	$6,500,000	$12,500,000	$1,500,000
200	$6,000,000	$8,100,000	$14,100,000	$3,100,000
500	$6,000,000	$11,300,000	$17,300,000	$6,300,000

Table 5. Fixed cost due to flooding

Cost impacts	$/day	Days	Total
Downtime	$25,000	15	$375,000
Pumping and Cleanup	$10,000	15	$150,000
Damage Cost	---	---	$1,000,000
Investment Cost	$5,000	15	$75,000
Liquidated Damages	$1,500	15	$22,500
Total:			**$1,622,500**

Table 6. Diversion costs versus flood frequency

Critical construction period	Frequency flood return periods (years)						
	2	10	25	50	100	200	500
P_n for 26 months	0.777	0.204	0.085	0.043	0.022	0.011	0.004
C_F for 26 months	$1.727 M	$0.454 M	$0.188 M	$0.095 M	$0.048 M	$0.024 M	$0.010 M
C_D diversion cost > 2-year flood	$0--	-$0.1 M-	$0.3 M	$0.7 M	$1.5 M	$3.1 M	$6.3 M

Note: M = million dollars.

- o The total diversion cost to accommodate a given annual construction diversion flood is estimated by using Reclamation's historical practice (five times the critical construction season) to establish a minimum annual diversion flood return period and estimating the difference of diversion system costs for annual floods greater than 2 years from the diversion system cost for the 2-year annual event. Table 6 summarizes the diversion system cost estimates. Also, table 4 summarizes the difference in diversion cost for flood return periods greater than a 2-year annual event from the diversion system cost for a 2-year annual event. The total diversion and probable flood costs versus flood return periods are portrayed in the figure 1.

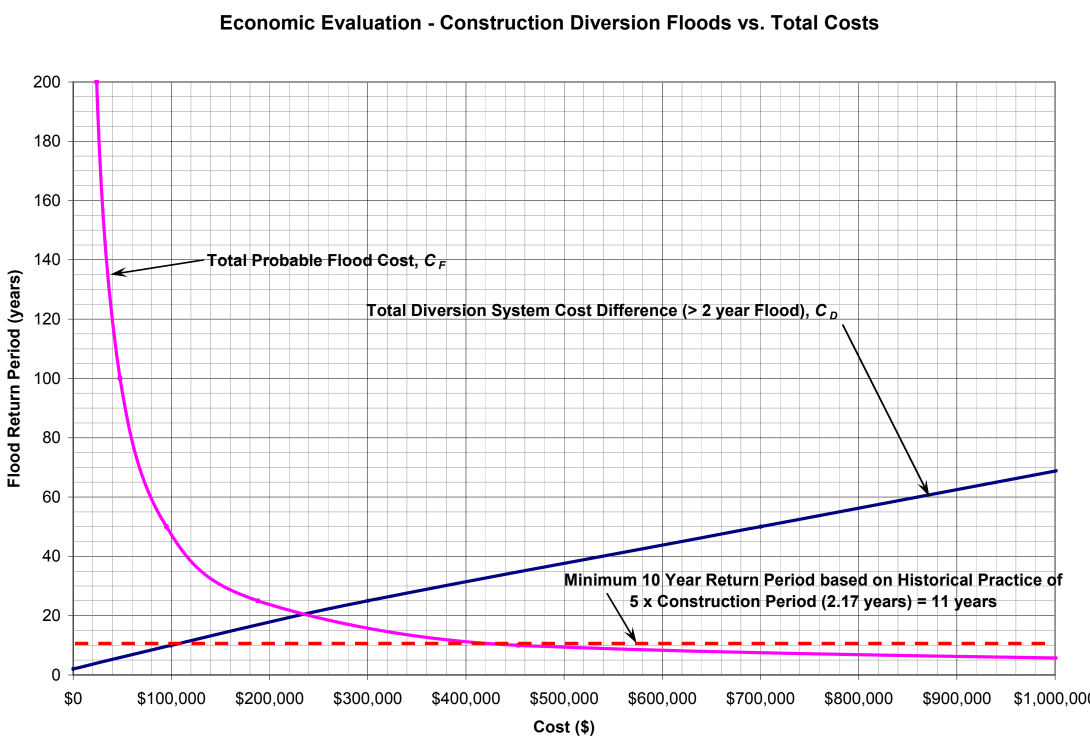

Figure 1. Total diversion costs and total probable flood costs versus construction diversion floods, Dam R.

- o The incremental changes of the total diversion cost and the total probable flood cost are estimated. Table 7 summarizes the estimated incremental change of each cost curve.

Table 7. Estimated Incremental Changes of Cost Curves

Flood return period (years)	Total Diversion System Cost > 2 year flood (C_D)	Total Probable Flood Cost (C_F)	Increment Change of Total Diversion System Cost (ΔC_D)	Incremental Change of Total Probable Flood Cost (ΔC_F)
2	$0	$1,727,494	---	---
10	$100,000	$453,611	$100,000	$1,273,883
25	$300,000	$188,132	$200,000	$265,479
50	$700,000	$95,186	$400,000	$92,946
100	$1,500,000	$47,873	$800,000	$47,313
200	$3,100,000	$24,007	$1,600,000	$23,867
500	$6,300,000	$9,620	$3,200,000	$14,387

- Figure 2 shows a plot of costs versus construction diversion flood return period using the estimated incremental change of total diversion costs (ΔC_D) and the incremental change of total probable flood costs (ΔC_F).

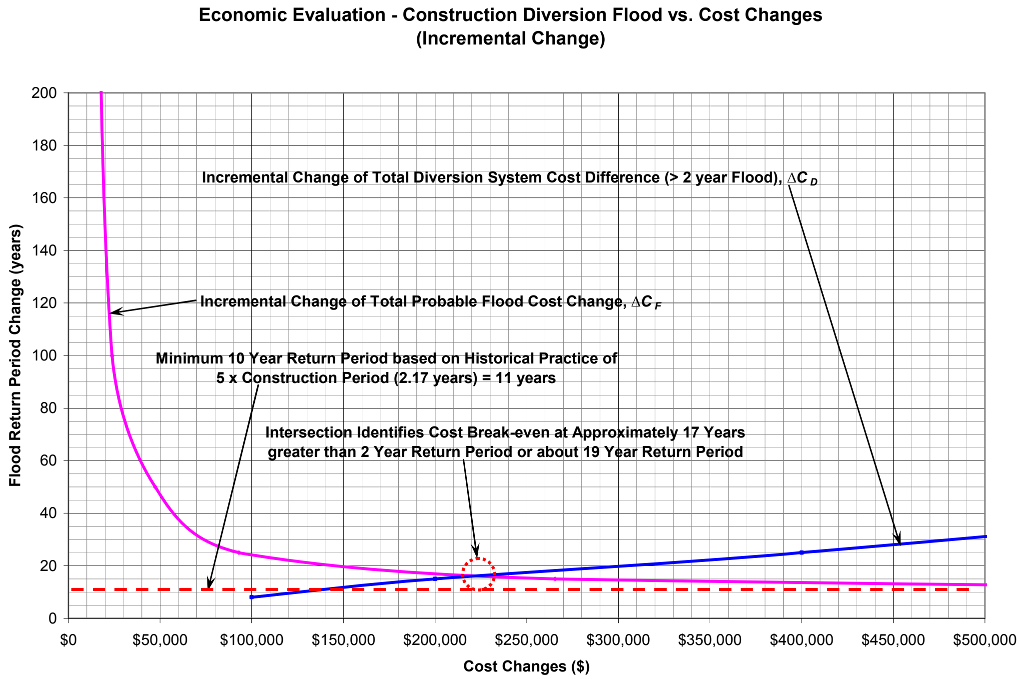

Figure 2. Identification of construction diversion flood base on costs, Dam R.

Findings

The following is associated with a feasibility design; further evaluation will be warranted during final design:

- Rockfill cofferdams:

    - To minimize the potential of transferring additional risk to an existing downstream Reclamation dam (Dam T) during the construction of Dam R, the construction diversion system is sized for no larger than a 100-year annual flood. Based on a review of some of the industry practices, a construction diversion annual flood in the range of a 10- to 25-year return period can be considered at this time.

    - Ranges of crest elevations for the upstream and downstream cofferdams are 556 to 560 feet and 483 to 503 feet, respectively (includes 3 feet of freeboard).

- Armored rockfill or nonerodible cofferdams:

    - Overtopping protection for rockfill cofferdams should be pursued to minimize the potential for rapid failure of the cofferdams, resulting in a breach hydrograph that could endanger people (recreationalists) on or near Dam T Reservoir.

    - In lieu of rockfill cofferdams, consider RCC cofferdams that can be integrated into Dam R. Based on preliminary economic analysis, this appears to result in an overall savings to the total project cost.

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: 2. Hydrologic Considerations

Brief Description of Information: Technical processes used by the Bureau of Reclamation to select construction diversion floods and the Inflow Design Floods (IDFs) for new and modified dams and their appurtenant structures, such as spillways and outlet works. A list of key technical references used for each major task involved with selecting construction diversion floods and IDFs.

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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