JAPAN LESSONS-LEARNED PROJECT DIRECTORATE

JLD-ISG-2013-01

Guidance For Assessment of Flooding Hazards Due to Dam Failure

Interim Staff Guidance

Revision 0

July 29, 2013 ML13151A153

U.S.NRC

UNITED STATES NUCLEAR REGULATORY COMMISSION

Protecting People and the Environment

<u>JAPAN LESSONS-LEARNED PROJECT DIRECTORATE</u>

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ADAMS Accession No.: ML13151A153 *Via E-mail

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INTERIM STAFF GUIDANCE

JAPAN LESSONS-LEARNED PROJECT DIRECTORATE GUIDANCE FOR ASSESSMENT OF FLOODING HAZARDS DUE TO DAM FAILURE

JLD-ISG-2013-01

PURPOSE

This interim staff guidance (ISG) is being issued to provide guidance acceptable to the staff of the U.S. Nuclear Regulatory Commission (NRC) for re-evaluating flooding hazards due to dam failure as described in NRC's March 12, 2012, request for information (Ref. 1) issued pursuant to Title 10 of the Code of Federal Regulations (10 CFR), Section 50.54, "Conditions of licenses," regarding Recommendation 2.1 of the enclosure to SECY-11-0093, "Recommendations for Enhancing Reactor Safety in the 21st Century, the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident" (Ref. 2). Among other actions, the letter dated March 12, 2012, requests that respondents reevaluate flood hazards at each site and compare the reevaluated hazard to the current design basis at the site for each flood mechanism. Addressees are requested to perform an integrated assessment if the current design-basis flood hazard does not bound the reevaluated flood hazard for all mechanisms. This ISG will assist operating power reactor respondents and holders of construction permits under 10 CFR Part 50 in performing flooding hazard assessments due to dam failure. The guidance provided in this ISG describes methods that can be used as part of performing the flooding hazard reanalysis requested in Enclosure 2 of the letter dated March 12, 2012.

BACKGROUND

Following the events at the Fukushima Dai-ichi nuclear power plant, the NRC established a senior-level agency task force referred to as the Near-Term Task Force (NTTF). The NTTF conducted a systematic and methodical review of the NRC regulations and processes and determined if the agency should make additional improvements to these programs in light of the events at Fukushima Dai-ichi. As a result of this review, the NTTF developed a comprehensive set of recommendations, documented in the enclosure to SECY-11-0093 (Ref. 2). These recommendations were enhanced by the NRC staff following interactions with stakeholders. Documentation of the NRC staff's efforts is contained in SECY-11-0124, "Recommended Actions to be Taken Without Delay From the Near-Term Task Force Report," dated September 9, 2011 (Ref. 3), and SECY-11-0137, "Prioritization of Recommended Actions to be Taken in Response to Fukushima Lessons Learned," dated October 3, 2011 (Ref. 4).

As directed by the staff requirements memorandum in the enclosure to SECY-11-0093 (Ref. 5), the NRC staff reviewed the NTTF recommendations within the context of the NRC's existing regulatory framework and considered the various regulatory vehicles available to the NRC to implement the recommendations. SECY-11-0124 and SECY-11-0137 established the staff's prioritization of the recommendations based upon the potential safety enhancements. As part of the staff requirements memorandum for SECY-11-0124, dated October 18, 2011 (Ref. 6), the Commission approved the staff's proposed actions, including the development of three information requests under 10 CFR 50.54(f). The information collected will be used to support the NRC staff's evaluation of whether further regulatory action should be pursued in the areas of seismic and flooding design and emergency preparedness. In addition to Commission direction, the Consolidated Appropriations Act, Public Law 112-074, which contains the Energy and Water Development Appropriations Act, 2012, was signed into law on December 23, 2011. Section 402 of the law requires a reevaluation of licensees' design basis for external hazards.

In response to the aforementioned Commission and Congressional direction, the NRC issued a request for information to all power reactor licensees and holders of construction permits under 10 CFR Part 50 on March 12, 2012 (50.54(f) letter)(Ref. 1). The March 12, 2012, 50.54(f) letter includes a request that respondents reevaluate flooding hazards at nuclear power plant sites using updated flooding hazard information and present-day regulatory guidance and methodologies. The letter also requests the comparison of the reevaluated hazard to the current design basis at the site for each potential flood mechanism. If the reevaluated flood hazard at a site is not bounded by the current design basis, respondents are requested to perform an integrated assessment. The integrated assessment will evaluate the total plant response to the flood hazard, considering multiple and diverse capabilities such as physical barriers, temporary protective measures, and operational procedures. The NRC staff will review the responses to this request for information and determine whether regulatory actions are necessary to provide additional protection against flooding. This ISG is specific to the assessment of flood hazards due to dam failure.

On April 25, 2013, the NRC staff issued a draft version of this ISG and published a notice of its availability for public comment in the Federal Register (78 FR 24439). The 30-day comment period ran April 25, 2013, through May 28, 2013, during that time the staff received 105 public comments in six submittals. Comments received were related to the following topical areas: (1) general comments; (2) comments specific to hydrologic dam failure; (3) comments specific to seismic dam failure; and (4) comments specific to sunny-day dam failure. In public meetings on May 2, 2013, and May 22, 2013, the NRC staff interacted with external stakeholders to discuss, understand, and resolve public comments. Modifications were made to text of the ISG in response to the public comments and the outcomes of the public meetings. Full detail of the comments, staff responses, and the staff's bases for changes to the ISG are contained in "NRC Response to Public Comments" to JLD-ISG-2013-01 (Docket ID NRC-2013-0073) (Ref. 7).

RATIONALE

On March 12, 2012, the NRC issued a request for information to all power reactor licensees and holders of construction permits under 10 CFR Part 50. The request was issued in accordance with the provisions of Sections 161c, 103b, and 182a of the Atomic Energy Act of 1954, as amended (the Act), and NRC regulation in Title 10 of the *Code of Federal Regulations*, Part 50, Paragraph 50.54(f). Pursuant to these provisions of the Act and this regulation, respondents were required to provide information to enable the staff to determine whether a nuclear plant license should be modified, suspended, or revoked.

The information request directed respondents to submit a reevaluated flooding hazard for their sites using updated information and present-day regulatory guidance and methodologies. This ISG describes approaches for assessment of flood hazards due to dam failure.

APPLICABILITY

This ISG shall be implemented on the day following its approval. It shall remain in effect until it has been superseded or withdrawn.

PROPOSED GUIDANCE

This ISG is applicable to holders of operating power reactor licenses and construction permits under 10 CFR Part 50. For combined license holders under 10 CFR Part 52, the issues in NTTF Recommendations 2.1 and 2.3 regarding seismic and flooding reevaluations and walkdowns are resolved and thus, this ISG is not applicable.

IMPLEMENTATION

Except in those cases in which a licensee or construction permit holder under 10 CFR Part 50 proposes an acceptable alternative method for the assessment of flood hazards due to dam failure, the NRC staff will use the methods described in this ISG to evaluate the results of the assessment.

BACKFITTING DISCUSSION

This ISG does not constitute backfitting as defined in 10 CFR 50.109 (the Backfit Rule) and is not otherwise inconsistent with the issue finality provisions in Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants," of 10 CFR. This ISG provides guidance on an acceptable method for responding to a portion of an information request issued pursuant to 10 CFR 50.54(f). Neither the information request, nor the ISG require the modification or addition to systems, structures, or components, or design of a facility. Applicants and licensees may voluntarily use the guidance in JLD-ISG-2013-01 to comply with the request for information. The information received by this request may, at a later date, be used in the basis for imposing a backfit. The appropriate backfit review process would be followed at that time.

FINAL RESOLUTION

The contents of this ISG, or a portion thereof, may subsequently be incorporated into other guidance documents, as appropriate.

ENCLOSURES

Guidance for Assessment of Flooding Hazards Due to Dam Failure

REFERENCES

1. U.S. Nuclear Regulatory Commission, "Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," March 12, 2012, Agencywide Documents Access and Management System (ADAMS) Accession No. ML12053A340.
2. U.S. Nuclear Regulatory Commission, "Recommendations for Enhancing Reactor Safety in the 21st Century, The Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," Enclosure to SECY-11-0093, July 12, 2011, ADAMS Accession No. ML111861807.

3. U.S. Nuclear Regulatory Commission, "Recommended Actions to be Taken Without Delay From the Near Term Task Force Report," SECY-11-0124, September 9, 2011, ADAMS Accession No. ML11245A158.
4. U.S. Nuclear Regulatory Commission, "Prioritization of Recommended Actions to be Taken in Response to Fukushima Lessons Learned," SECY-11-0137, October 3, 2011, ADAMS Accession No. ML11272A111.
5. U.S. Nuclear Regulatory Commission, "Staff Requirements - SECY-11-0093 - Near-Term Report and Recommendations for Agency Actions Following the Events in Japan," August 19, 2011, ADAMS Accession No. ML112310021.
6. U.S. Nuclear Regulatory Commission, "Staff Requirements - SECY-11-0124 - Recommended Actions to be Taken Without Delay From the Near-Term Task Force Report," October 18, 2011, ADAMS Accession No. ML112911571.
7. U.S. Nuclear Regulatory Commission, "NRC Responses to Public Comments, Japan Lessons-Learned Project Directorate Interim Staff Guidance (JLD-ISG-2013-01): Guidance for Assessment of Flooding Hazards Due to Dam Failure in Response to the March 2012 Request for Information Letter," July 29, 2013, ADAMS Accession No. ML13151A153.

Guidance for Assessment of

Flooding Hazards Due to Dam Failure

Enclosure

Table of Contents

Tables

Figures

Guidance for Assessment of Flooding Hazards Due to Dam Failure

1. INTRODUCTION

When evaluating flooding hazards for nuclear power plants, floods resulting from dam failures need to be considered. In engineering terms, dams and levees fail when they do not deliver the services for which they are designed, such as flood protection, water supply, and hydropower. However, this interim staff guidance (ISG) defines failure from a point of view at the nuclear power plant (NPP). Therefore, in this ISG dam failure refers to flooding caused by any uncontrolled release of water that threatens to impact structures, systems and components (SSCs) important to safety at a NPP site.

It should also be noted that there may be instances where a controlled release of water from a dam can also lead to the inundation of a NPP site. Examples include, but are not limited to: (a) releases performed in order to prevent dam failure during flood conditions; (b) releases performed to rapidly drawdown a reservoir to prevent incipient failure after a seismic event; and (c) releases performed to rapidly drawdown a reservoir to prevent incipient sunny day failure.

In some cases, the elevation of the site provides the principle protection from flooding hazards. In some cases, SSCs important to safety are protected by passive (e.g., structures), or active (e.g., equipment), flood protection features. In other cases, flood protection is provided by procedures. NPPs may also use some combination of the protection methods outlined above. Therefore, the site elevation and the lowest flood protection elevation of SSCs important to safety are the primary criteria for flood hazard assessment.

In general, failure of any dam upstream from the plant site is a potential flooding mechanism (consideration of upstream dams should include all water-impounding structures, whether or not they are defined as dams in the traditional sense). Dams that are not upstream from the plant, but whose failure would impact the plant because of backwater effects, may also present potential flooding hazards. Failures of dikes or levees in the watershed surrounding the site may contribute to or ameliorate flooding hazards, depending on the location of the levee and the circumstances under which it fails.

Failure of a dam or levee that impounds the ultimate heat sink constitutes a hazard to the plant. In addition, failures of onsite water-storage or water-control structures (such as onsite cooling or auxiliary water reservoirs and onsite levees) that are located at or above the grade of SSCs important to safety are potential flooding mechanisms.

The dam failure itself may be due to flooding or some other cause such as a seismic event, a structural defect, or human performance related issues. The potential for these mechanisms to initiate dam failure, as well as the potential failure modes must be evaluated to fully characterize the dam failure flooding hazard.

1.1 Purpose

The purpose of this ISG is to provide guidance on methods acceptable to U.S. Nuclear Regulatory Commission (NRC) staff for re-evaluating flooding hazards due to dam failure for the purpose of responding to the March 2012 request for information (Agencywide Documents Access and Management System (ADAMS Accession No. ML12053A340).

However, licensees are not required to follow this guidance. Approaches and methods that differ from those presented in this ISG will be evaluated on a case-by-case basis. It should be noted that dam failures discussed as a result of applying this guidance are postulated solely to ensure the safety of a NPP. This guidance should in no way supersede or be used in place of guidance developed by any agency that owns, operates or regulates the dam(s) of interest.

This ISG supplements and clarifies other NRC guidance that discusses dam failure, such as:

- Regulatory Guide 1.59, “Design Basis Floods for Nuclear Power Plants”, Revision 2 (USNRC, 1977)[1]
- NUREG-0800, “Review of Safety Analysis Reports for Nuclear Power Plants” (USNRC, 2007)
- NUREG/CR-7046, “Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America”, (USNRC, 2011)

This ISG is intended to provide guidance that is broadly consistent with published federal guidance from agencies that have direct responsibility for ownership, operation or regulation of dams, or direct responsibility for emergency planning and response for dam failure incidents. Therefore, this guide draws from guidelines developed by the Federal Emergency Management Agency (FEMA), Federal Energy Regulatory Commission (FERC), the Bureau of Reclamation (USBR), and the U.S. Army Corps of Engineers (USACE). Some portions of this guidance draw from dam safety guidelines developed by states, including California, Colorado, and Washington. A draft white paper on dam failure prepared by the Nuclear Energy Institute was also used.

Although this ISG is broadly consistent with best practices identified in the Federal and State guidance discussed above, there may be differences. In some cases, guidance is not uniform across agencies. In some cases, variance between this ISG and guidance of other agencies is due to differences in risk tolerance levels between the nuclear power sector and sectors such as water resources and flood control.

Certain widely-used modeling software packages are mentioned in this ISG for illustrative purposes, but the NRC does not recommend or endorse specific software packages. In general, hydrologic, hydraulic, geotechnical and structural simulation models accepted in standard engineering practice by Federal agencies and other authorities responsible for similar design considerations may be used. Justification for selection and use of a particular modeling method, approach or software package is the responsibility of the licensee.

1.2 Scope

A prioritization memo dated May 11, 2012 (ADAMS Accession No. ML12097A509) specified three categories for submittal of reevaluated flooding hazard reports. Those plants in Category 1 should have already submitted their flood reevaluation report by March 11, 2013 (unless an extension has been granted), which predates issuance of this ISG. Therefore, this ISG is not strictly applicable to Category 1 sites that have submitted their completed flood reevaluation report, and their dam failure flood hazard evaluations will be reviewed using present-day methodologies and regulatory guidance, as described in the request for information. This ISG is applicable to Category 2 and Category 3 sites (i.e., sites with

[1] Regulatory Guide 1.59, Rev. 2 included ANSI Standard N170-1976, “Standards for Determining Design Basis Flooding at Power Reactor Sites” as an appendix. ANSI-N170-1976 was superseded by ANSI/ANS-2.8, “Determining Design Basis Flooding at Power Reactor Sites” in 1981. ANSI/ANS-2.8 was last updated in 1992. It has lapsed as an ANSI standard, but is still used by NRC staff.

submittal dates of March 12, 2014 and 2015, respectively), as well as most Category 1 sites that have been granted an extension. Instances where Category 1 sites have been granted a very short extension (e.g. a few weeks), will be considered on a case-by-case basis.

This ISG is applicable to estimating flood hazards due to failure of all offsite and some onsite water control structures and impoundments. For offsite structures, hydrologic, seismic and sunny-day failure mechanisms are within the scope of the R2.1 Flooding Hazard Reevaluation and this ISG. This ISG provides guidance that is applicable to the evaluation of onsite dams and levees, including dam- or levee-like structures associated with onsite reservoirs (e.g., earthen cooling reservoir impoundments). Thus, while Section 2.4.4 of NUREG-0800 includes failure of all onsite water control or storage structures (e.g., levees, dikes, and any engineered water storage facilities that are located above site grade and may induce flooding at the site such as tanks and basins), this ISG provides guidance applicable to only a subset of those onsite structures. For example, even though the evaluation of site flooding from structures such as concrete cooling tower basins is within the scope of the NTTF Recommendation 2.1 flood hazard reevaluations, provision of guidance to support evaluation of such structures is not within the scope of this ISG. Moreover, evaluation of flooding from tanks is not within the scope of the NTTF Recommendation 2.1 flood hazard reevaluations and associated guidance is not provided in this ISG. Seismic failure of onsite structures may require input from the R2.1 Seismic Reevaluations.

1.3 Framework for Dam Failure Flood Hazard Estimation

1.3.1 Screening

Any sufficiently large watershed in the U.S. typically contains many dams (hundreds to thousands for major watersheds). It is generally not practicable to perform detailed failure analysis on each dam in the watershed. Even if it were a tractable problem, a large number of the dams in the watershed will have no impact on flooding at a NPP due to some combination of small size or large distance from the NPP. Therefore, it is useful to perform a screening level analysis to identify these dams. Section 3 describes several procedures for identifying the small/distant dams whose failure would likely have negligible impacts on flooding at a NPP site. The approach identifies several classes of dams for the purposes of this ISG. Dams that can be removed from consideration without analysis because they meet criteria described in ISG Section 3.1 (e.g., dams not owned by a NPP licensee and identified by Federal or State agencies as having minimal or no adverse failure consequences beyond the dam owner's property), are called "inconsequential" dams. Dams that can be shown to have little impact on flooding at a NPP site using simplified analyses (as described in Section 3) are termed "noncritical" dams. All other dams are considered "potentially critical." Detailed analyses will be required to further assess these dams. Detailed analysis will show which of the "potentially critical" dams are truly critical to flood hazard estimates at a NPP site. Critical dams are those whose failure, either alone, or as part of a cascading or multiple dam failure scenario, would cause inundation of a NPP site. Figure 1 illustrates the screening concept and the various dam classes. Details of the screening methods are provided in Section 3.

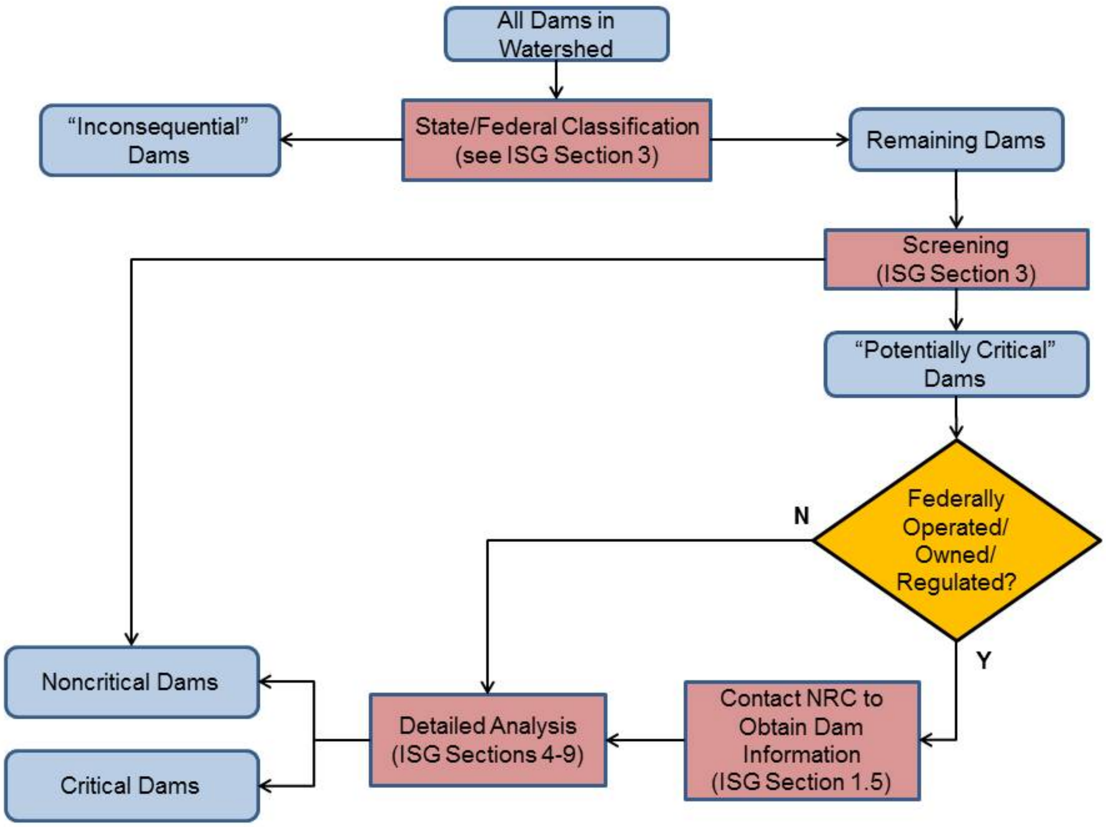

Figure 1. Levels of Analysis

1.3.2 Detailed Analysis

For potentially critical dams (e.g., those not screened out as discussed in the preceding section), the first step in detailed dam failure flood hazard estimation is determining the demand or loading cases that will be applied to the dam. For the purposes of responding to the March 2012 Request for Information (NRC, 2012), failure under hydrologic loadings associated with extreme floods, as well as ground motions associated with earthquakes must be considered. In addition, failure due to non-hydrologic, non-seismic causes (i.e., sunny day failures) must be considered. Sunny day failures encompass a wide variety of mechanisms (e.g., geologic or structural defects, misoperation, etc.).

Detailed dam failure flood hazard estimation will require collecting data on the dam(s) to be analyzed (e.g., design documents, construction records, maintenance, and inspection program, planned modifications) as well as hydrometeorological and hydrologic data (e.g., design storms, topography, rainfall-runoff characteristics) on the river basin(s) in question. Typically, information about the dam is obtained from the dam owner and/or regulator. In the U.S., there is no single entity responsible for regulation of dams. Instead, dam regulation is distributed among various Federal agencies and State authorities. Dams may be privately owned, or owned by Federal, State or local agencies. Many large dams on major rivers are owned by self-regulating Federal agencies (e.g., the USACE, USBR, or the Tennessee Valley Authority (TVA)). Information on the physical characteristics and flooding history of many watersheds can be obtained from Federal agencies (e.g., the U.S. Geological Survey (USGS), or National Oceanographic and Atmospheric Administration/National Weather Service (NOAA/NWS)), states, and organizations such as river basin commissions and flood plain managers.

Existing estimates for design storms and floods (e.g., probable maximum precipitation (PMP) and probable maximum flood (PMF)) in the region of interest developed by Federal, State or other agencies may be used. However, some of these reports may be quite old (e.g., the NOAA/NWS Hydrometeorological Report 51 for the Eastern United States was published in 1978). The licensee should exercise due diligence and examine the record of extreme storms and floods in the region of interest to provide assurance that the existing estimates are still valid.

Once the demand or loads have been estimated, the capacity of the dam to withstand the estimated loads is considered. The level of detail and effort expended will depend on several factors including, but not limited to, consequence of failure (e.g. a very large dam or a dam very near an NPP site versus a very small dam or one that is very far away), availability of design and construction information, and availability of recent studies to support capacity estimates (e.g., spillway capacity ratings, seismic capacity ratings, inspection and maintenance records, etc.). In place of a detailed analysis, one can simply assume that the dam fails under appropriate loading and move on to estimation of the consequences.

Comparison of the estimated capacities to the applied loads is used to assess the credibility of failure modes associated with those cases. The assessment may consider factors of safety incorporated into the dam design or dam capacity assessments, with appropriate justification. Likewise, uncertainties in capacity and loading estimates should be considered to arrive at an appropriately conservative decision. If it cannot be demonstrated that the dam-failure likelihood over the expected remaining life of the nuclear power plant is extremely low (or that the consequences of failure are negligible), failure should be postulated and the flooding consequences estimated. It is recognized that such assessments will often require a combination of deterministic, qualitative probabilistic, and/or quantitative probabilistic analysis. For example, current NRC guidance accepts deterministic analysis of hydraulic hazards (e.g., PMP, PMF). Deterministic analyses of capacity to withstand loads that were arrived at by probabilistic or deterministic analysis are also accepted. Detailed guidance on identifying potential failure modes is beyond the scope of this ISG. The USBR and the USACE have jointly developed guidance on this topic (USBR 2011).

Dam failure consequence analysis will generally include estimating the reservoir outflow hydrograph (discharge hydrograph) resulting from dam failure (dam-breach analysis) and routing of the dam breach discharge to the plant site. The flood routing analysis should consider any potential for domino-type or cascading dam failures. Transport of sediment and debris by floodwaters should be considered.

In summary, the dam-failure flood hazard analysis for potentially critical dams will comprise the following steps (see also Figure 2):

1. Data collection
    a. Compile information on the dam(s) (design, construction, inspection, maintenance, etc.)
    b. Compile information on the river basin upstream and downstream from each dam (topography, bathymetry, reservoir volumes, reservoir flood inflows, etc.)
2. Estimation of demand and loads
    a. Flooding case
    b. Seismic case
    c. Sunnyday case (assumed to occur)
3. Assess credible failure modes/scenarios under the various loading cases (flooding, seismic, sunny day), including potential for multiple or cascading failures
    a. Compare loadings to estimated capacities, taking into account uncertainties as well as factors of safety
    b. For each credible failure, perform steps 4 through 6
    c. If the failure is not considered credible, the analysis is complete
4. Breach analysis
    a. Estimate breach parameters (geometry and failure time)
    b. Compute reservoir routing and breach hydrograph
5. Flood routing
    a. Establish initial and boundary conditions
    b. Select hydrodynamic modeling approach and develop basin model (one-dimensional, two-dimensional, or hybrid)
    c. Perform flood-routing simulations
    d. Estimate impacts of sediment and debris transport
    e. Estimate Water-Surface Elevation (WSE) at plant site.
6. Inundation mapping—develop maps delineating the areas and structures at the plant site that would be inundated in the event of dam failure

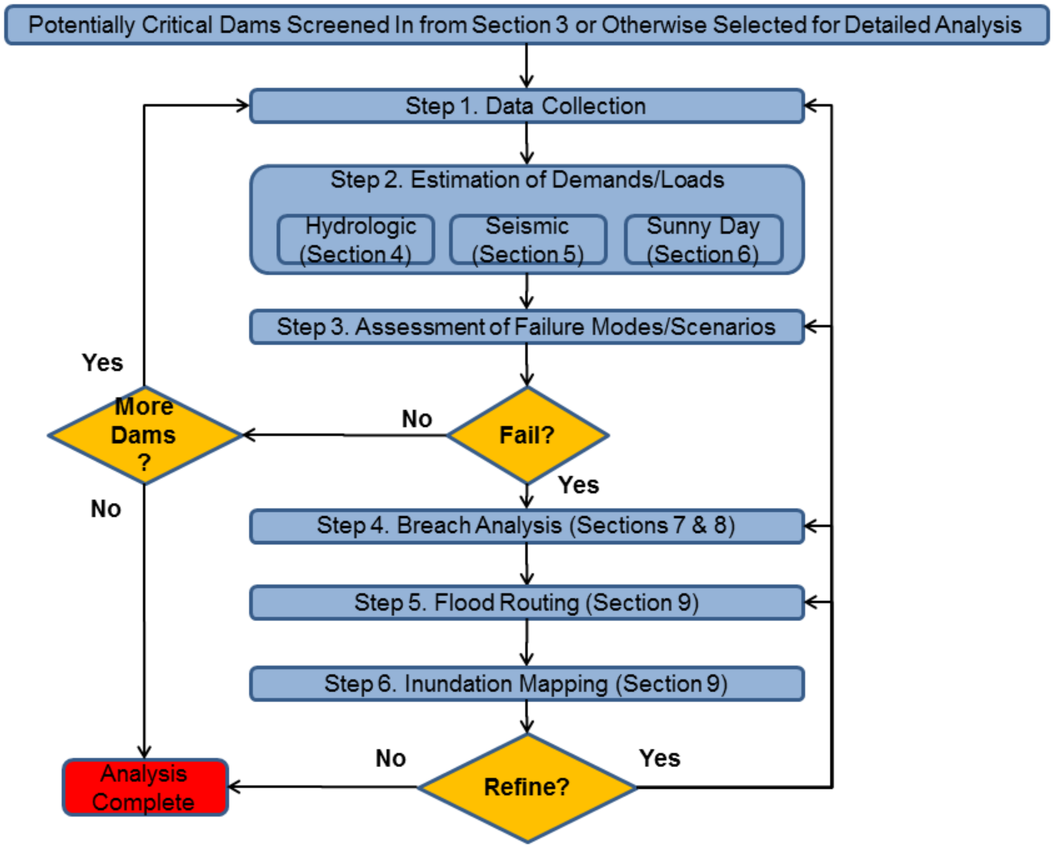

Figure 2. Overview of Detailed Dam Failure Flood Hazard Analysis

1.4 Probabilistic and Deterministic Hazard Analysis

The current state of practice in dam safety analysis uses a combination of deterministic and probabilistic approaches. Probabilistic seismic hazard analysis is accepted current practice in both the nuclear and dam safety communities, as reflected in Federal guidance and industry consensus standards. Probabilistic approaches for estimating the extreme rainfall and flood events of interest in this ISG (e.g. $1x\ 10^{-4}$ per year or lower annual exceedance probability) exist, but there are no industry consensus standards or Federal guidance that defines current accepted practice. NRC has established probabilistic screening criteria for man-related hazards (e.g., between $1x10^{-7}$ and $1x10^{-6}$ annual exceedance probability) that are, in theory, applicable to sunny-day dam failures. However, no widely accepted methodology exists for estimating sunny-day dam failure probabilities on the order of $1x10^{-7}$-$1x10^{-6}$ annual exceedance probability.

1.4.1 Historical Dam and Levee Failure Rates

Nearly 1,500 dam failures have been recorded in the United States since the middle of the 19th century (records are unreliable for prior periods). Over this period, the long-term average rate of dam failures is about 10 per year, although this figure represents dams of all sizes and types, including small dams, whose failures have little or no consequences (National Academy of Sciences (NAS), 2012). If instead one looks at a running 10-year average of the dam failure rates since 1850, the failure rate has been over 20 per year for most of the period since the late 1970s (NAS, 2012). Expressed in terms of dam years, numerous studies of dam failures in the U.S. and worldwide have indicated an average failure rate on the order of 10^{-4} per dam year (e.g., Baecher et al., 1980). These historical rates for dam failure provide useful information about generic failure probabilities, but generic dam failure rates are not sufficient to screen out dam failure for the purpose of the Recommendation 2.1 flooding hazard reevaluations (see ADAMS Accession No. ML090510269).

1.4.2 Hydrologic Failure

Probabilistic approaches for estimating the extreme rainfall and flood events of interest in this ISG (e.g. $1x\ 10^{-4}$ per year or lower annual exceedance probability) exist, but there are no industry consensus standards or Federal guidance that defines current accepted practice. Therefore, for the purpose of the Recommendation 2.1 Flooding Hazard reevaluation, a deterministic approach based on the PMP and PMF should be used. The PMP is an estimate of the maximum possible precipitation depth over a given size catchment for a given length of time (Stedinger et al., 1996). The PMF is the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the drainage basin under study. Consult Section 4 for additional detail on hydrologic failure.

Staff Position:

A dam should be assumed to fail due to hydrologic hazard if it cannot withstand its basin specific PMF, with associated effects.

1.4.3 Seismic Failure

Probabilistic seismic hazard analysis (PSHA) is considered to be present-day methodology in both the dam safety and the nuclear safety communities. Estimation of seismic hazards (e.g., vibratory ground motion, fault displacement, loss of strength) at annual exceedances of 10^{-4} per year is routine. Widely accepted earthquake source characterization data sets, ground motion prediction equations, and site amplification factors are publicly available. Section 5 provides additional detail on analysis of seismic dam failures.

Staff Position:

A dam should be assumed to fail if it cannot withstand the relevant seismic hazards (e.g., vibratory ground motion at spectral frequencies of importance, fault displacement, loss of strength) with an annual exceedance probability of $1x10^{-4}$ per year. Although the probability of extreme flooding occurring at the same time as an earthquake is extremely low, the probability of lesser floods should not be neglected. In addition, if the seismic capacity of the dam is considerably less than what is required to withstand the 10^{-4} seismic hazard, the possibility of large (though not extreme) floods should be considered. Therefore, the dam should be assumed to fail due to seismic hazard if it cannot withstand the more severe of the following combinations:

- 10^{-4} annual exceedance seismic hazard combined with a 25-year flood
- half of the 10^{-4} ground motion, combined with a 500-year flood.

1.4.4 Sunny-Day Failure

The hydrologic and seismic failures discussed in the previous subsections require a natural hazard initiator, and can therefore be considered, at least in part, a natural hazard. On the other hand, sunny-day failures are clearly a purely man-related hazard.

The NRC's traditional approach to assessing impacts from man-related hazards is provided in the Standard Review Plan for the Safety Review of Nuclear Power Plants (NUREG-0800), Sections 2.2.1 through 2.2.3. The NRC considers that design-basis events resulting from the presence of hazardous materials or activities in the vicinity of the plant is acceptable based on estimated annual frequency. If a postulated accident type meets the NRC staff objective (with an order of magnitude of 10^{-7} per year) then the potential exposures are considered to meet the requirements of Title 10 of the *Code of Federal Regulations* (10 CFR) 50.34(a)(1) as it relates to the requirements of 10 CFR Part 100, "Reactor Site Criteria." If data are not available to make an accurate estimate of the event probability, an expected rate of occurrence of potential exposures resulting in radiological dose in excess of 10 CFR 50.34(a)(1) as relates to the requirements of 10 CFR Part 100, by an order of magnitude of 10^{-6} per year is acceptable if, when combined with reasonable qualitative arguments, the realistic probability can be shown to be lower. This exception is made because data are often not available to enable the accurate calculation of probabilities given the low probabilities associated with the events under consideration.

The approach outlined in the preceding paragraph has been applied to man-related hazards such as those associated with industrial and transportation activities. However, as discussed above in the introductory paragraph to Section 1.4, even when failure probability estimates are developed based on site and dam specific data and information, no widely accepted current engineering practice exists for estimating failure rates on the order of at the 1×10^{-6} per year.

Staff Position:

> Because no widely accepted current engineering practice exists for estimating failure rates on the order of at the 1×10^{-6} per year, sunny-day failure should be assumed to occur and the consequences estimated.

1.5 Interfacing with Owners and Regulators of Dams and Levees

There are roughly 84,000 dams (USACE, 2011b) and over 100,000 miles of levees (National Committee on Levee Safety (NCLS), 2009) in the U.S., constructed by a variety of public sector agencies (local, State and Federal) as well as numerous private sector entities (e.g., individuals, groups, and corporations). Dam and levee safety program governance in the United States is shaped by laws, policies, and practice, and is similar to the governance that has evolved for emergency response in the U.S. (NAS, 2012).

1.5.1 Dam Safety Governance

In general, State and local governments have responsibility for dam safety governance of non-Federally owned or operated dams. Almost all states have formal dam safety programs tied to Federal guidelines. FEMA has recently published a summary of existing dam safety guidance that provides information on individual states' dam safety programs (FEMA, 2012).

Federal regulatory authority for non-Federal dams is limited to the roughly 2,100 dams used for hydropower projects regulated by the Federal Energy Regulatory Commission (FERC), and mine-tailings dams regulated by the Mine Safety and Health Administration. In a few cases, states have jurisdiction over dams that are also regulated by a Federal agency (e.g., California regulates hydropower dams).

Federally owned dams are regulated not by an independent agency but according to the policies and guidance of the individual Federal agencies that own the dams. Table 1 summarizes Federal dam ownership and dam safety roles.

1.5.2 Dam Safety Guidance by Other Federal Agencies

At the Federal level, FEMA has been charged with encouraging the establishment and maintenance of effective Federal and State programs, policies and guidelines. It implements this charge through leadership of the National Dam Safety Program (NDSP), the National Dam Safety Review Board (NDSRB), and the Interagency Committee on Dam Safety (ICODS). ICODS has generated and released a series of guidance documents in an attempt to provide a uniform and consistent dam safety framework for Federal, State, and private dam owners and regulators. However, adherence to this guidance is not mandatory. For example, FEMA has oversight responsibility for developing guidance, but no direct regulatory authority for dam safety. Other Federal agencies such as USACE, the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service, the U.S. Department of the Interior (USDOI) Bureau of Reclamation, and FERC have published dam safety guidelines. FEMA has recently published a summary of existing dam safety guidance that provides information on Federal dam safety programs (FEMA, 2012).

Table 1. Roles of Federal Agencies in Dam Safety[1]

Agency	Primary Roles	Dams under Jurisdiction
U.S. Department of Homeland Security, Federal Emergency Management Agency (FEMA)	Lead agency for National Dam Safety Program; Chairs National Dam Safety Review Board and Interagency Committee on Dam Safety	Does not own any dams
U.S. Department of Agriculture (USDA)	Owns or regulates dams; Supports private owners with planning, design, finance, and construction	More than one-third of dams in the National Inventory of Dams (NID) are associated with the USDA
U.S. Department of Defense (DOD)	Plans, designs, finances, constructs, owns, operates, and permits dams; limited to military lands with exception of USACE civil-works programs	DOD has total of 267 dams under its jurisdiction on military lands
U.S. Army Corps of Engineers (USACE)	Plans, designs, constructs, operates, and regulates dams; permits and inspects dams	Jurisdiction over USACE dams, dams constructed by USACE but operated by others, and other flood-control dams subject to Federal regulation; 631 dams in the NID are associated with USACE
U.S. Department of the Interior, (USDOI)	Plans, designs, constructs, operates and maintains dams	About 2,000 dams in the NID under five bureaus, mainly Bureau of Reclamation

U.S. Department of Labor (USDOL)	Regulates safety and health-related aspects of miners	About 1,400 dams under Mine Safety and Health Administration
Federal Energy Regulatory Commission (FERC)	Issues licenses, performs inspections, and regulates non-Federal dams with hydroelectric capability	2,530 dams in NID affecting navigable waters
Tennessee Valley Authority	Plans, designs, constructs, operates, and maintains dams	Approximately 49 major dams in Tennessee River Valley

[1] NRC regulates dams providing ultimate heat sink at NPPs as well as tailings dams at uranium mill tailings sites.

Source: NRC (2012) and FEMA (2009)

1.5.3 Obtaining Information on Dams and Levees

Obtaining detailed information to support the dam failure flood hazard evaluation may be challenging because of the dispersed nature of dam ownership and regulation in the United States. In most cases, licensees do not operate or own the dams or levees that potentially may contribute to flooding hazards at a NPP site.

National and State dam inventories can be used to identify dams within the watershed of a stream or river and to obtain characteristics for each dam (location, height, and volume). The USACE maintains the National Inventory of Dams (NID), which provides information on thousands of dams (USACE, 2011b). Following Hurricane Katrina, Congress authorized the USACE to develop a National Levee Database (NLD). Initially, the NLD contained information only for USACE levees. However, integration of levee data collected by the FEMA National Flood Insurance Program (NFIP) into the NLD, which is under way, will increase the total number of miles of levee systems in the NLD. These databases and inventories are useful sources of basic geographic and physical information on dams and levees in the U.S.

Staff Positions:

- In the case of dams and levees owned or operated by U.S. Federal agencies, the Federal agency responsible for (owner or operator of) the dam should be involved in any discussions, including possibly reviewing any analysis performed. Evaluation of dams is complex, requiring extensive expertise and site-specific knowledge. It is critical for the owner or operator of the dam to assist the NRC or its licensees when modifying the assumptions or methods used to develop the inundation maps for a specific area. If a Federally owned dam is identified as critical to the flooding reanalysis, the licensee should contact the NRC promptly. The NRC will act as the interface between these agencies and licensees. Memoranda of Agreement or other mechanisms are being developed to facilitate sharing of data (including necessary safeguards to protect sensitive information) between the NRC and the appropriate Federal agencies. It is important to note that in many cases Federal agencies that own or operate dams have conducted detailed failure analysis. To the extent that these analyses are applicable, they should be used in the Recommendation 2.1 flooding reanalysis.

- In some cases, the dam or levee will be owned or operated by a private entity, but regulated by a Federal agency. In this case, the NRC will interface with the Federal regulatory agency to obtain available information. Interactions between the licensee and the owner should be coordinated with the NRC and the Federal regulator.
- In most cases, dams and levees will be owned and operated by private entities and regulated by a State agency. In this case, the licensee should interact directly with the owner and regulator. The licensee should notify the NRC if it encounters difficulties in obtaining information. On a case-by-case basis, the NRC might be able to provide some assistance in interfacing with State agencies.

1.6 Organization of guidance

ISG Section 1 provides an overview of the flood hazard reevaluation process; discusses dam safety governance and existing guidance and discusses procedures for obtaining information on dams and levees, particularly in regard to those that are Federally owned or regulated. ISG Section 2 presents an overview of dams and levee types and causative mechanisms for dam failure. ISG Section 3 discusses screening methods and simplified analysis approaches for drainage basins with many small dams. ISG Section 4 discusses dam failure due to hydrologic mechanisms. ISG Section 5 discusses dam failure due to seismic mechanisms. ISG Section 6 discusses dam failure resulting from causes other than hydrologic or seismic mechanisms (e.g., sunny-day failures). ISG Section 7 provides details of dam breach modeling. ISG Section 8 discusses levee breach modeling. ISG Section 9 provides details of flood routing. ISG Section 10 contains a list of terms and definitions. ISG Section 11 lists the references used to develop this guidance.

2. BACKGROUND

This section of the ISG provides a brief overview of the various types of dams in common use, and the principal classes of mechanisms that initiate damage and failure. Failure mechanisms vary with the type of dam, as well as materials used in the construction.

2.1 Classification of Dams and Levees

Dams can be classified using several criteria (e.g., purpose, size, construction material, hazard potential, etc.). This ISG will use a classification system based mainly on construction material since modes of failure, as well as susceptibility to a given initiating mechanism, are generally correlated with dam construction material. The two major categories discussed in this ISG are concrete and embankment dams. There are also a large number of so-called composite dams comprised of both concrete and embankment sections currently in service.

A levee or dike is a manmade barrier (embankment, floodwall, or structure) along a watercourse constructed for the primary purpose to provide hurricane, storm, and flood protection relating to seasonal high water, storm surges, precipitation, and other weather events; and that normally is subject to water loading for only a few days or weeks during a year. Almost all levees are earthen embankments, although some may have parapets or floodwalls comprised of other materials built on their crest.

2.1.1 Concrete Dams

As the name implies, concrete dams are typically constructed of concrete or other masonry components. The major types of concrete dams include gravity, arch, multi-arch, and buttress dams (as shown in Figure 3 through Figure 6). Some dams are hybrids being combinations of the major types. For example, gravity-arch dams combine the features of arch and gravity designs. Multi-arch dams typically employ buttresses to support the arches (Figure 6).

Concrete gravity dams typically consist of a solid concrete structure that maintains the stability against design loads by relying on the geometric shape, and the mass and strength of the concrete (Figure 3). Conventionally placed mass concrete and roller-compacted concrete (RCC) are the two general concrete construction methods for concrete gravity dams (USACE, 1995a,b). Gravity dams depend primarily on their own weight for stability. Generally, gravity dams are sized and shaped to resist overturning, sliding, and crushing at the dam toe. If the moment around the turning point caused by the hydraulic load of the reservoir is smaller than the moment caused by the weight of the dam, the dam will not overturn. This is the case if the resultant force of hydraulic load and weight of the dam falls within the base of the dam. Typically, gravity dams are constructed on a straight axis, though they may be slightly angled or curved, in an arch shape. In earlier periods of dam design, gravity dams were built of masonry materials such as stone, brick, or concrete blocks jointed with mortar. Additionally, gravity dams can have a hollow interior with concrete or masonry used on the outside. Engineering manuals published by the U.S. Army Corps of Engineers (USACE, 1995a), the Bureau of Reclamation (USBR, 1976, 1977b), and the Federal Energy Regulatory Commission (FERC, 2002) provide more detailed discussion of concrete gravity dam engineering and design.

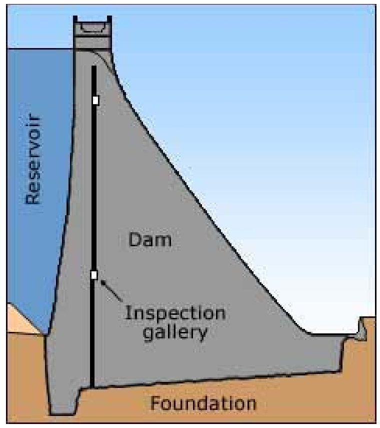

Figure 3. Section View of Concrete Gravity Dam (British Dam Society, 2013)

An arch dam is a structure that is designed to curve upstream so that the force of the water in the upstream reservoir presses against the arch, compressing and strengthening the structure as it pushes into its foundation or abutments (Figure 4). Because they are thinner than any other dam type, they require less material to construct, making them both economical and practical in remote areas. There are two basic designs for an arch dam: *constant-radius dams*, which have constant radius of curvature, and *variable-radius dams*, which have both upstream and downstream curves that systematically decrease in radius below the crest. Arch dams can be double-curved in both horizontal and vertical planes. Arch dams with more than one contiguous arch or plane are described as multiple-arch dams (see Figure 6). The foundation or abutments for an arch dam must be very stable with strength proportionate to that of the concrete. Engineering manuals published by several Federal agencies provide more detailed discussion of concrete arch dam engineering and design (USACE, 1994; USBR, 1977a,b; FERC, 1999).

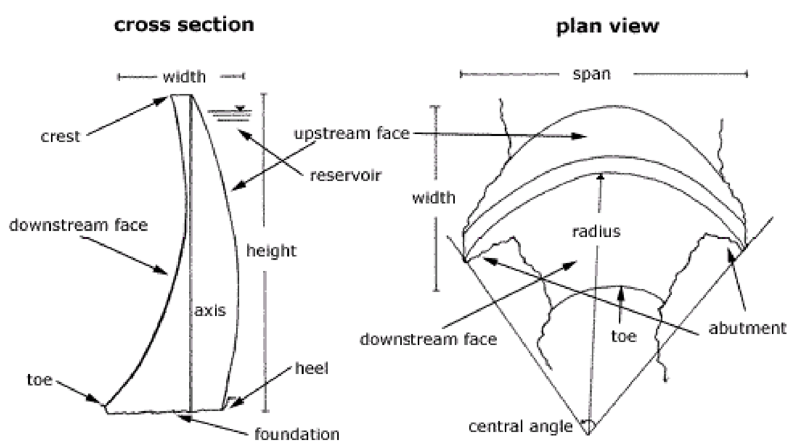

Figure 4. Typical Concrete Arch Dam Cross-Section and Plan View (Youssef, 2013)

Buttress dams are concrete structures consisting of two basic features: an upstream water barrier and buttresses (Figure 5). Buttress dams are typically designed as reinforced concrete structures. The upstream water barrier can be a flat slab or massive heads. The upstream water barrier transfers the reservoir load into the buttresses that then transfer the load into the foundation through frictional resistance like a gravity dam. Buttress dams can be thought of as hollowed-out gravity dams with a sloping upstream face. The sloping upstream face allows the buttresses to efficiently carry static loads because the weight of the water on the dam adds to the vertical force transmitted to the foundation and therefore the stability of the dam. Depending on the thickness of the concrete members, buttress dams might or might not have reinforcing steel. Engineering manuals published by several Federal agencies provide more detailed discussion of concrete buttress dam engineering and design (USBR, 1976, FERC, 1997).

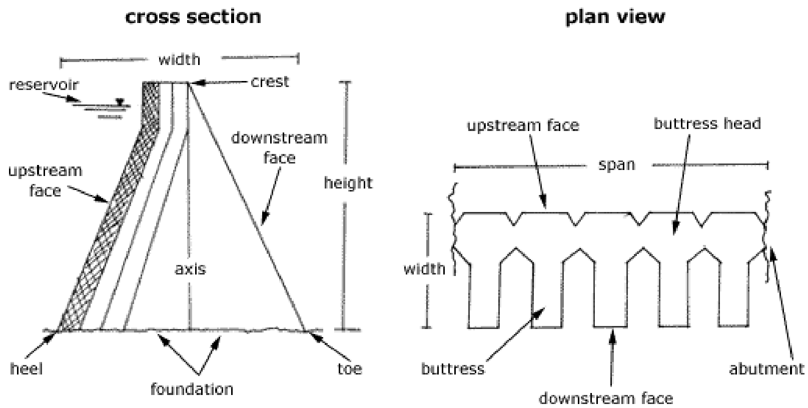

Figure 5. Concrete Buttress Dam Cross-Section and Plan View. (SimScience, 2013)

Figure 6. Concrete Multi-Arch Dam (Bartlett Dam. U.S. Bureau of Reclamation; Photo: National Park Service, 2013)

2.1.2 Embankment Dams

Embankment dams are made from compacted natural (earthen) materials. Earthfill dams are typically trapezoidal in shape and rely on their weight to resist the hydrostatic loads created by the water, similar to concrete gravity dams. The two most common types of embankment dams are rockfill and earthfill dams (see Figure 7 and Figure 8).

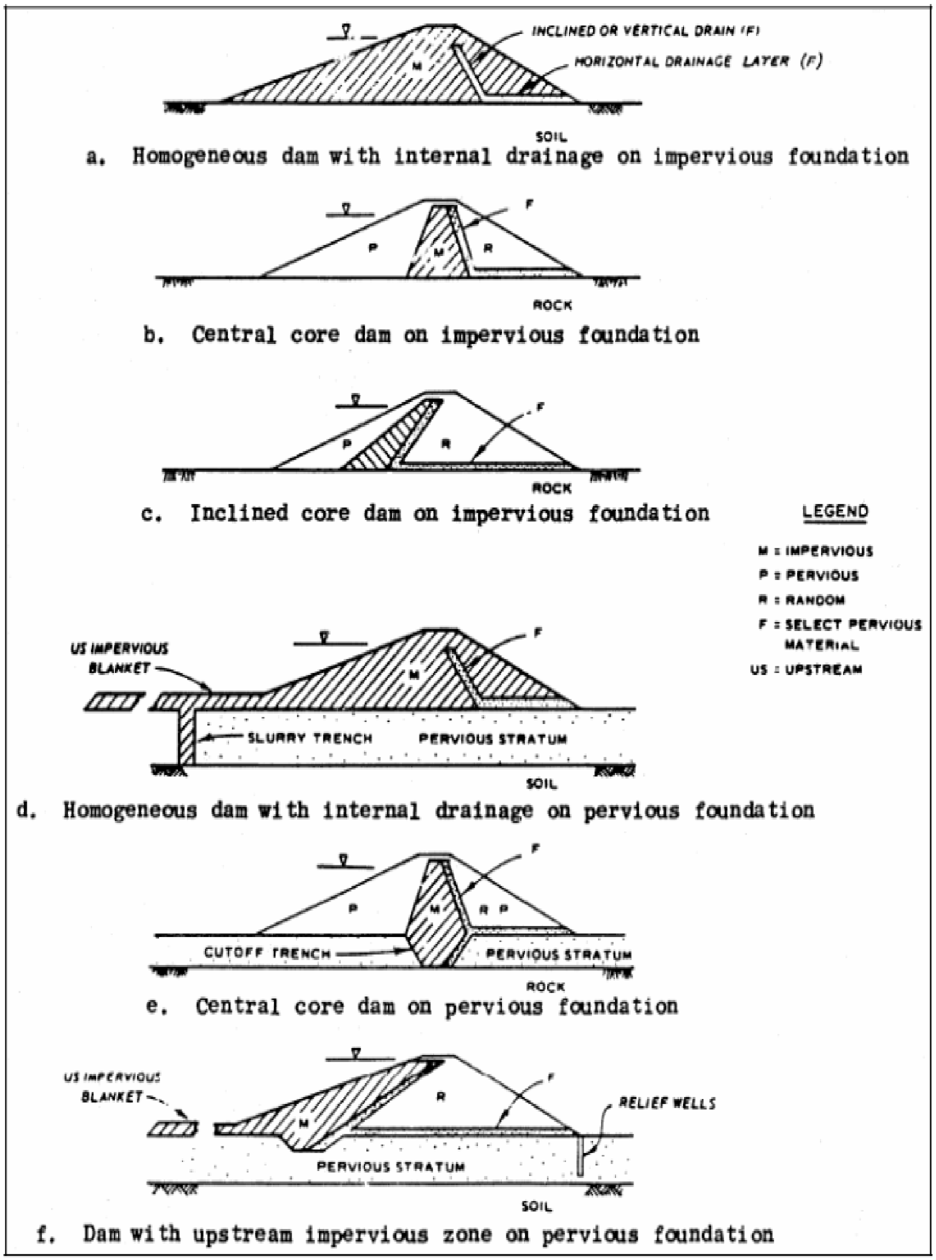

Figure 7. Typical Earthfill Embankment Dams (USACE, 2004)

Earthfill dams (Figure 7) are composed of suitable soils that are spread and compacted in layers by mechanical means. Earthfill dams can be constructed with homogenous layers (homogeneous dam) or zones of different materials of varying characteristics (zoned-earth dam). Typical zones include a clay core and filter and drain zones.

Rockfill dams (Figure 8) are constructed from compacted earth fill that contains a high percentage of rocks and other larger aggregate materials. To prevent seepage, rockfill dams have an impervious zone on the upstream side of the dam or within the embankment. The impervious zone can be made from a variety of materials including masonry, concrete, plastic, steel pile sheets, timber, or clay. If clay is used, it is often separated from the fill by a filter to prevent erosion of the clay into the fill material.

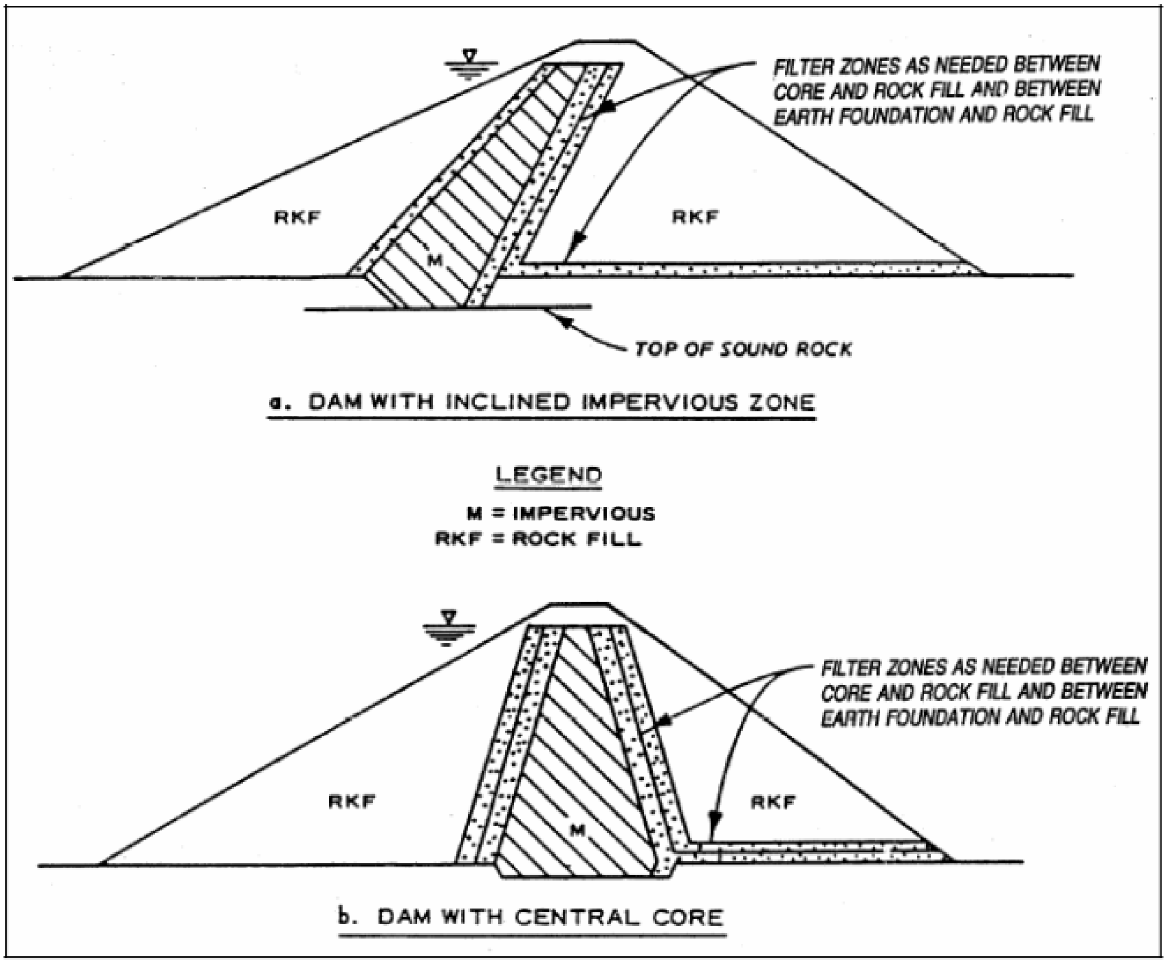

Figure 8. Typical Rockfill Dams (USACE, 2004)

Earthfill or rockfill dams can include a watertight core made from asphalt concrete. Dams with this type of core are called concrete-asphalt core embankment dams. Most concrete-asphalt dams use rock or gravel as the main fill material. These types of dams are considered especially appropriate for areas susceptible to earthquakes because of the flexible (elastic) nature of the asphalt core.

Engineering manuals published by several Federal agencies provide more detailed discussion of embankment dam engineering and design (USACE, 2004; USBR,1987; FERC, 1991).

2.1.3 Water Levels and Storage Volumes

The vocabulary and terminology used to describe storage volumes and corresponding water levels (pool levels) in the water resources and dam safety literature vary between agencies and practitioners. Figure 9 illustrates the terminology that has been adopted for use in this ISG (see also Section 10, Terms and Definitions).

Reservoir storage consists of dead storage, inactive storage, active storage, and flood surcharge storage. *Dead storage* is the volume in a reservoir below the lowest controllable level. *Inactive storage* is the capacity below which the reservoir is not normally drawn. Active (or usable) storage is the total amount of reservoir capacity normally available for release from a reservoir. *Active storage* is usually composed of conservation storage (used for water supply, irrigation, recreation, hydropower, navigation, etc.), and flood-control storage. The top of the active storage is referred to as the full pool or maximum normal pool elevation. *Flood surcharge storage* is volume between the maximum water surface elevation for which the dam is designed and the crest of an uncontrolled spillway (or the normal full-pool elevation of the reservoir with the crest gates in the normal closed position).

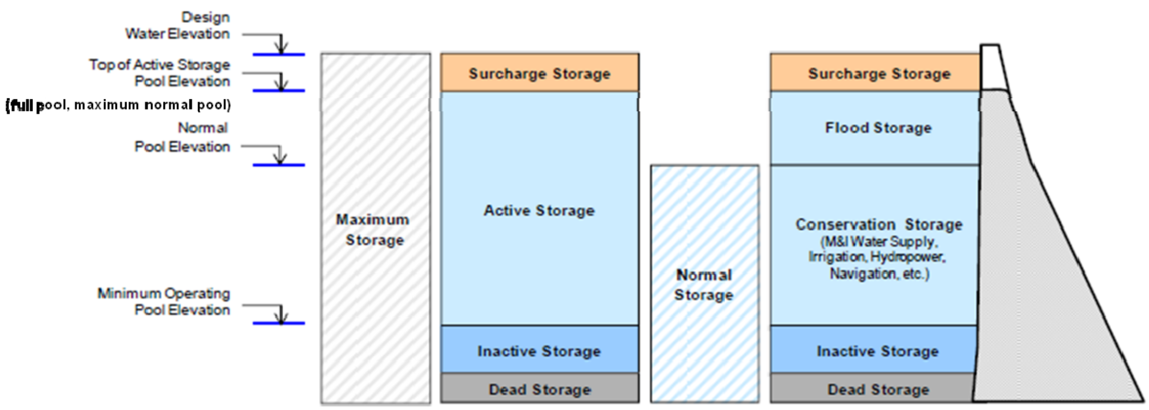

Figure 9. Reservoir water levels and corresponding storage volumes.

2.1.4 Levees

A levee or dike (Figure 10) is a manmade barrier (embankment, floodwall, or structure) along a water course that is 1) constructed primarily to provide hurricane, storm, and flood protection relating to seasonal high water, storm surges, precipitation, and other weather events; and 2) normally subject to water loading for only a few days or weeks during a year. Embankments that are subject to water load for prolonged periods (longer than normal flood-protection requirements) or permanently are sometimes referred to as "frequently loaded" levees; such levees should be designed in accordance with earth dam criteria rather than levee criteria. The potential failure of levees intended to provide flood protection to a NPP site should be considered. For the purposes of this ISG, distant levees are generally not of great concern. In general, levees should be assumed to fail when they are overtopped as a result of any mechanism (e.g., floods or dam break flow waves).

Levees might also include embankments, floodwalls, and similar types of structures intended to provide flood protection to lands below sea level and other lowlands and that might be subject to water loading for much, if not all, portions of the year, but do not constitute barriers across watercourses or constrain water along canals.

As with dams, levees incorporate features and appurtenances that are critical to proper functioning. Examples include floodwall sections, closure structures, pumping stations, interior drainage works, and flood damage reduction channels.

Most levees and dikes are constructed using clay, silt, or sand with a clay core or cover, often on a foundation of substrata that is also subject to erosion. The levee definition used here does not include shoreline protection or riverbank protection systems such as revetments, barrier islands, etc. Such shoreline or riverbank protection systems are hardened structures that inhibit erosion but do not necessarily hold back water. Natural coastal barriers often consist mostly of sandy material.

Even though levees are functionally similar to small earth dams they differ from earth dams in the following important respects: a) a levee embankment may become saturated for only a short period of time beyond the limit of capillary saturation; b) levee alignment is dictated primarily by flood protection requirements, which often leads to construction on poor foundations; and c) borrow soil is generally obtained from shallow pits or from channels excavated adjacent to the levee, which produce fill material that is often heterogeneous and far from ideal.

A levee system comprises one or more levee segments that collectively reduce flood damage to a defined area. A levee segment is a discrete portion of a levee system that is owned, operated and maintained by a single entity, or discrete set of entities. A levee segment might have one or more levee features. Highway and railroad embankments can be considered levees only if they are designed to function as part of a flood control system.

More detail on the design, maintenance, and inspection of levees and floodwalls can be found in several engineering manuals and technical letters published by the U.S. Army Corps of Engineers (USACE, 1989, 2000, 2011b).

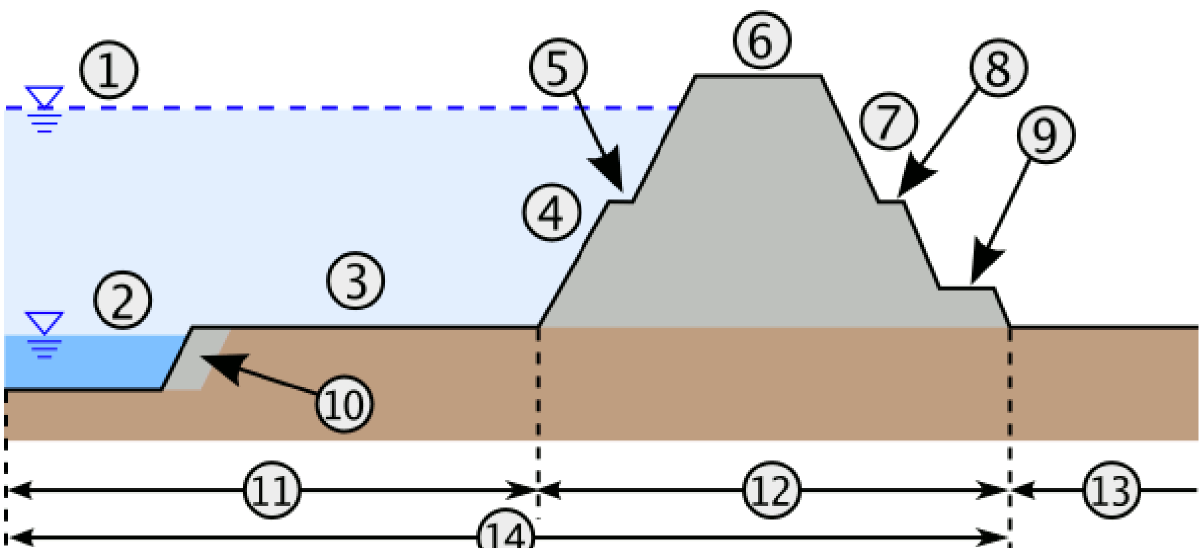

Figure 10. Typical Levee Cross-Section (Wikimedia Commons, 2013)

(1) design high water level (HWL); (2) low water channel; (3) flood channel; (4) riverside slope; (5) riverside banquette; (6) levee crown; (7) landside slope; (8) landside banquette; (9) berm; (10) low water revetment; (11) riverside land; (12) levee; (13) protected lowland; (14) river zone

2.2 Classification of Dam Failures

Dam failure is a complex phenomenon and the root causes of actual dam failures are sometimes difficult to determine precisely. Identification of potential failure modes can only be performed after thoroughly reviewing all relevant background information on a dam, including geology, design, analysis, construction, flood and seismic loadings, operations, dam safety evaluations, and performance and monitoring documentation. Many failure modes are progressive and can be subdivided into phases, typically including initiation, progression and, finally, breach and uncontrolled reservoir release.

Hydrologic dam failure refers to those failures that are initiated by a hydrologic event (e.g. inflow flood). The most common scenario is a large flood that overwhelms the dam spillway discharge capacity, with floodwaters overtopping the dam crest, leading to erosion of the downstream dam face, foundation materials, or abutments and eventual failure (breach).

Seismic dam failure occurs as the result of an earthquake (e.g., ground shaking, surface faulting, landsliding, or liquefaction). Strong ground shaking is the most common earthquake effect. Ground-shaking may directly damage the dam structure and appurtenances or induce subsequent failure modes (e.g., failure of gates leading to overtopping, reservoir landslide or seiche leading to overtopping, cracking or deformation of the embankment that leads to overtopping or an internal erosion failure).

In spite of their progressive nature, failures are often broadly categorized according to the predominant initiating mechanism for failure: (a) hydrologic dam failure, (b) seismic dam failure, and (c) dam failure from other causes. However, these categories are not mutually exclusive.

Dam failures not caused by a concurrent extreme flood or seismic event can arise from a wide variety of causes. Examples include, but are not limited to the following:

- latent design or construction errors
- age-related weakness or deterioration of embankment material, foundations, abutments or spillways
- malfunction or misoperation of appurtenances such as floodgates, valves, conduits, and other components may also lead to dam failure

Dam failures caused by hydrologic events are discussed in Section 4. Seismic dam failure is discussed in Section 5. Dam failures due to nonhydrologic, nonseismic events are discussed in Section 6.

2.2.1 Influence of Dam Type on Failure Modes

As mentioned in Section 2.1, predominant causes for failure, failure modes, and failure progression for dams depend on the dam type. If the dam fails, the breach shape and timing will also depend on the dam type.

2.2.1.1 Concrete Dams

In general, concrete dams are much stronger in compression than in tension. With the exception of buttress dams, concrete dams are typically made of plain concrete that possesses limited tensile strength. Therefore, dam structural response to tensile loads is best characterized using a classic stress-strain relationship composed of elastic and inelastic strain ranges followed by a complete loss of strength (failure). The inelastic-strain range provides only limited inelastic behavior. The dam response beyond this range is governed by complete loss of strength, sliding, and nonlinear response behavior of discrete blocks bounded by opened joints and cracked sections.

Because of the nature of their design and manner of construction, concrete dams can usually withstand some degree of overtopping. Some concrete dams actually are designed to be overtopped. However, overtopping can lead to erosion of the dam foundation and its abutments.

Observation and analysis indicate that degree and speed of failure for concrete dams depends on dam type. Because of their strength and mass, concrete gravity, dams are typically subject to partial rather than complete failure; failure typically is not instantaneous. By contrast, concrete arch dams typically fail completely, and almost instantaneously. For buttress and multi-arch dams, failure of one or more sections is much more common than complete failure. Failure of the sections is typically treated as essentially instantaneous.

2.2.1.2 Embankment Dams

Failure modes for embankment dams are heavily influenced by the design (e.g., homogenous vs. zoned), the materials (e.g., cohesive vs. noncohesive soils), and construction methods (e.g., degree of compaction) used.

Causes of failure for embankment dams under hydrologic load associated with flooding mainly fall into three categories: a) increased internal seepage rates; b) overtopping which initiates embankment erosion; and c) structural overstressing. The design, materials and construction methods employed will heavily influence failure initiation and progression in each of these categories. For example, cohesionless soils are less able to withstand erosion caused by overtopping, internal seepage pressures or structural overstressing than cohesive soils. The degree to which a given soil is compacted is an important factor determining its load-bearing capacity and resistance to erosion. Zoned dams with internal drainage layers and filters are better able to accommodate significant internal seepage rates.

The ability of an embankment dam to withstand earthquake shaking without loss of strength or liquefaction of foundation or embankment soils (leading to deformation, sliding, cracking or other failures) is very dependent on the materials and degree of compaction used. Design features such as conduits passing through the dam can be an important consideration in its behavior under seismic loading.

2.2.2 Failure of Spillways, Gates, Outlet Works, and Other Appurtenances

There are a number of dam features, not unique to any one dam type, important to dam functioning for which loss of function could directly cause uncontrolled release of the reservoir or lead to uncontrolled release of the reservoir through overtopping, erosion, or some combination thereof. Chief among these are spillways, gates and other outlet works. Spillway discharge capacity is usually the critical component in passing large floods. More details on spillway failure are provided in Section 4, which discusses hydrologic failure mechanisms

A variety of gates is used to control spillways. Gates range in complexity from simple slide gates (e.g., fixed-wheel gates or roller gates), to float-type gates (e.g., drum gates and ring gates), to gates which use hydrodynamic forces of flowing water to assist in actuation (e.g., radial or tainter gates). Uncontrolled releases could result from gates failing open, while overtopping and eventual loss of the entire dam could result from gates failing closed. Gate failures may be associated with hydrologic, seismic or sunny-day failure mechanisms. ISG

Sections 4, 5, and 6 discuss gate failures associated with hydrologic, seismic and sunny-day failure mechanisms, respectively.

Outlet works are typically less important because they involve smaller flows. They will not be discussed further in this guide, but the potential impact of this type of failure should be considered.

2.2.3 Operational Failures and Controlled Releases

Certain operational failures and even certain controlled releases can lead to flooding at the NPP site. These failures can occur in a variety of situations and cannot be neatly categorized as having or being part of hydrologic, seismic or sunny-day failure. They might be a compounding factor in any operational failure. Operational failures occur when equipment, instrumentation, control systems (including both hardware and software), or processes fail to perform as intended. This, in turn, can lead to uncontrolled reservoir release. Instances where controlled releases can lead to inundation at a NPP site include, but are not limited to: a) releases performed in order to prevent dam failure during flood conditions; b) releases performed to rapidly draw down a reservoir to prevent incipient failure after a seismic event; and c) releases performed to rapidly draw down a reservoir to prevent incipient sunny-day failure. ISG Section 4.2.7 further discusses operational failures and controlled releases.

2.3 Multiple Dam Failures

At some NPP sites, the potential might exist for flooding caused by essentially simultaneous failure of multiple dams or the domino failure of a series of dams. For example, the site might be located in a region in which dams are located close enough to one another that a single storm or seismic event can cause multiple failures. Failure of a critically located dam storing a large volume of water may produce a flood wave that triggers domino-type failures of downstream dams. ISG Section 4.2.5 provides additional detail on multiple dam failure due to hydrometeorological phenomena. Section 5 provides additional detail on multiple dam failure due to a seismic event.

3. SCREENING AND SIMPLIFIED MODELING APPROACHES FOR WATERSHEDS WITH MANY DAMS

Section 1.3.1 and Figure 1 provided an overview of “screening” approaches intended to reduce the analysis burden for watersheds with many dams. This section discusses this issue in detail, including the criteria used to identify those dams that may be removed and not given further consideration in the analysis (i.e., “inconsequential” dams) and conservative screening approaches based on simplified empirical and mechanistic methods. These screening approaches are intended to reduce the amount of effort required to show that failure of certain upstream dams does not result in water levels above the flood protection level of structures, systems, and components (SSCs) important to safety, or plant grade, if appropriate (i.e., the approaches screen out “noncritical” dams). The guidance in this section may be applied to both single dams and groups of dams. All other dams should be considered “potentially critical” dams and subjected to further evaluation. Refer to Figure 11.

These screening methods are intended to be used with publicly available information. Online information on thousands of dams is available from the U.S. Army Corps of Engineers (USACE) National Inventory of Dams (NID; http://nid.usace.army.mil). Other Federal agencies, as well as State and local government agencies may also maintain information on dams that they own or regulate. Data used to delineate and describe watersheds (e.g., topographic maps, digital elevation datasets, watershed boundaries) are available from the U.S. Geological Survey (USGS) (at http://ned.usgs.gov, and http://nhd.usgs.gov/wbd.html).

A justification for using simplified methods should be developed on a site-specific basis and included in the flood hazard reevaluation report. Note that other methods can be used and will be reviewed on a case-by-case basis.

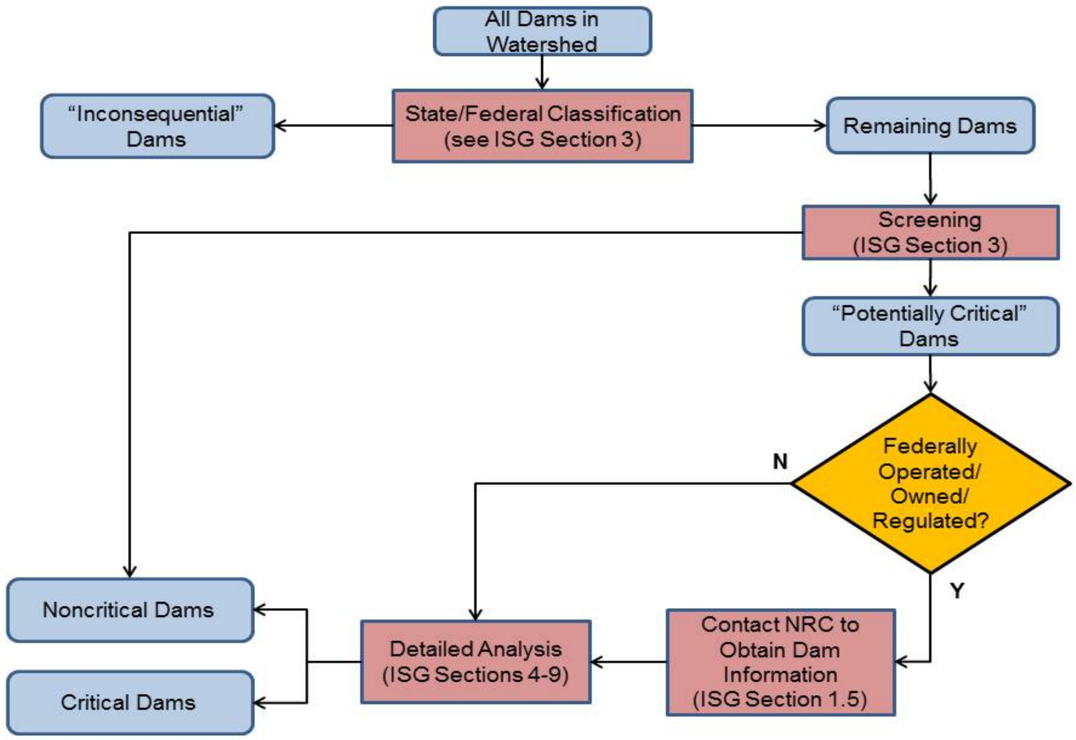

Figure 11. Screening approach for watersheds with many dams.

3.1 Criteria for "Inconsequential" Dams

Those dams identified by the USACE as meeting the requirements described in Appendix H, "Dams Exempt from Portfolio Management Process," to ER 1110-2-1156, "Safety of Dams – Policy and Procedures," (USACE, 2011c) may be removed from consideration for site impacts. The USACE states that there is "essentially no concern with their possible failure, and thus, expenditure of scarce dam safety resources thereon is to be minimized. Non-routine management will generally take place…" Additionally those dams that upon failure would only cause damage to the property of the dam owner may be removed. In some cases, dams in this category have been identified by State dam safety programs. For example, the State of Colorado identifies such dams as "No Public Hazard" (NPH), while the Commonwealth of Virginia uses the term "low hazard with special criteria." These dams are referred to as "inconsequential dams" in this ISG. Removal of dams from the Recommendation 2.1 reevaluation based only on damage being limited to the owner's property does not apply to licensee-owned dams (or onsite water-control structures). In this situation, additional analysis would be needed to justify that the dam or water-control structure meets the intent of the "inconsequential" category and may be removed from further consideration.

Staff Positions:

- Dams identified by Federal or State agencies as having minimal or no adverse failure consequences beyond the owner's property may be removed from further consideration in the Recommendation 2.1 reevaluation. Dams owned by licensees may not be removed. Other inconsequential dams may be removed with appropriate justification (e.g., if they can be easily shown to have minimal or no adverse downstream failure consequences).

- Continued consideration should be given to the failure consequences for clusters of dams that individually meet the above criteria if engineering judgment indicates that their collective failure would exceed the removal criteria.

3.2 Simplified Modeling Approaches

Several optional methods discussed below provide a quantitative basis for simplified modeling of upstream dams. The methods are presented in a gradation of conservatism that is considered hierarchical-hazard-assessment (HHA) (see NUREG/CR-7046), and are applicable to all initiating events (hydrologic, seismic, and sunny-day).

SSCs important to safety located below site grade must also be confirmed to have flood protection to the elevation of the site in order to apply the screening methods. If SSCs important to safety do not have this level of flood protection, then replace “site grade” in the screening discussion with the lowest flood-protection elevation of SSCs important to safety.

The following methods may be applied sequentially in a HHA-type gradation of conservatism. Alternatively, a single method or a subset of the methods may be applied, as appropriate. The methods are described below and illustrated in Figure 12 through Figure 15.

1. Volume Method: This calculation is representative of having the total upstream storage volume simultaneously transferred to the site without attenuation. The following steps illustrate the method (see also Figure 12):
    a. Estimate and sum the storage volume for all upstream dams (“inconsequential” dams may be excluded) in the watershed, assuming pool levels are at levels corresponding to the maximum storage volume (i.e. corresponding to the top of the dam).
    b. The 500-year flood is used to capture antecedent flood conditions at a NPP site. Current information on 500-year water surface elevations may be used, if available. Existing stage-discharge functions or USGS streamflow rating curves may also be used to estimate the flood stage at the site corresponding to the 500-year return period. If neither estimates of 500-year water surface elevations nor stage-discharge functions exist, then they may be developed using appropriate methods (e.g., using hydrologic and hydraulic models).
    c. Using available topographic data (e.g., LiDAR datasets or USGS digital elevation models), develop the stage-storage function at the site. The lowest stage should correspond to the 500-year flood elevation estimated in step (b). The stage-storage function should exclude remote floodplain storage areas that could not be accessed by overbank floodwaters. Compute the flood elevation at the site by applying the total storage volume for all upstream dams (step a) to the stage-storage function.
    d. If the resulting water surface elevation is above the flood protection level of SSCs important to safety (or plant grade, if appropriate), iteratively repeat the process, removing volumes from largest dams, to segregate potentially critical dams from dams with small cumulative effect of failure at the site (small in the sense that detailed modeling is not required to conservatively account for their effect). The dams that are removed are “potentially critical” and should be evaluated separately, using refined methods. The cumulative effect of the “noncritical” dams will be carried forward and eventually added to refined estimates for the critical dams.

2. Peak Outflow without Attenuation Method: This method is based on summing estimated discharges from simultaneous failures of upstream dams arriving at the site without attenuation. The following steps illustrate the method (see also Figure 13):

    a. Use applicable regression equations for estimating the peak breach outflow. For those equations that use water level behind the dam at time of failure, assume pool levels corresponding to the maximum storage volume (i.e. corresponding to the top of the dam). Because of the potentially large number of dams at this stage of the analysis, justification of applicability for individual dams will not be practical. Therefore, use of demonstrated conservative regression relations such as those developed by the Bureau of Reclamation (USBR, 1982) is recommended.

    b. Sum the peak failure outflows for all upstream dams (i.e., assume flows from all of the upstream dams reach the site simultaneously, ignoring attenuation). As in step 1.a, "inconsequential" dams may be excluded.

    c. Using an existing stage-discharge function (e.g., from available hydraulic models of the watershed or USGS streamflow rating curves), estimate the flood stage at a NPP site corresponding to the 500 year return period. If stage-discharge functions do not exist, they may be developed using appropriate methods.

    d. Using the stage-discharge function developed in step (c), estimate the flood stage corresponding to the peak failure outflow sum (step b), using the 500-year flood elevation estimated in step (c) as the initial stage. Compare the estimated flood stage to the flood protection level of SSCs important to safety (or plant grade, if appropriate).

    e. If the resulting water surface elevation is above the flood protection level of SSCs important to safety (or plant grade, if appropriate), iteratively repeat the process, removing peak flow rates from largest dams, to segregate potentially critical dams from dams with small cumulative effect of failure at the site (see step 1d). The dams that are removed are "potentially critical" and should be evaluated separately, using refined methods. The cumulative effect of the "noncritical" dams will be carried forward and eventually added to refined estimates for the critical dams.

3. Peak Outflow with Attenuation Method: This method is based on summing estimated discharges from simultaneous failures of upstream dams arriving at the site with attenuation (i.e., using Method 2 with attenuation). The following steps illustrate the method (see also Figure 14):

    a. Same as Method 2, Step (a).

    b. Sum the peak failure outflows for all upstream dams (i.e., assume flows from all of the upstream dams reach the site simultaneously, taking into account attenuation based on distance). As in step 1.a, "inconsequential" dams may be excluded. The distance from the dam(s) to the site can be determined using GIS tools. Either the distance from the dam(s) through the river network to the site or the straight-line distance from the dam(s) to the site (more conservative) may be used. Regression equations for attenuation provided in USBR (1982) may be used, but should be tested against available models and/or studies to justify their applicability to the river/floodplain system.

c. Same as Method 2, Step (c).

d. Same as Method 2, Step (d).

e. Same as Method 2, Step (e).

4. Hydrologic Model Method (see Figure 15): Use an available rainfall-runoff-routing software package (e.g., HEC-HMS) to assess dam failure scenarios. The advantage to this approach is a more realistic representation of the effects of multiple upstream dam failures and attenuation to the site. The use of simplified hydrologic routing must be justified and shown to be appropriate for use (Section 9). Additionally, this method requires additional basin-specific inputs (e.g., watershed topography, roughness, unit hydrographs, and antecedent conditions), as well as dam breach parameters. Appropriate justification for these inputs should be provided.

For watersheds with many dams, setting up a single hypothetical dam to conservatively represent multiple dams in a rainfall-runoff-routing model involves much less effort than modeling actual dams. The hypothetical dam(s) should include representative situations of dams in series and cascading failures (see example illustration in Figure 16). The hypothetical dams should conserve the impounded volume of the dams they represent. The stage-storage relationship of the hypothetical dam should be based on the topography of its chosen location. As in Method 2, use available topographic data (e.g., LiDAR datasets or USGS digital elevation models). See Section 3.2.1 for additional detail on dam clustering and hypothetical dams.

Compare the estimated flood stage to the flood protection level of SSCs important to safety (or plant grade, if appropriate). As with Methods 1 through 3, it might be necessary to iteratively remove dams (hypothetical or real), larger to smaller, to the point at which the resultant water surface elevation is below the flood protection level of SSCs important to safety (or plant grade, if appropriate). The dams that are removed are “potentially critical” and should be evaluated separately, using refined methods. The cumulative effect of the “noncritical” dams will be carried forward and eventually added to refined estimates for the critical dams.

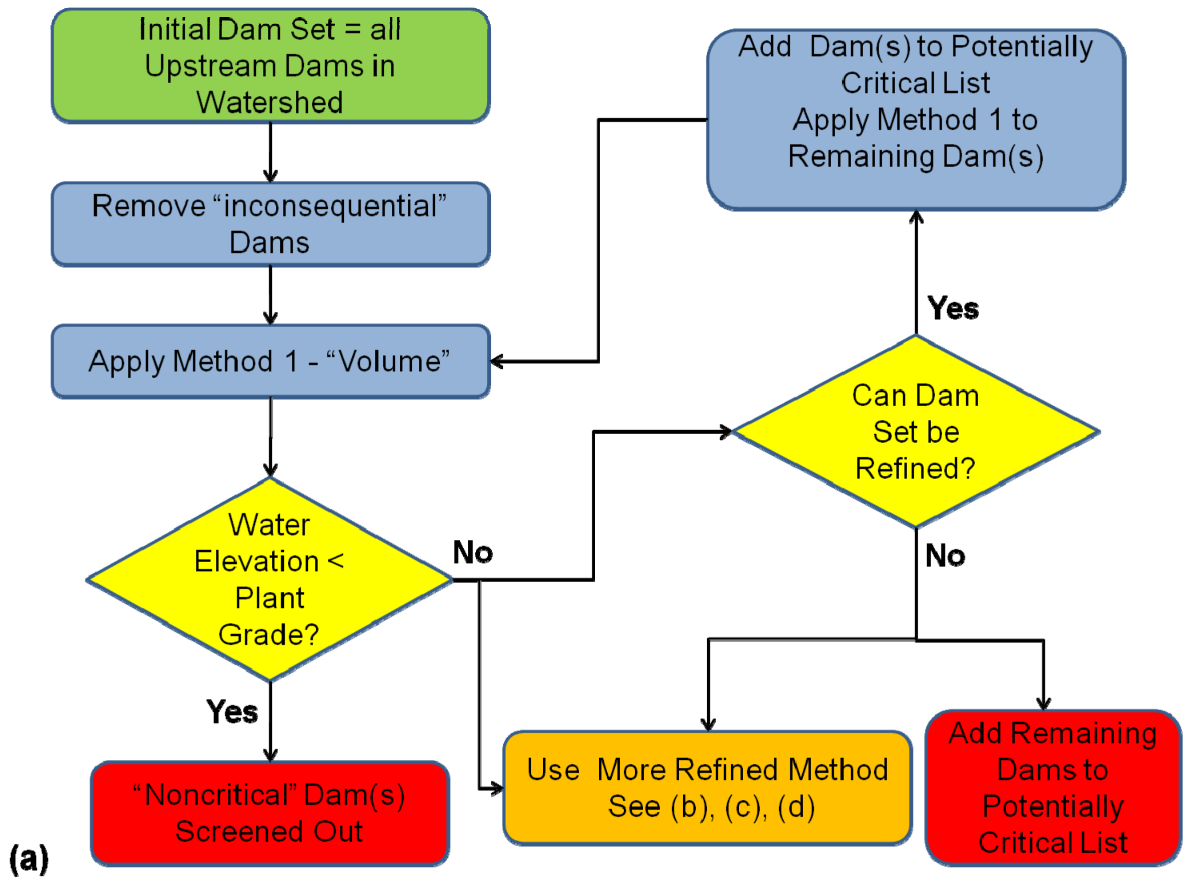

Figure 12. Screening Method Flowchart (a) – Method 1 (Volume)

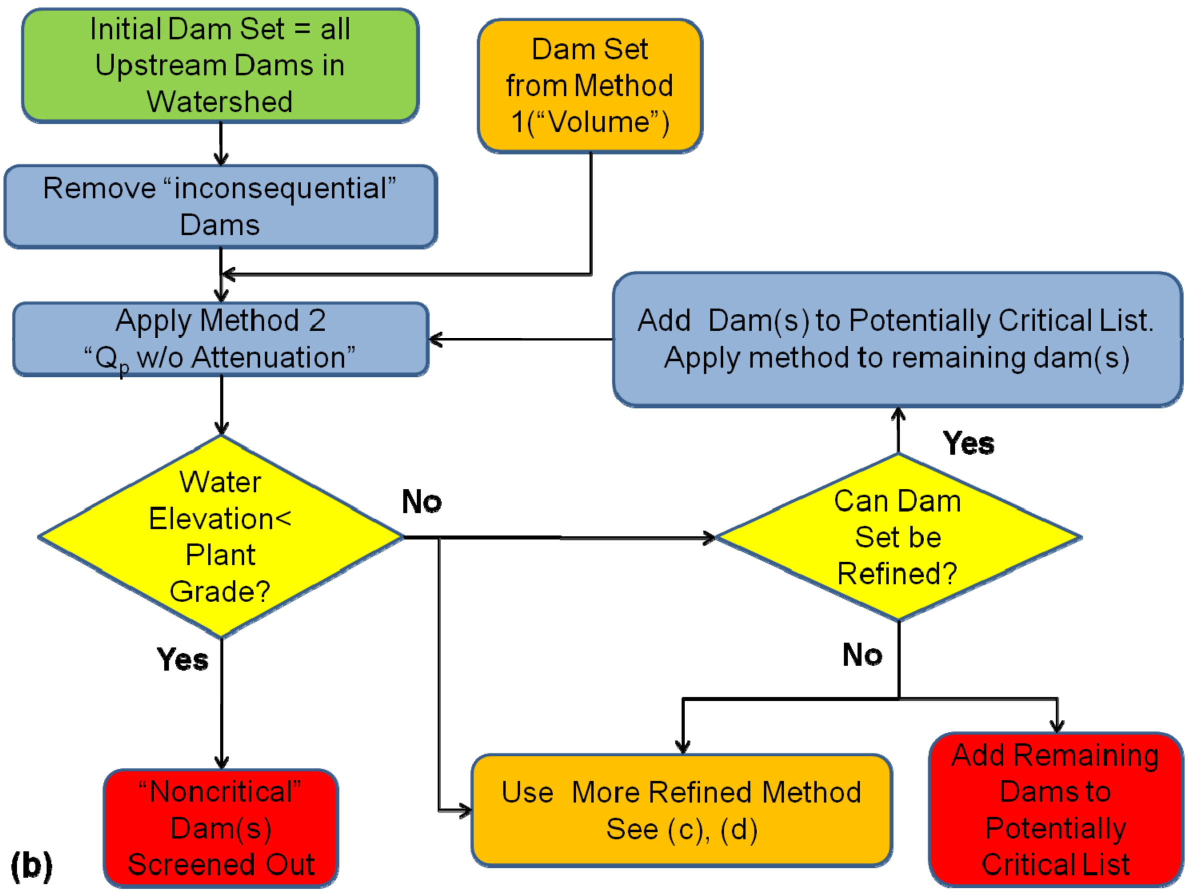

Figure 13. Screening Method Flowchart (b) – Method 2 (Peak Flow without Attenuation)

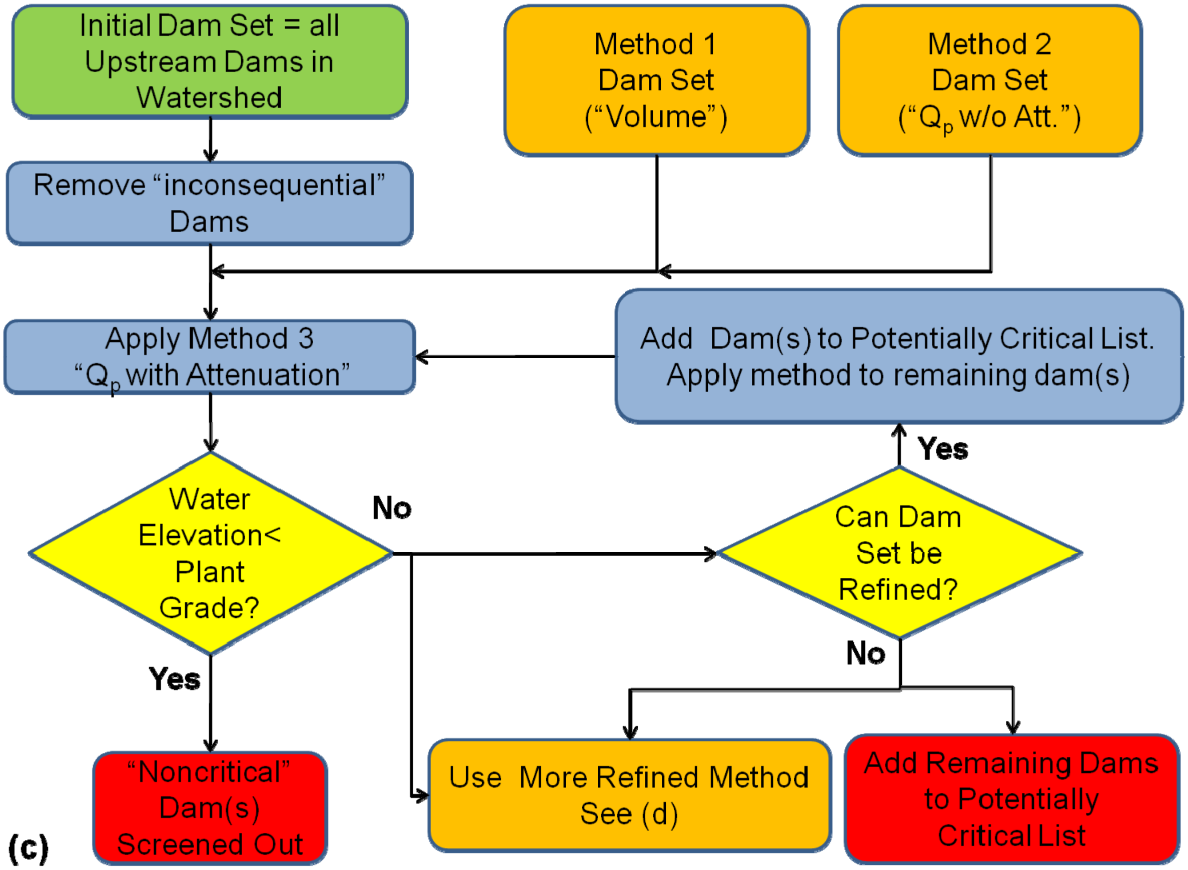

Figure 14. Screening Method Flowchart (c) – Method 3 (Peak Flow with Attenuation)

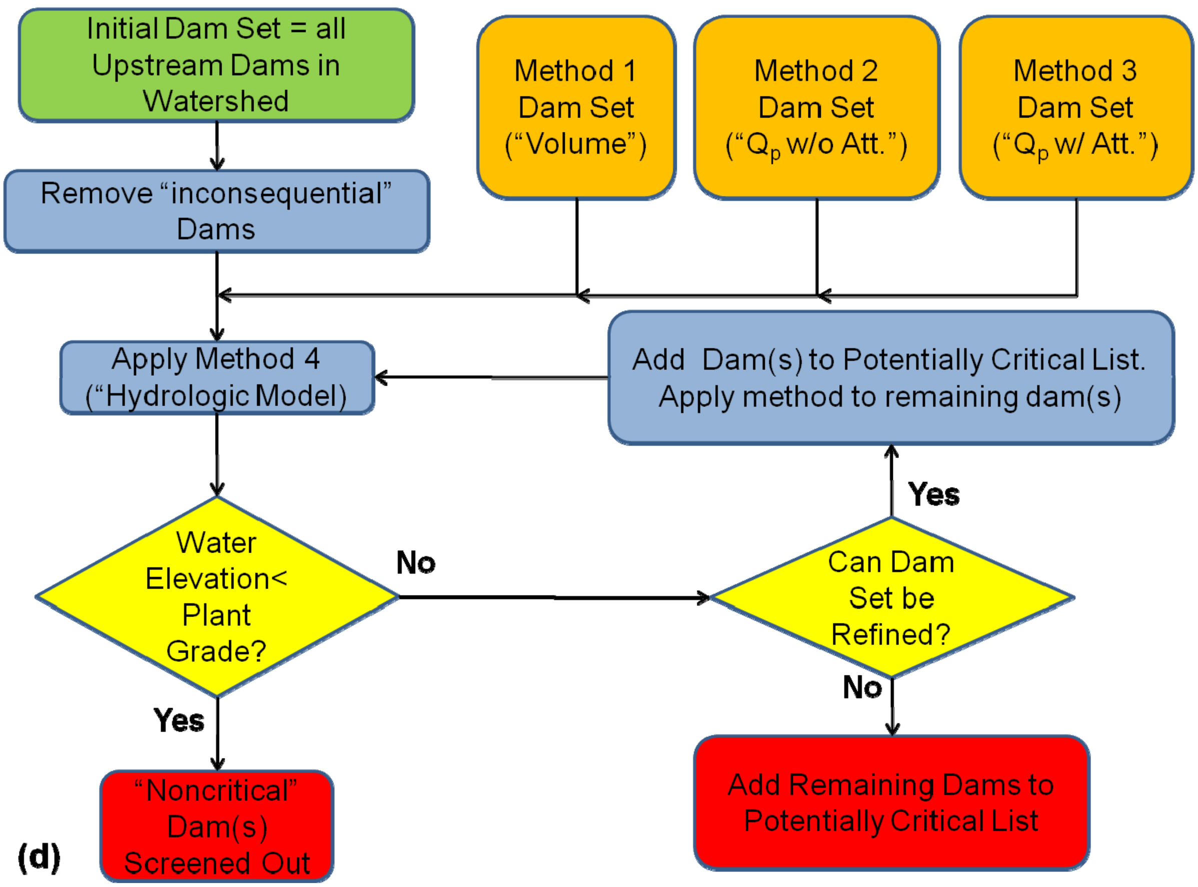

Figure 15. Screening Method Flowchart (d) – Method 4 (Hydrologic Method)

3.2.1 Representing Clusters of Dams

To reduce the level of effort necessary to evaluate the flood levels occurring due to a dam breach, dams may be grouped together or clustered and represented as a larger hypothetical dam. The volume of this larger hypothetical dam would be the cumulative volume of the real dams it is intended to represent. The location of this hypothetical dam must be at either the location of the most downstream (DS) dam in the cluster or even further downstream toward the site (see Figure 16). Note that clustering of dams into fictitious configurations is only to be used in screening. It is not appropriate to apply this technique in detailed analyses.

Topographic information from LiDAR or a digital elevation map (DEM) at the location of the hypothetical dam is used to develop a stage-storage function for the hypothetical dam. This stage-storage function is used to determine the water surface elevation of the hypothetical dam.

As an alternative, if topographic information is not used to develop a stage-storage curve for the hypothetical dam, the stage-storage curve may be derived by summing the storage curves of the individual dams. The height of a hypothetical dam developed in this manner would be equal to the height of the tallest actual individual dam with a maximum storage equal to the summed storage of the individual dams. The invert elevation of the hypothetical dam would be derived from the topographic information.

While choosing which dams to cluster and where to place the hypothetical dam representing the actual dams, one must keep in mind that the clustering must make hydrologic sense. For example in Figure 16, the following dams may be represented or clustered according to Table 2.

Table 2. Possible Dam Clustering Combinations

Dam	Location of hypothetical dam	Comment
Dams 1, 2, and 3	DS of 1, 2, and 3	Illustrated in Figure 15
Dams 1 and 2	At or DS of 2	Dam 2 is closer to the site
Dams 1 and 3	At or DS of 3	Dam 3 is closer to the site
Dams 2 and 3	At or DS of 2	Dam 2 is closer to the site

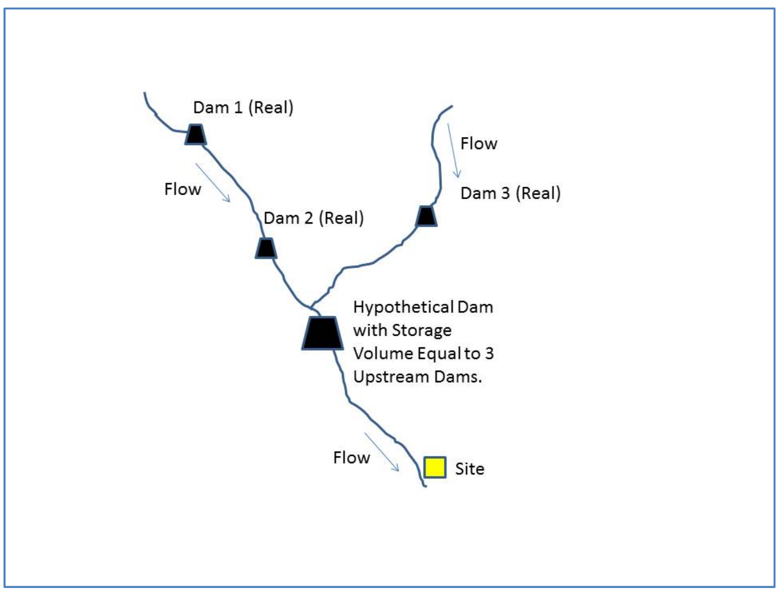

Figure 16. Hypothetical Dam Representing Storage Upstream

4. HYDROLOGIC DAM FAILURE

Hydrologic dam failures can be induced by extreme rainfall or snowmelt events that can lead to natural floods of variable magnitude. The main causes of hydrologic dam failure include overtopping, structural overstressing, and surface erosion due to high velocity flow and wave action. Section 4.1 provides an overview of hydrologic failure by dam type. Section 4.2 provides more detail on analysis of various hydrologic failure modes.

4.1 Hydrologic Failure by Structure Type

4.1.1 Concrete Dams

Concrete dams are generally perceived to be relatively resistant to overtopping failure. Nonoverflow sections of concrete dams (i.e. sections not designed to be overtopped) are typically able to withstand some overtopping due to the inherent structural properties of their concrete. However, the foundation or abutments may be susceptible to significant erosion during overtopping flows (e.g. due to weak/fractured rock, or erodible soils), and if foundation or abutment support is lost due to overtopping erosion, the dam could fail. Other portions of the concrete dam or appurtenances may be vulnerable to flood-induced hydrologic loading. Examples include, but are not limited to: (a) erosion of an unlined tunnel or spillway chute; (b) erosion of a channel downstream from a stilling basin due to flow in excess of capacity; and (c) erosion of the spillway foundation where floor slabs have been damaged or lost; and (d) cavitation damage to lined tunnels or spillway chutes.

Overstressing of a dam may occur under flood conditions. As the reservoir rises during flood loading, there may be a level at which the heel of the dam goes into tension (based on effective stress), in which case the potential for cracking along a lift joint at that elevation may increase. At some point, the estimated tensile strength of the concrete may be exceeded, leading to failure of the dam.

Overstressing of an abutment may be a concern for concrete arch dams. An abutment foundation block on which the dam rests could become unstable under increased loading due to flood conditions. The increase in reservoir level not only affects the dam loads on the block, but also the hydraulic forces on the block bounding planes (joints, faults, shears, bedding plane partings, foliation planes, etc.).

Staff Position:

Concrete dams should be evaluated for potential hydrologic failure modes including, but not limited to:

- overtopping of the main dam, and overtopping erosion of a dam abutment or foundation
- erosion of an unlined tunnel or spillway chute
- erosion of a channel downstream from a stilling basin due to flow in excess of capacity
- erosion of the spillway foundation where floor slabs have been damaged or lost
- overstressing of the dam, foundation, or abutments
- cavitation damage to spillway and outlet flow surfaces

4.1.2 Embankment Dams

Hydrologic loadings on embankments associated with flooding mainly fall into two categories: (a) increased internal seepage pressures; and (b) overtopping which initiates embankment erosion. Overtopping may be due to stillwater elevation alone, or in combination with wave action. Overtopping may also be due to failure of gates, outlet works, or other appurtenances. Deterioration or plugging of drains may lead to increased internal seepage pressures.

Staff Position:

Embankment dams should be analyzed for conditions leading to, and the effects of:

- overtopping
- increases in internal seepage pressures

4.1.3 Spillways, Gates, Outlet Works, and Other Appurtenances

There are a number of dam features, not unique to any one particular dam type, for which loss of function during flooding events could directly cause uncontrolled release of the reservoir or lead to uncontrolled release of the reservoir because of overtopping, erosion, or some combination of these. Chief among these are spillways, gates and other outlet works. Sections 4.2.5 and 4.2.6 further discuss treatment of spillways and gates, respectively (as well as other outlet works), in the analysis of hydrologic failures.

Staff Position:

- Analysis of hydrologic failure modes should consider the potential for loss or degraded function of spillways, gates, outlet works, and other appurtenances. If failure is not assumed, provide an engineering justification.

4.1.4 Levees

Levees that provide flood protection to a NPP site should be evaluated. Should they be overtopped in the event of a flood, it can be assumed that the levee has failed. Stability of the levee when not overtopped by a flood should be handled on a case-by-case basis. Distant levees are generally not of great concern.

Overtopping can lead to significant landside erosion of the levee or even be the mechanism for a complete breach. Often earthen embankment levees are armored or reinforced with rocks or concrete to minimize erosion and prevent failure. In the riverine context, levee overtopping will initiate when floodwaters exceed the lowest crest of the levee system. Wind waves and setup may contribute to the overtopping. In the coastal context, overtopping from the seaward side is most often caused by sustained high water levels and waves, due to a combination of storm surges and tide (and potentially tsunamis). Overtopping can also occur from the landward side if the water level is raised under extreme precipitation in the basin.

Except for so-called “frequently loaded” levees, levees are generally not designed to withstand high water levels for long periods. A frequently loaded levee is one that experiences a water surface elevation of one foot or higher above the elevation of the landside levee toe at least once a day for more than 36 days per year on average (CADWR, 2012). Frequently loaded levees are generally designed to earthen dam standards.

4.2 Analysis of Hydrologic Failure Modes

Overtopping is the most widely recognized hydrologic failure mode. Other common modes include overstressing of the dam or its abutments caused by hydrologic loads, erosion of embankments due to wave action, and erosion or cavitation in spillways. The next several subsections discuss in more detail the analysis of these and potential other failure modes associated with flooding, as well as the potential failure of multiple dams as a result of a single storm event.

4.2.1 Internal Pressure

Estimating internal seepage pressures associated with various reservoir levels is an essential element of embankment dam design. However, deterioration or plugging of drains, as well as internal erosion mechanisms, can lead to increased internal pressures and seepage. These conditions can compromise the structural integrity of the dam.

Staff Position:

Embankment dams should be evaluated for potential failures due to internal pressures from a large hydrologic inflow event (flood). Potential failure modes that should be evaluated include deterioration or plugging of drains and internal erosion mechanisms. Evaluation should generally include reviewing the dam design to assure that appropriate filters, drains, and monitoring points are included. Monitoring records from piezometers, observation wells or other observation methods can be used to infer an absence of deficiencies.

4.2.2 Overtopping

Overtopping occurs when the water surface elevation in the reservoir exceeds the height of the dam, allowing it to flow over the crest of the dam, an abutment, or a low point in the reservoir rim.

During a severe overtopping event, the foundation and abutments of concrete dams may also be eroded, leading to a loss of support and failure from sliding or overturning (FEMA, 2004a). Overtopping of a dam because of flooding, leading to erosion and breach of the embankment, is the most common failure mode for embankment dams. The details of breach modeling are discussed in Section 7.

Dams are typically designed to accommodate the so-called inflow design flood (IDF). In many cases, the IDF is the Probable Maximum Flood (PMF) developed by analyzing the impacts of the Probable Maximum Precipitation (PMP) event over the dam's upstream watershed. In some cases, a lesser flood is considered. Inadequacy of the dam/spillway system and reservoir storage capacity to handle the inflow design flood is the most common cause of overtopping (inflow design flood estimates often change over time as more data is acquired or changes occur in the watershed). An overtopping failure may also occur when a reservoir's outlet system is not functioning properly, thereby raising the water surface elevation of the reservoir.

Staff Position:

- Dams unable to pass their individual PMF should be considered for failure.
    - Embankment dams should generally be assumed to fail when overtopped. If failure is not assumed when a dam is overtopped, justification should include a detailed engineering analysis supported by site-specific information, including material properties of the embankment and foundation soils, material properties of embankment protection (if any), dam condition, etc.
    - Concrete dams are not assumed to fail due to minor overtopping, but should be evaluated for failure due to loss of foundation or abutment support. Impact of the flood flows on structures such as tunnels, spillways, chutes, and stilling basins should be examined.

- The potential for overtopping due to nonfunctioning gates, outlets and other appurtenances should be evaluated to determine what the appropriate failure assumptions are with appropriate engineering justification.
- Onsite or offsite temporary structures can continue to be credited in the Recommendation 2.1 flood hazard reevaluation if such credit has been evaluated and accepted by the NRC staff prior to the request for information letter (NRC, 2012). No other temporary structures or measures (including mitigation or compensatory measures) should be credited in the flood hazard reevaluation. Temporary structures or measures not credited in the hazard reevaluation may be proposed as interim actions and discussed in the appropriate section(s) of the hazard reevaluation response as described in the request for information letter (NRC, 2012).

4.2.2.1 Reservoir Capacity

The reservoir capacity will influence the maximum water surface elevation as well as the rate of change in elevation during floods. Consideration should be given to the potential for reductions in reservoir capacity over the life of nuclear power plant. The most common reason for reservoir capacity reduction is sedimentation. Other potential mechanisms, although much less likely, include mud or debris flows (e.g., from fire-impacted watersheds), failure of upstream coal-ash and mine-tailings impoundments.

Staff Position:

Consideration should be given to the potential for reductions in reservoir capacity due to sedimentation over the life of Nuclear Power Plant (NPP). Records from periodic bathymetric surveys of the reservoir, records of sediment production in upstream reaches, or estimates for sediment production rates for the upstream watershed can be used to support modeling assumptions.

4.2.2.2 Starting Reservoir Elevation

The starting reservoir water surface elevation at the beginning of a flood can impact the maximum reservoir water surface elevation, and thus the potential for overtopping. A lower starting reservoir water surface elevation can lower the maximum water surface achieved in flood routings due to the additional surcharge space within the reservoir. Some reservoirs are operated to provide more surcharge storage during flood season.

Staff Position:

In view of the uncertainties involved in estimating reservoir levels that might reasonably be expected to prevail at the beginning of a flooding event, the default starting water surface elevation used in flood routings for evaluation of overtopping should be the maximum normal pool elevation (i.e., the top of the active storage pool). Other starting water surface elevations may be used, with appropriate justification. Justification should be based on the operating rules and operating history of the reservoir. For example, if the flood being considered is associated with a distinct season and the operation of the dam has seasonal variations that are codified and have historically been followed, it may be reasonable to select a starting reservoir elevation consistent with the operating rules and history. The operating history used should be of sufficient length to support any conclusions (e.g., 20 years or more). However, consideration should be given to possible instances in which the operating history or rules have been influenced by anomalous conditions such as drought.

4.2.2.3 Reservoir Surcharge Capacity

Reservoir surcharge capacity can affect the ability to pass large floods at a dam. Reservoir surcharge space can be used to store a portion of the incoming flood and in combination with the spillway capacity, can attenuate the peak of the flood (the peak outflow released through the spillway may be significantly less than the peak flood inflow). The amount of the peak inflow attenuation is a function of the reservoir surcharge volume in comparison to the flood volume, in addition to the spillway type and capacity. If the reservoir surcharge volume is large in comparison to the flood volume, significant attenuation will occur.

Staff Position:

> Reservoir surcharge capacity can be credited in flood routings for evaluation of overtopping, with appropriate justification and documentation.

4.2.2.4 Spillway Discharge Capacity

Spillway discharge capacity is one of the most significant factors in the ability of a dam to pass floods. Spillway discharge capacity is usually the critical component in passing large floods, but in some cases, release capacity through other waterways (outlets, turbines, etc.) may be significant and will contribute to the total available release capacity. The term “spillway discharge capacity” as used in this document is intended to include spillway discharge capacity and any additional release capacity that would be available through other release structures at the dam.

In general, if the spillway discharge capacity is roughly equal to the peak inflow from large floods (approaching the PMF), dam overtopping is usually not an issue. If the spillway discharge capacity is significantly less than the peak inflow of a large flood, and if the volume of the flood is large in comparison to the surcharge capacity of the reservoir, dam overtopping could occur. The likelihood of these floods and erodibility of the dam or foundation materials controls the risk.

With regard to crediting release capacity through appurtenances other than the spillway (e.g., outlets or turbines), existing Federal guidance is not consistent. For example, USACE engineering manual EM 1110-2-1603, “Hydraulic Design of Spillways,” states that a powerhouse should not be considered as a reliable discharge facility when considering the safe conveyance of the spillway. Conversely, FERC Engineering Guidelines for the Evaluation of Hydropower Projects states that those release facilities that can be expected to operate reliably under the assumed flood condition can be credited for flood routing. USBR best practice guidelines (USBR, 2011) suggest that at least one turbine should always be assumed to be down (e.g., for maintenance or other reasons) in performing flood routing.

Operational history on generating unit outages (e.g., maintenance, planned, and forced outages) can be used to inform assumptions about release capacity through turbines. The North American Electric Reliability Corporation (NERC) provides reports on generating unit availability for North America (e.g., NERC, 2012), which can be used if site-specific information is not available. Site-specific records from past flooding events, if available, should be reviewed.

Staff Positions:

- Release capacity through appurtenances other than the spillway (e.g., outlets and turbines) may be credited as part of the total available release capacity, with appropriate engineering justification that these appurtenances will be available and remain operational during a flood event. Access to the site during a flood event should be considered.

- The generators and transmission facilities to support the credited turbine(s) must be shown to be operational under concurrent flood and expected prevailing weather conditions if the turbines are credited as part of the total available release capacity.

<u>Potential for Reservoir Debris to Block Spillway</u>

Watershed runoff following a major storm event typically includes a large amount of debris and this debris has the potential to block spillway bays. Figure 17 shows debris build up at Lake Lynn Dam on the Cheat River (West Virginia) during a large flood in 1985. The spillway capacity was reduced by approximately 35% from the theoretical flow and thirteen out of twenty-six Tainter gates were almost fully blocked (Schadinger et al., 2012).

Figure 17. Debris Upstream of Lake Lynn Dam after 1985 Flood Event (Schadinger et al., 2012)

Many dams have debris management facilities (e.g., trash booms or trash gates) and programs. Sturdy trash booms may be able to capture debris before it reaches the spillway, but if not, the debris may clog the spillway opening. Trash gates may be used to route debris way from spillway structures and pass it downstream.

Historical information for debris production on the watershed (or similar watersheds) can be used to gauge the potential for debris blockage. Periodic debris studies are often performed by dam owners, dam regulators, or river basin commissions.

<u>Staff Position:</u>

- The potential for flood-borne debris to reduce spillway capacity should be considered. Historical information on debris production in the watershed or similar watersheds should be used to assess the potential debris volumes.

- For dams that have debris management, a sensitivity study assuming a 5 to 10% reduction in capacity should be performed. Describe structures, equipment, and procedures used to prevent spillway blockage by waterborne debris.
- For dams that lack debris management, greater capacity reductions should be considered. The appropriate capacity reduction will vary on a case-by-case basis. Justification for the reduction used should be provided (e.g., debris studies for the watershed or similar watersheds).

4.2.2.5 Wave Action

In addition to stillwater levels associated with flood flows, wind-generated wave action may lead to overtopping of a dam. In extreme circumstances, overtopping of the dam solely due to wave action could initiate erosion of the embankment and ultimately breach the dam. Part of the evaluation should be to determine the potential for waves to exceed the dam freeboard (based on the prevailing wind direction, the wind speeds and the fetch of the reservoir).

Parapet walls are sometimes employed to contain waves that might overtop the dam and may need to be evaluated for a sustained water load. If a parapet wall is constructed on the dam crest across the entire length of the dam, dam overtopping will initiate when the reservoir water surface exceeds the elevation of the top of the parapet wall. If a parapet wall overtops, the impinging jet from overtopping flows may erode the dam crest and undermine the parapet wall. If the parapet wall or a section of the wall fails, the depth of flows overtopping the dam crest could be significant and the embankment will likely erode rapidly.

Staff Position:

> Overtopping due to wave action should be evaluated, in addition to stillwater levels. Coincident wind waves should be estimated at the dam site based on the longest fetch length and a sustained 2-year wind speed and added to the stillwater elevation.

4.2.3 Structural Overstressing of Dam Components

Higher loading conditions are typically found in dams where the reservoir elevation is increased due to a hydrologic event. While the dam itself may not be overtopped, the surcharge may be increased, overstressing the dam's structural components. This overstressing may then result in an overturning failure, sliding failure, or failure of specific components of the dam.

Embankment dams may be at risk when increased water surface elevations produce increased pore pressures and seepage rates that exceed the design seepage control measures for the dam. Concrete dams may be at risk due to potential failure of specific components of the dam, such as overturning or slipping of a slab section (FEMA, 2004a).

Staff Position:

> Static stability of the dam and key appurtenances under hydrologic loads associated with the dam's PMF should be demonstrated using current methods and standards. If the dam cannot withstand the applied loads, the dam should be assumed to fail. If the appurtenance cannot withstand the load, assume failure of the appurtenance and estimate the impact of its failure on stability of the dam. If the dam stability is not impacted, one still should consider the downstream impact of uncontrolled release (if any) associated with appurtenance failure.

4.2.4 Surface Erosion from High Flow Velocity and Wave Action

Surface erosion can occur along earthen spillways, the upstream or downstream embankment slopes, or along other inlet and outlet channels of appurtenant structures. Surface erosion is primarily caused by high velocity runoff, reservoir wave action, and ice action. High flow velocities may cause headcutting along spillway sides that can progress towards the spillway crest, eventually leading to a full dam breach (FEMA, 2004a).

Staff Position:

> Surface erosion of earthen embankments, spillways, channels, etc. due to wave action, high velocity flows, and ice effects should be considered.

4.2.5 Failure of Spillways

Concrete-lined spillways, as well as unlined or grass-lined earthen spillways and unlined spillways excavated through rock, are subject to processes that may lead to failure during high flow events such as flooding.

Concrete-lined spillways are subject to stagnation pressure related failures that occur because of water flowing into cracks and joints during spillway releases. If water entering a joint or a crack reaches the foundation, failure can result from excessive pressure and/or flow into the foundation. If the foundation has no drainage system, or if the drainage system is inadequate, and the slab is insufficiently tied down, the buildup of hydrodynamic pressure under a concrete slab can cause hydraulic jacking. If drainage paths are available, but are not adequately filtered, erosion of foundation material is possible and structural collapse may occur.

Concrete-lined spillways are also subject to cavitation related failures. Cavitation occurs in high-velocity flow where the water pressure is reduced locally because of an irregularity in the flow surface. If the pressure drops below the vapor pressure, the water “boils” at ambient temperatures and water vapor bubbles form in the flow. As the vapor cavities move into a zone of higher pressure, they rapidly collapse as they return to the liquid state, sending out high-pressure shock waves. If the cavities collapse near a flow surface, there may be damage to the surface material. Cracks, offsets, surface irregularities and/or open joints in chute slabs and the lower portions of chute walls exposed to flow, may allow this failure mode to initiate. The geometry of the flow surface irregularities will affect the initiation of cavitation. The more abrupt the irregularity, the more prone the spillway will be to the initiation of cavitation. Once a flow surface is damaged by cavitation, the intensity of cavitation produced by the roughened surface increases, so damage can become severe in a short time.

Concrete deterioration in the form of delamination, alkali-silica reaction, freeze-thaw damage and sulfate attack can exacerbate failures related to stagnation pressure or cavitation by initiating cracks, opening cracks and joints in the chute concrete, creating offsets into the flow at joints, and causing separation of the chute from the supporting foundation.

Unlined (soil or grass-covered) spillways are subject to erosion phenomena similar to those associated with overtopping of embankments. The most common scenarios involve: (a) failure of the grass or vegetation cover in the spillway; (b) concentrated erosion that initiates a headcut; and (c) deepening and upstream advance of the headcut. The U.S. Department of Agriculture’s Agricultural Research Service (USDA/ARS) and Natural Resources Conservation Service (USDA/NRCS) have developed tools to assess erosion in earthen and vegetated auxiliary spillways of dams. The Water Resource Site Analysis Computer Program (SITES) model and the Windows Dam Analysis Modules (WinDAM) are publicly available (NRCS, 2009, 2011). Both computer programs implement similar technology for evaluating spillway integrity. They are able to indicate whether breach of a spillway due to headcutting is likely, but do not model the consequences of the breach (i.e., the simulations stop when spillway breach initiation is predicted; enlargement of the spillway breach and release of the reservoir storage are not modeled). A detailed discussion of causative mechanisms and predictive models for erosion of unlined soil or grass-covered spillways is provided in USSD (2006).

For spillways excavated in rock, the models discussed in the previous paragraph have some ability to accommodate rock-like materials through their use of the headcut erodibility index, which is defined for both soil-like and rock-like materials. Appropriate conservatism should be exercised when applying this model to a rock channel, because it was not originally developed in that environment. Another alternative for dealing with scour of rock materials is the use of a curve relating the headcut erodibility index and the required stream power to produce scour. Variations of this type of curve have been proposed by Annandale (2006), and Wibowo et al. (2005). USSD (2006) discusses in detail the causative mechanisms and predictive models for erosion of unlined spillways excavated in rock.

Staff Positions:

- Dams should be evaluated for potential failure due to spillway failure.
- Concrete spillways should be evaluated for relevant failure modes including stagnation pressure failures, cavitation, concrete deterioration (e.g., delamination, alkali-silica reaction, freeze-thaw damage, and sulfate attack) and other relevant modes.
- Other (non-concrete) spillways should be evaluated for potential failures including failure of the grass or vegetation cover in the spillway; concentrated erosion that initiates a headcut, deepening and upstream advance of the headcut, and other relevant modes.

4.2.6 Failure of Gates

A variety of gates is used to control spillways. Gates range in complexity from simple slide gates (e.g., fixed-wheel gates or roller gates) to float-type gates (e.g. drum gates and ring gates) to gates which are shaped to balance hydrostatic forces (e.g., radial or tainter gates).

Another class of spillway gate is the fuse plug, which is a collapsible dam installed on spillways to increase the dam's capacity. The principle behind the fuse plug is that the majority of water that overflows a dam's spillway can be safely dammed except during high flood conditions. The fuse plug may be a sand-filled container, a steel structure or a concrete block. Under normal flow conditions, water will spill over the fuse plug and down the spillway. In high flood conditions, where the water velocity may be so high that the dam itself may be put in danger, the fuse plug breaches, and the floodwaters safely spill over the dam.

Gates may fail to operate due to mechanical or power failures. Gates may also fail to operate when needed in flooding situations due to excessive friction or corrosion. This is more common with gates that are not properly maintained or seldom used. There is also the potential for actual gate operations to differ from planned operations (e.g., inability of an operator to access gate controls or an operator decision to delay opening the gates due to downstream flooding concerns).

Fuse plugs are generally considered reliable, but there is some inherent uncertainty about the exact depth and duration of overtopping needed to initiate their breach. There is also uncertainty about the exact rate of breach development. Understanding the magnitude of these uncertainties is important because delayed operation of the fuse plug could lead to failure of the dam.

Staff Position:

- The evaluation should consider the potential for gate failure under flooding conditions to lead to an uncontrolled release of the reservoir.
- With regard to fuse plugs, one should show that flood routings are not sensitive to the depth and duration of overtopping needed to initiate a breach so that delayed operation does not lead to the failure of the main dam.

4.2.7 Operational Failures and Controlled Releases

Certain operational failures and even certain controlled releases can lead to flooding at a NPP site. They may occur in a variety of situations, but the primary concern is that operational failures or controlled releases may be a compounding factor in flooding situations.

4.2.7.1 Operational Failures

Operational failures can occur at dams when equipment, instrumentation, control systems (including both hardware and software), or processes fail to perform as intended. This, in turn, can lead to uncontrolled reservoir release. Some illustrative examples of these types of failures include:

- Failure of a log boom allows reservoir debris to drift into and plug the spillway, leading to premature overtopping of the dam.
- Gates fail to operate as intended causing premature overtopping of the dam. This could result from mechanical or electrical failure, control-system failure, or failure of the decision process for opening the gates. Gates may also fail to operate when needed due to excessive friction or corrosion. This is more common with gates that are not maintained or used very seldom.
- Loss of access to operate key equipment during a flood leads to overtopping of the dam or other uncontrolled releases.
- Loss of release capacity leads to overtopping of the dam. For example, if releases through the power plant are a major component of the release capacity and the switchyard is taken out during a flood or earthquake, that release capacity will be lost. If the powerhouse is lower than the switchyard, loss of the powerhouse without loss of the switchyard would also result in loss of release capacity.
- Mechanical equipment failure due to changes in operation without a corresponding change in maintenance leads to premature dam overtopping. For example, if river operation requires frequent gate opening to enhance fisheries without a corresponding increase in the frequency of gate lubrication, component failure could occur when the gate is needed to pass a flood.
- Overfilling pumped-storage reservoirs can lead to overtopping and failure of the dam. This could happen due to faulty instrumentation, control system issues, or operator error.

- Failure to detect hazardous flows or a breakdown in the communication process to get people out of harm's way leads to failure of the dam's safety measures. For example, power and phone lines may be cut by a large earthquake or flood. This may result in inability to warn people in advance of life-threatening downstream flows.

An exhaustive analysis of all potential operational failures is generally not required. Instead, the intent is to understand the site-specific relationship between potential operational failures and existing safety margins (e.g., available freeboard).

Staff Position:

- Operational failures that may lead to uncontrolled releases and threaten to inundate a NPP site should be considered. Applicable operational failures should be identified, and consequences for the most likely failures should be evaluated. Operational history of similar dams, equipment, and procedures should be used to identify and rank operational failures.

4.2.7.2 Controlled Releases

There may be instances where controlled releases can lead to inundation at a NPP site. Examples include, but are not limited to: 1) releases performed in order to prevent dam failure during flood conditions; 2) releases performed to rapidly drawdown a reservoir to prevent incipient failure after a seismic event; and 3) releases performed to rapidly drawdown reservoir to prevent incipient sunny day failure.

Consideration of the potential for controlled releases to cause flooding at a NPP site may include examination of spillway and gate discharge capacities and examination of reservoir/dam operating rules and procedures. Communication plans and systems for warning downstream entities of impending release should also be considered.

Staff Position:

- The potential for controlled releases that may threaten SSCs important to safety at a NPP site should be considered.

4.2.8 Waterborne Debris

Waterborne debris (e.g., trees, logs, or other objects) produces drag and impact loads that may damage or destroy buildings, structures, or parts thereof. The magnitude of these loads is very difficult to predict, yet some reasonable allowance must be made for them in evaluating dam performance. The loads are influenced by where the structure is in the potential debris stream:

- immediately adjacent to or downstream from another building
- downstream from large floatable objects (e.g., exposed or minimally covered storage tanks)
- among closely spaced buildings

Building standard ASCE 7-10 developed by the American Society of Civil Engineers (ACSE 2010) describes a methodology for determining impact loads based on the momentum-impulse method. The methodology differs from the classic impulse-momentum approach (USACE 1995d, FEMA, 2011) in that it assumes a half-sine form for the applied load and includes several coefficients to allow design professionals to adapt the resulting force to local flood, debris, and building characteristics. The ASCE 7-10 methodology incorporates an importance coefficient to represent the risk category of the impacted structure. For critical or potentially critical dams (and for SSCs important to safety at NPP sites), an importance coefficient of 1.3, corresponding to Risk Category IV is appropriate. The ASCE 7-10 methodology also uses a depth coefficient meant to take into account the structure's location within the flood hazard zone and flood depth. For dam and NPP sites, this coefficient should be based on stillwater depth (i.e., disregard the selection criteria based on flood insurance rate map zones). ASCE 7-10 also provides a method for estimating drag loads on structures.

ACSE 7-10 provides guidance regarding the debris object weight selection for impact loads. The standard states that large woody debris with weights typically ranging from 1,000 to 2,000 lbs are appropriate for riverine floodplains in most areas of the U.S. In the Pacific Northwest, larger tree and log sizes suggest a typical 4,000 lb debris weight. Debris weights in riverine areas subject to floating ice typically range from 1,000 to 4,000 lb. ASCE considers the 1,000 lb object to represent a reasonable weight for other types of debris ranging from small ice floes, to boulders, to manmade objects. However, licensees should consider regional and/or local conditions before the final debris weight is selected.

Staff Positions:

- Drag and impact loads due to waterborne debris carried by flood waters should be considered with regard to impacts on the dam (i.e., to gates and associated mechanical equipment, appurtenances, parapets, etc.).
- In the case of dam break flood waves, debris loads on SSCs important to safety should be considered.
- The methodologies for debris load estimation described in ASCE 7-10, with the caveats described above, are acceptable to NRC staff.
- Licensees should consider regional and/or local conditions before the final debris weight is selected. On navigable waterways, for example, the potential for impact from watercraft and barges should be considered in addition to that from trees, logs and common manmade objects.

4.2.9 Multiple Dam Failure due to Single Storm Scenario

At some NPP sites, there may be potential for flooding due to multiple dam failures (e.g. dams on different reaches or tributaries above the NPP site) or the domino failure of a series of dams on the same reach. For example, the site may be located in a watershed where dams are located close enough to one another that a single storm event can cause multiple failures that have a compound effect on flood waves reaching the site. Failure of a critically located dam storing a large volume of water may produce a flood wave compounded by domino-type failures of downstream dams (e.g., failure of an upstream dam may generate a flood that would become an inflow to the reservoir impounded by a downstream dam and may cause failure by overtopping the downstream dam; if several such dams exist in a river basin, each sequence of dams within the river basin could fail in a cascade).

Staff Positions:

- Those dams unable to be removed as "inconsequential" or screened out as "noncritical" (see Section 3) remain potentially critical dams. These dams should be evaluated for hydrologic failure that leads to cascading downstream failures and/or simultaneous failures of tributary dams that ultimately produce flood conditions at the site. Operational rules may be considered but the starting water surface elevation at the most upstream dam under evaluation should be as specified in Section 4.2.2.1.

River flows should be based on the precipitation / runoff from the basin encompassing the multiple dam scenario(s) under consideration. Flood waves from multiple dam failures should be assumed to reach a NPP site simultaneously unless appropriate justification for differing flood arrival times is provided.

- Three cases of multiple dam failure should be considered: (a) failure of individual dams on separate tributaries upstream from the site, (b) cascading or domino-like failures of dams upstream from the site, and (c) a combination of cases (a) and (b).
    - In the first case, one or more dams may be located upstream from the site but on different tributaries, so the flood generated from the failure of an individual dam would not flow into the reservoir impounded by another dam. These individual dam failures should be analyzed together because of the potential for a severe storm to cause large floods on multiple tributaries.
    - In the second case, failure of an upstream dam may generate a flood that would become an inflow to the reservoir impounded by a downstream dam and may cause failure by overtopping the downstream dam. If several such dams exist in a river basin, each sequence of dams within the river basin could fail in a cascade. Each of these cascading failure sequences should be investigated to determine one or more sequences of dam failures that may generate the most severe flood at the site. Simplified estimates of the total volume of storage in each of the potential cascades should provide a good indication of the most severe combination. If multiple cascading dam failures that cannot be separated by simple hydrologic reasoning, all of the candidate cascades that are comparable in terms of their potential to generate the most severe flood at the site should be simulated using the methods described in Section 9. The most severe flood at the site resulting from these cascades should be considered in determining the design-basis flood.
- Depending on the storage capacities of the reservoirs impounded by dams in a given cascading scenario, it may be reasoned that the scenario that would release the largest volume of stored water would likely lead to the most severe flooding scenario. However, the distance a flood has to travel to reach a plant site can also affect the severity of the flood at the site. If a definite conclusion cannot be reached, all possible cascading scenarios should be simulated to determine the most severe scenario.

4.2.10 Levee Failures

Failure of levees that provide flood protection to a NPP site should be considered. Such levees should be considered to fail when overtopped. Their stability when they are not overtopped should be handled on a case-by-case basis. Distant levees are generally not of great concern.

Earthen levees (the most common type) are designed to withstand flood conditions, but typically for limited durations, discharges and water surface elevations. Under flooding conditions, pore pressures within the embankment soils may increase to the point where the embankment slopes become unstable. Slope failure and subsequent breaching may be quite sudden. Similar instability may arise in the foundation soils. Such conditions are often accompanied by levee boils, or sand boils, in which underseepage resurfaces on the landside, in the form of a volcano-like cone of sand. Boils signal a condition of incipient instability which may lead to erosion of the levee toe or foundation or sinking of the levee into the liquefied foundation below (i.e., the boils may be the result of internal erosion or piping or they may also be a symptom of generalized instability of the foundation).

Lack of inspection, maintenance, and control is often a major contributing factor in levee failure. Uncontrolled vegetation growth (especially trees) or animal burrows may be sources of local weaknesses.

Natural geomorphic processes associated with channel migration may endanger a riverine levee system. For example, the downstream end of bends and areas across from tributary inflows are areas of high-energy river flow and significant erosion may occur resulting in bank retreat and eventual levee failure. Embankments constructed across ancient riverbeds or stream channel meanders can provide weak points for seepage and pipe formation.

In some cases, levees are breached intentionally, in order to protect other areas. In most cases, an intentional breach is not initiated without significant planning and notification.

Not all levees are earthen embankment type. Concrete and sheet piles are sometimes used. Some earthen levees have sheet pile or concrete parapets.

Staff Positions:

- In general, earthen embankment levees should be assumed to fail when overtopped. The case for nonfailure should be developed using detailed engineering analysis supported by site-specific information, including material properties of the embankment and foundation soils, material properties of embankment protection (if any), levee condition, etc. Other forms of levees (e.g., pile walls and concrete flood walls) should be evaluated for potential failures applicable to the particular type of levee.
- Levees are generally not designed to withstand high water levels for long periods. However, no generally accepted method currently exists for predicting how long a levee will continue to function under high loading conditions. Therefore, historical information is the best available basis for predicting levee performance. The historical information should be from levees that have similar design and construction characteristics as the levee being analyzed.
- If the performance of levees is potentially important to estimation of inundation at a NPP site, failures should be treated in a conservative, but realistic, manner.
- If credit is taken for a specific levee behavior (either failure or nonfailure), an engineering justification should be provided.
- Crediting intentional levee breaching will generally not be accepted due to large uncertainties in implementation of such plans (e.g., decisions about such actions are often political in nature).
- Assumptions regarding conveyance and off-stream storage should be supported with engineering justifications.
- The potential for loss or degraded function of levee control works should be considered.
- Because levees are typically designed to function as a system, the potential for failure of an individual segment should be evaluated for its impact on the functioning of the levee system as a whole.

5. SEISMIC DAM FAILURE

Seismic hazard is generally defined as the physical effects that occur as the result of an earthquake (e.g., ground shaking, surface faulting, landsliding, or liquefaction). The severity of these effects depends on factors such as the intensity and spectral characteristics of ground motions, dam type, dam construction materials and methods, and local site conditions. For some dam sites, the potential for surface fault displacement through the site is a major concern, but strong ground shaking is the most common earthquake effect. Ground shaking may directly damage the dam structure and appurtenances or induce subsequent failure modes. For example, seismically induced soil liquefaction can lead to embankment failure for earthen embankment dams or foundation failure for other types of dams.

Another possibility concerns an active fault passing through the reservoir of a dam. Fault offset within the reservoir could create a seiche wave capable of overtopping and eroding the dam. Seiche waves can be generated by large fault offsets beneath the reservoir or by regional ground tilting that encompasses the entire reservoir. “Sloshing” can lead to multiple overtopping waves from these phenomena.

Note that the seismic dam failure scenario is one in which “load combinations” come into play (e.g., a more frequent earthquake combined with a flood event). This is discussed in Section 5.6. In some instances (e.g., when downstream consequences are likely to be small), the licensee may elect to simply assume that the dam fails seismically in lieu of conducting a seismic analysis. In this case, the question arises of what flood event to assume, since no frequency is assigned to the seismic event. In Section 5.6, the 500-year flood is used in conjunction with the lower of the two seismic hazard levels. Therefore, a lesser flood would not be appropriate in this case.

Staff Position:

> If seismic failure is simply assumed without analysis, the seismic failure should be assumed to occur under 500-year flood conditions (or ½ PMF, whichever is less).

5.1 Overview

A complete seismic evaluation of an existing dam typically includes: 1) an assessment of site-specific geological and seismological conditions to determine seismic potential and associated ground motions (a field and laboratory testing program may be needed to characterize the distribution and properties of the soils if they are not known); and 2) an analysis of the effects of earthquake shaking on the dam’s structure and its appurtenances.

The basic steps in analyzing the seismic failure problem are as follows:

1. Estimate earthquake ground motions:
    a. Characterize earthquake sources
    b. Apply attenuation relations to estimate bedrock motion at the site
    c. Determine site amplification function to estimate ground surface response at the site
2. Estimate the loadings imposed on the dam by the earthquake ground motions
3. Analyze the ability of the dam to withstand the earthquake-induced loadings

The behavior of dams and their foundations under earthquake loading is an extremely complex problem. It is therefore essential that seismic investigations be conducted by knowledgeable seismic engineers following the “state of the practice” in the profession.

5.1.1 Seismic Hazard Characterization

Seismic parameters represent one of several ground-motion-related variables or characteristics, such as peak ground acceleration (PGA), peak ground velocity (PGV) displacement, response spectra, acceleration time histories, or duration. They can be obtained using deterministic or probabilistic seismic hazard analysis (PSHA) procedures.

Preferably, seismic evaluation parameters should be specified using site-dependent considerations, making use of existing knowledge and actual observations that pertain to earthquake records obtained from sites with similar characteristics. In particular, attenuation characteristics should not be applied blindly due to differences in earthquake focal depths, transmission paths, and tectonic settings. It is important to indicate if seismic parameters predicted by attenuation relationships take into account the effect of surface soil layers since soft deposits can alter the bedrock motions dramatically. The duration of shaking is a significant seismic evaluation parameter, as it has been shown to be directly related to the extent of damage, especially in the case of embankment dams. This is even more critical when the foundation or the embankment contain soils that are prone to accumulate excess pore pressures during an earthquake. Local conditions may affect the expected duration of earthquake shaking and should be considered on a case-by-case basis.

Vertical peak and spectral accelerations are usually considered less critical than horizontal motions for embankment dams that are distant from the earthquake source(s). They are more important for concrete dams and concrete appurtenances. Vertical motions are sometimes estimated by scaling horizontal accelerations, along with corresponding frequencies in the case of spectral values, using factors in the range of 1/2 to 2/3. When vertical accelerations are critical, it is preferable to rely on attenuation relationships developed specifically for vertical accelerations.

While definition of seismic parameters by peak values and spectral shapes is sufficient for some dam applications, in other cases a time history analysis may be required. This will be the case when induced stresses approach the strength of the dam or foundation materials, or when it is necessary to consider the inelastic behavior of the dam.

5.1.1.1 Use of USGS National Seismic Hazard Maps

The USGS National Seismic Hazard Maps (USGS, 2008) are developed from seismic sources and ground-motion equations specific to the Central and Eastern United States earthquakes, to the Western United States crustal fault earthquakes, and to subduction-zone interface and in-slab earthquakes. In the Central and Eastern United States, the USGS generally calculates ground motions from sources that are up to 1,000 km from the site. In the Western United States, the USGS calculates ground motion from crustal sources less than 200 km and subduction sources less than 1,000 km from the site. The USGS also maintains a Web site where the maps (and the data and software used to create them) are available (http://earthquake.usgs.gov/hazards/?source=sitenav).

5.1.2 Structural Considerations

It is important for the engineers who will be doing the fragility evaluation to coordinate with the seismologist on generating the hazard curves, uniform hazard spectra, and time-history accelerograms. These products should reflect the parameters that control the structural response of the dam and/or appurtenant structures. Typically, this is the spectral acceleration at the predominant period of a structure, or perhaps the area under a response spectrum curve covering more than one structural vibration period if several modes contribute to the structural response. It may be necessary to ask for different hazard curves for different structures forming the reservoir retention system. In some cases, a certain combination of acceleration and velocity may be critical to the structural response, and hazard curves would need to be developed that relate to simultaneous exceedance of given acceleration and velocity levels.

5.1.3 Probabilistic Seismic Hazard Analysis

A probabilistic seismic hazard analysis (PSHA) involves relating a ground-motion parameter to its probability of exceedance at the site. The value of the ground-motion parameter to be used for the seismic evaluation is then selected after defining a probability level, applicable to the dam and site considered. PSHA considers the contributions from all potential sources of earthquake shaking collectively. Uncertainty is treated explicitly, and the annual probability of exceeding specified ground motions (commonly expressed as response spectra acceleration(s) at the period of interest), are computed. Alternatively, the analysis may be performed for a specified duration of time (such as the operating life of the dam). PSHA involves a thorough mathematical and statistical process that takes into account local and regional geologic and tectonic settings, as well as applicable historic and geologic rates of seismic activity. The results are typically expressed in terms of peak ground acceleration (PGA), peak ground velocity (PGV), or spectral amplitudes at specified periods.

Whereas in the past deterministic approaches have been favored in dam engineering, there has been a gradual shift to probabilistic methods for determining ground-motion parameters. Therefore, the rest of this document will concentrate on the PSHA approach. Any PSHA study has three basic components: 1) seismic source characterization, 2) development of ground-motion estimates, and 3) development of the site response. Additional steps include the development of uniform hazard spectra and development of acceleration time histories.

Staff Position:

- PSHA is considered the state of practice for evaluating seismic hazards for dam failure.

5.2 Seismic Failure by Structure Type

The impact of seismic ground motions and key failure modes of interest will depend on the design and construction of the dam. The following sections discuss the key concerns and seismic failure modes for concrete and embankment dams, as well as spillways, gates, and other appurtenances. This is followed by a short discussion of levees.

5.2.1 Concrete Dams

Under earthquake loading, concrete dams will respond to the level and frequency of the ground shaking, combined with the forces due to water in the reservoir. The tensile strength of the concrete under such dynamic loading is typically an important consideration. However, both structural and foundation failure modes may be important. If the shaking is severe enough, cracking and subsequent partial or complete separation of the contact surface between blocks may propagate through the structure. Ground surface displacement along a fault or liquefaction of foundation soils could lead to cracking and failure. It should be noted that the post-earthquake stability of the dam and foundation may be reduced depending on the level and duration of shaking experienced. An earthquake may also damage the dam’s drainage system. The stability of the dam could be threatened if the drain functions are impaired to the point that uplift pressure increases significantly.

Although no concrete dam foundations are known to have failed as a result of earthquake shaking, unprecedented seismic loads would in effect be a first-loading condition that could trigger movement and failure of arch dam foundation blocks. Therefore, it is important to analyze and evaluate the risks associated with potential earthquake-induced foundation instability.

Historically, concrete arch dam failures have resulted primarily from sliding of large blocks within the foundation or abutments. However, since there have been no known arch dam failures as a result of earthquake shaking (USBR 2011), there is no direct empirical evidence to indicate how an arch dam would structurally fail under seismic loading. Shake table model studies and numerical simulations (e.g., three-dimensional dynamic finite element analysis) provide the basis for postulated failure modes. These studies indicate that structural failure is initiated by cantilever cracking across the lower central portion of the dam, followed by diagonal cracking parallel to the abutments. This type of cracking eventually leads to isolated blocks within the dam. The isolated blocks may subsequently rotate (swing downstream or upstream), catastrophically failing the dam and releasing the reservoir.

The design of older buttress dams generally considered only the gravity and water pressure loads, and the buttress configuration is remarkably efficient in providing the resistance required for such loading. However, in the interest of efficiency, the buttresses were made very slender and thus they have very little strength for resisting cross-stream accelerations. Under strong shaking, it is conceivable that an older slab and buttress or multiple arch dam designed in this manner may suffer significant cracking/buckling and fail in domino fashion through the successive collapse of its buttresses. Typically both structural and foundation failure modes should be considered. The foundation stiffness can have a large effect on the rotations at the base of the buttresses and the dynamic response of the dam.

Under earthquake loading, concrete buttress and multi-arch dams will respond to the level and frequency of the ground shaking, combined with the forces due to water in the reservoir. Forces due to the water will depend upon the details of the design. The entire mass of water directly over the dam face will move with the vertical seismic motion. For flat slabs, there will be little or no cross-canyon hydrodynamic forces generated. Designs with cylindrical arches, domes or massive head buttresses will be subject to cross-canyon hydrodynamic forces. Depending upon the element of the structure under consideration, either the tensile, shear, or compressive strength will be an important consideration. For struts (which provide lateral support to the buttresses, when present) compressive strength is important. Slab-type water barriers supported by a corbel carry load by compression, shear and moment. Shear and tensile strength is important for the buttresses and the supporting corbels.

Staff Positions:

- Seismic analysis of concrete dams should include assessment of ground shaking, surface displacement, and forces due to water in the reservoir.
- Both structural and foundation failure modes should be considered.
- Foundation liquefaction/deformation potential should be considered.
- Structural failure modes considered should take into account the unique concerns for the type of dam in question.

5.2.2 Embankment Dams

Although many embankment dams have been exposed to earthquake shaking, there have been few instances where an earthquake has damaged an embankment dam enough to result in the uncontrolled release of reservoir water. Either the damage caused by the earthquake was not extensive enough, or in the rare cases where damage was extensive, the reservoir was far below the damage and uncontrolled releases did not occur. However, in spite of the relatively few failures experienced, it remains true that earthquakes can initiate a wide variety of potential failure modes in embankment dams. Shaking can cause loss of strength or even liquefaction of foundation or embankment soils, leading to deformation, sliding, or cracking failures.

Extensive shear strength reduction beneath an embankment slope can trigger a flow slide that, in turn, can produce a very rapid dam failure. Many cycles of low-amplitude loading can also induce a fatigue-like shear strength loss in dense, saturated, materials. A translational failure can occur if the entire foundation beneath an embankment liquefies and the reservoir pushes the embankment downstream far enough to create a gap in the vicinity of an abutment.

There are many ways in which cracking can occur due to seismic shaking, such as differential settlement upon shaking, general disruption of the embankment crest, offset of a foundation fault, or separation at spillway walls. Surface displacements can lead to cracking of the dam foundation, embankment or conduits passing through the dam. Shearing of a conduit passing through an embankment dam due to fault displacement can allow transmission of high-pressure water into the dam, leading to increased gradients and potential for internal erosion.

Staff Positions:

- Seismic analysis of embankment dams should include assessment of ground shaking and surface displacement.
- Both structural and foundation failure modes should be considered.
- Deformation and liquefaction potential of both the dam and the foundation should be considered.

5.2.3 Spillways, Gates, Outlet Works and Other Appurtenances

There are a number of facilities, not unique to any one dam type, for which loss of function during or following a seismic event could directly cause uncontrolled release of the reservoir through the failed gate or lead to uncontrolled release of the reservoir via overtopping, erosion, or some combination of these. Chief among these are spillways, gates and other outlet works.

Gates may fail to operate for a variety of reasons during seismic events. Dynamic loading may cause buckling of the gate itself. The seismic event could damage a gate hoist mechanism mounted above the gates, or cause shear or moment failure of supporting structures such as the piers in which the gates are mounted. Inoperability of a gate can cause a reservoir to fill beyond its design maximum water level (causing failure due to increased hydrostatic forces or overtopping) if not corrected in a timely manner.

Staff Position:

Seismic evaluation of dams should include consideration of whether a seismic event could lead to dam failure and subsequent uncontrolled release of the reservoir due to loss or degraded function of spillways, gates, outlet works and other appurtenances.

5.2.4 Levees

Earthquakes can damage or cause complete failures of levees. The most common mode of earthquake-induced damage is expected to be lateral spreading and cracking associated with earthquake shaking. As for earthen dams, shaking may cause liquefaction of soils within the levee or in the foundation soils. Design of levee systems for seismic performance has generally had low priority in the past, except for so-called “loaded levees” with a high likelihood of having coincident high water and earthquake loading. (e.g., levees in the Sacramento-San Joaquin Delta of California).

Staff Position:

- As with hydrologic levee failures, seismic failure of distant levees is not of concern. Failure of an offsite levee that provides flood protection to the NPP site, where applicable, is of interest (seismic analysis of onsite water control structures including levees, if applicable, are part of the Recommendation 2.1 Seismic Reevaluation.
- Levees without seismic design criteria should be assumed to fail during a seismic event. Survival of a loaded levee during an earthquake event should be justified through appropriate engineering analysis.
- In general, levees are not designed to withstand significant seismic loads. Therefore, to examine consequences of seismic failure, assume a starting water level elevation corresponding to a 500-year flood or top of the levee, whichever is less.
- Levees should not be assumed to fail in a beneficial manner, without appropriate engineering justification.

5.3 Analysis of Seismic Hazards Using Readily Available Tools and Information

Because there will generally be insufficient time and resources to perform detailed seismic analyses for all dams upstream from a NPP site, the following approach may be applied. It is assumed that the screening approach described in Section 3 has already been applied (i.e., inconsequential dams have been removed and non-critical dams have been screened out).

The analysis approach outlined in this section and in Section 5.4 is meant to take advantage of existing information for the dam (e.g. seismic design information or seismic qualification studies), along with a consistent level analysis of the seismic hazard (e.g. existing seismic hazard curves or seismic hazard assessments developed using readily available tools and data). In order to apply this approach, the seismic capacity of the dam (i.e., based on seismic design or post-construction seismic capacity studies) must be known. The seismic capacity should be characterized for frequencies of importance to the dam (e.g. design response spectrum). Note that there may be different capacities depending on the failure mode. For example, the capacity for concrete cracking for a concrete dam may be different from the capacity for sliding.

The licensee has the option of using this approach or conducting a more detailed site-specific characterization of seismic hazards at the dam site (as discussed in Section 5.7) as well as more detailed analysis of the seismic capacity of the dam (discussed in Section5.8). The options for performing seismic hazard analysis are outlined in Figure 18.

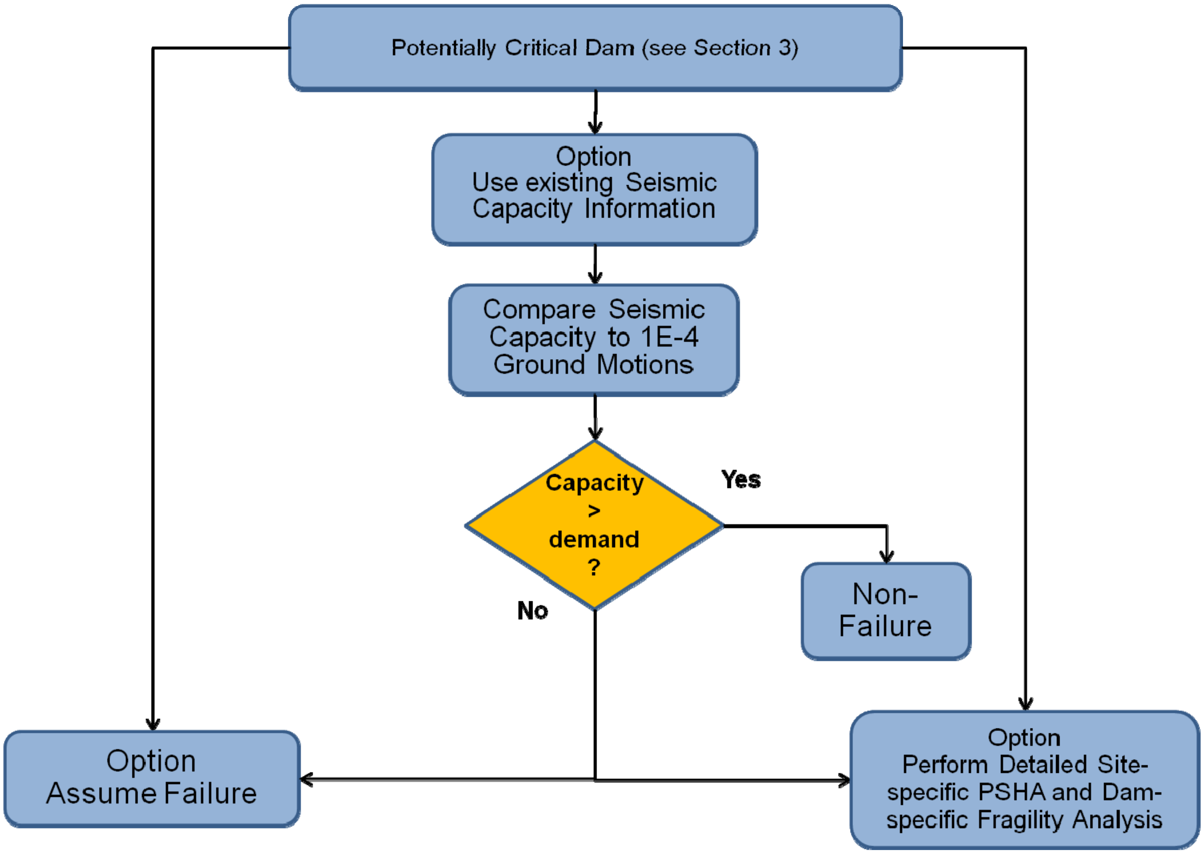

Figure 18. Seismic Dam Failure Analysis Options

5.3.1 Ground Shaking

Ground shaking is one of the most common seismic loads that should be considered for dams. As discussed in Section 1.4.2, it acceptable to use the $1x10^{-4}$ annual frequency ground motions, at spectral frequencies important to the dam, for seismic evaluation. Uniform hazard spectra (UHS) are computed or developed from the seismic hazard curves. This is done by developing hazard curves (i.e., spectral acceleration versus exceedance probability) for several vibration periods to define the response spectra. Then, for a given exceedance probability or return period, the ordinates are taken from the hazard curves for each spectral acceleration, and an “equal hazard” response spectrum is generated. Thus, the response spectra curves are generated for specified annual exceedance frequencies of interest.

Staff positions:

- The seismic hazard at the dam site should be characterized using probabilistic seismic hazard assessment (PSHA) for the spectral frequencies of interest to the dam:
  - The data and software tools available from USGS, which were used to develop the most recent version of the National Seismic Hazard Maps (this is the 2008 version as of the publishing of this guidance) are suitable for developing bedrock hazard curves and uniform hazard spectra at $1x10^{-4}$ annual frequency of exceedance. (USGS, 2008). However, due diligence should be applied to demonstrate the continued validity of the data used in USGS (2008) for sites in the western US (see Section 5.7.1.1).

    - The site amplification functions developed by the Electric Power Research Institute (EPRI, 1989) should be used with the bedrock hazard curves (developed using methods described above) to obtain site-specific soil hazard curves as described in NUREG/CR-6728 (USNRC, 2001).

- As an alternative to the use of the USGS seismic hazard curves, it is acceptable to perform a detailed site-specific PSHA consistent with the methodologies suitable for use in characterizing seismic hazard at U.S. nuclear power plant sites, as described in Regulatory Guide 1.208 (USNRC, 2007). General aspects of a site-specific seismic hazard analysis are discussed in Section 5.7. Regulatory Guide 1.208 provides more definitive and detailed guidance.

5.3.2 Fault Displacement

For some dam sites, the potential for surface fault displacement through the dam site or foundation is a concern. Another possibility concerns an active fault passing through the reservoir of an embankment dam. Fault offset within the reservoir could create a seiche wave capable of overtopping and eroding the dam. Seiche waves can be generated by large fault offsets beneath the reservoir or by regional ground tilting that encompasses the entire reservoir. “Sloshing” can lead to multiple overtopping waves from these phenomena.

Two types of surface faulting are generally recognized: principal (or primary) and distributed (or secondary) surface faulting. Principal faulting occurs along the main fault plane(s) that is the locus of release of seismic energy. Distributed, or secondary faulting, is displacement that occurs on a fault or fracture away from the primary rupture and can be quite spatially discontinuous.

Probabilistic fault displacement hazard analyses (PFDHA) can be performed in a manner analogous to that used for probabilistic ground motion. The results are represented by a hazard curve, which shows annual occurrence of fault displacement values (i.e., the annual frequency of exceeding a specified amount of displacement). A recent example is the analysis conducted for Lauro Dam near Santa Barbara, California (Anderson and Ake, 2003). This analysis followed the methodology that was used for the proposed Yucca Mountain, Nevada nuclear waste repository (Stepp et al. 2001; Youngs et al., 2003).

Staff Position:

- Dam sites should be evaluated for the potential for surface fault displacement to cause damage to the dam.
- The potential for primary and secondary surface faulting should be considered.
- It is acceptable to utilize existing analyses that demonstrate that a dam is not susceptible to fault displacement.

5.3.3 Liquefaction

During an earthquake, soils may undergo either transient or permanent reduction in undrained shear resistance because of excess pore water pressures or disruption of the soil structure accompanying cyclic loading. Such strength degradation may range from slight diminution of shear resistance to the catastrophic and extreme case of seismically induced liquefaction, which is a transient phenomenon. In this guide, the term seismically induced liquefaction includes any drastic loss of undrained shear resistance (stiffness and/or strength) resulting from repeated rapid straining, regardless of the state of stress prior to loading. The term is interchangeably applied to the development of either excessive cyclic strains or complete loss of effective stress within an undrained laboratory specimen under cyclic loading (sometimes referred to as initial liquefaction).

An initial assessment of the potential for earthquake-induced ground failure typically includes:

1. Geomorphology of the site.
2. A soil profile, including classification of soil properties and the origin of soils at the site
3. Water level records, representative of both current and historical fluctuations.
4. Evidence obtained from historical records, aerial photographs, or previous investigations of past ground failure at the site or at similar (geologically and seismologically) nearby areas (including historical records of liquefaction, topographical evidence of landslides, sand boils, effects of ground instability on trees and other vegetation, subsidence, and sand intrusions in the subsurface).
5. Seismic history of the site.

Detailed investigations would include surveys, in situ field testing, and laboratory testing, as appropriate, to (a) refine the preliminary interpretation of the stratigraphy and the extent of potentially liquefiable soils, (b) measure in situ densities and dynamic properties for input to dynamic response analyses, and (c) recover undisturbed samples for laboratory testing when site soils are not adequately represented in the available database.

Regulatory Guide 1.198, "*Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plants*" (USNRC, 2003) provides guidance on acceptable methods for evaluating the potential for earthquake-induced instability of soils resulting from liquefaction and strength degradation. It provides descriptions of screening techniques as well as procedures for detailed analysis.

<u>Staff positions:</u>

- The dam site should be evaluated for liquefaction potential.
- Regulatory Guide 1.198 provides guidance on acceptable methods for evaluating the potential for earthquake-induced instability of soils resulting from liquefaction and strength degradation.

5.4 Assessment of Seismic Performance of Dams Using Existing Studies

In lieu of performing a new seismic hazard evaluation of dam performance, it is acceptable to utilize existing studies or design documentation to demonstrate the seismic capability of a dam.

<u>Staff Positions</u>

Existing studies will be accepted on a case-by-case basis. However, studies utilized should ideally consider seismic capacity for both the maximum normal operating pool level (i.e. top of active storage) and average pool level (i.e. 50% exceedance duration pool level calculated using average daily water levels for the period of record). The average non-flood tailwater level should be used with both headwater conditions above.

5.4.1 Ground Shaking

In order to utilize existing studies to demonstrate the seismic capability of a dam, the seismic capacity of the dam (e.g., based on seismic design or post-construction seismic capacity studies) must be known for spectral frequencies of importance to the dam (e.g., using design response spectrum).

Staff positions:

- The seismic demands on the structure should be defined using the site-specific hazard spectrum (based on the UHS and accounting for site amplification) as described in Section 5.3.1. The design spectrum (or spectrum determined by other seismic analyses) is compared against the site-specific hazard spectrum to assess the failure potential of the dam. If the capacity of the structure exceeds the site-specific seismic demands at the spectral frequencies of relevance to the dam, with appropriate margin to account for uncertainties in the analysis, the dam can be assumed not to fail due to seismic ground shaking. Appropriate margin is usually expressed as a factor of safety, which will depend upon the type of dam and failure mode under consideration. FEMA guidelines on earthquake analysis of dams (FEMA 2005) provide additional detail. Note that the factor of safety should be applied relative to the ground motion criteria defined in this ISG. If the relevant Federal agency guidance proposes a factor of safety of 1.4 under the ground motions they consider, then the licensee would show that this factor of safety (1.4) is maintained when the dam is subjected to the 1e-4 ground motion.
- In cases where information does not exist to characterize the capacity of the dam by response spectrum or define capacities at the frequencies of relevance to the dam (e.g., in the case when the dam design was based on pseudo-static analysis using a single demand such as peak ground acceleration and the dam has not been reevaluated to define capacity in terms of other intensity measures), the licensee may leverage such analysis with appropriate justification. Examples of appropriate justification include demonstration of the conservatism and applicability of the analysis, in light of the UHS developed in Section 5.3.1 including effects of site amplification of a range of spectral frequencies.
- Dams that cannot be shown to have sufficient capacity should be assumed to fail and breach parameters computed as described in Section 7 Moreover, dams that are susceptible to seismic failure should be evaluated for the potential for multiple dams to fail during a single seismic event as described in Section 5.5. Alternatively, it is acceptable to perform a more detailed assessment of the performance of the dam (i.e., performing new assessments) as described in Section 5.8.

5.4.2 Fault Displacement

Staff position:

- Existing studies or data on dam or foundation materials can be used to assess performance of the dam with respect to surface displacement, in light of the seismic hazard defined for the site, with appropriate justification of their applicability and with appropriate conservatism to account for uncertainties.

5.4.3 Liquefaction

Staff position:

- Existing studies or data on dam or foundation soils can be used to assess performance of the dam with respect to liquefaction or loss of strength, in light of the seismic hazard defined for the site, with appropriate justification of their applicability and with appropriate conservatism to account for uncertainties.

5.5 Multiple Dam Failure Due to a Single Seismic Event

Comparison of the seismic capacity of a dam to the dam-specific seismic hazard, as described above, may produce a set of dams that are vulnerable to failure at or below the ground motion level associated with a $1E^{-4}$ annual frequency of exceedance. For these dams, it is necessary to consider the potential for a single seismic event to cause multiple dam failures. In general, the potential for multiple dam failures can be addressed through consideration of the distance between the dams, as described below.

In some cases, using knowledge about the attenuation of ground motion with distance relative to the distance between dams may provide useful information to assess the potential for common failure. For example, by considering one or more ground motion prediction equations (GMPEs) and amplification relations applicable for the location and characteristics of the site, it is possible to evaluate how, for a large magnitude event (e.g., M=6.5), the ground motion attenuates with distance for relevant ground motion measures (e.g., spectral accelerations at predominant frequencies of the dam). By considering a conservative estimate of ground motion attenuation (e.g., use of 84^{th} percentile versus median values), it may be shown that two dams are sufficiently far apart that an earthquake affecting one dam will be unlikely to affect the other dam because the ground motion would likely attenuate to negligible level. Figure 19 provides a graphical illustration of the above concept. A GMPE is used to conservatively select a distance beyond which the ground motion (conservatively) attenuates to a negligible level (i.e., relative to the design capacity of the dam). This distance is used to define a “ring” around the dam with radius defined according to the selected distance. If the circles do not overlap for two dams, then failure during the same event could be considered unlikely.

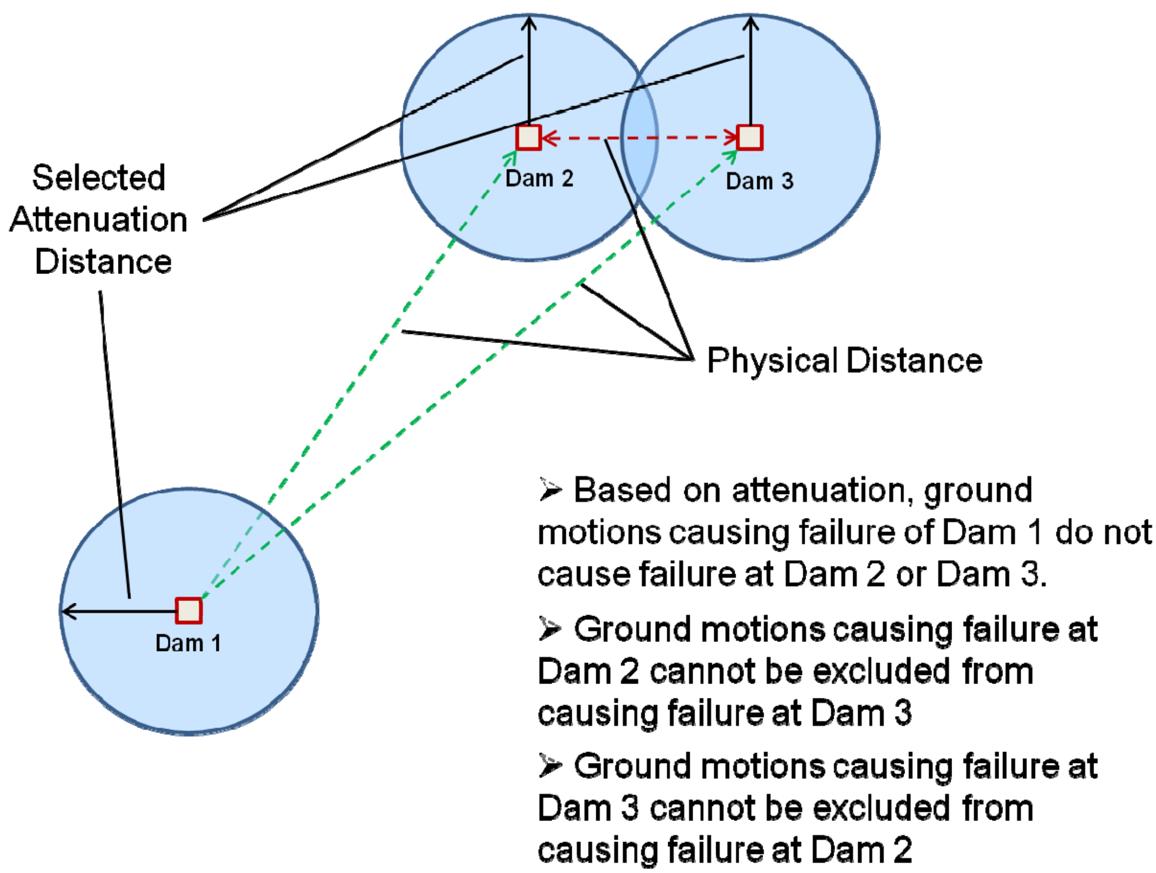

Figure 19. Using Knowledge about the Attenuation of Ground Motion with Distance

If the attenuation distance approach outlined above does not rule out combined failure, the potential for multiple failures may be further refined through deaggregation of the seismic hazard at the location of the dams. Deaggregation provides information and insight into the seismic sources that impact the hazard at a particular site. As a result, insights into scenarios leading to multiple dam failures may be gained through deaggregation of seismic hazard for relevant ground motion measures and at ground motion levels corresponding to the multiple annual frequencies (e.g., 10^{-2}, 10^{-3}, 10^{-4}). When considering relevant ground motion measures, multiple annual frequencies, and site characteristics, if the deaggregation of the hazard indicates that a large portion of the hazard (e.g., greater than 85-90%) comes from scenarios associated with earthquakes within a specified distance, this distance can be used, in combination with similar information for other dams, to justify that dams are sufficiently far apart such that it is unlikely they will fail during a single seismic event. Graphically, this corresponds to modifying the radius of the “ring” around a dam site in accordance with the results of the hazard deaggregation (see Figure 20).

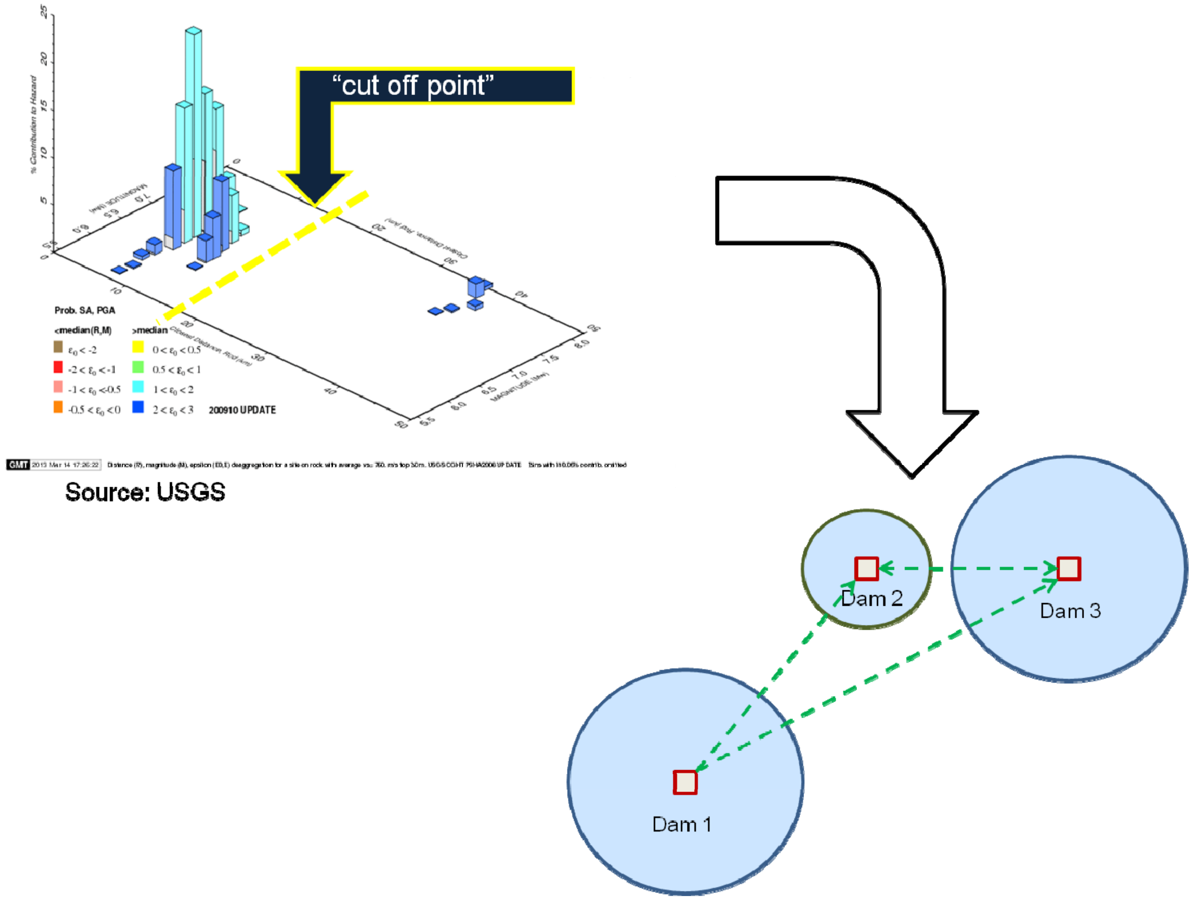

Figure 20. Refinement of Seismic Influence Using Deaggregation

<u>Staff positions:</u>

- The approaches outlined above can be used to identify the collection of dams that should be considered in a multiple-failure scenario, but not the precise sequence of the failures.
- Once the critical dams have been identified, various failure sequences should be considered to arrive at a suitably conservative estimate of the multiple-failure consequences.

5.6 Modeling Consequences of Seismic Dam Failure

Once a dam has been assumed to fail under seismic load, the consequences of dam failure will be developed through breach modeling and flood wave routing as discussed in Sections 7 and 9, respectively. However, assumptions regarding headwater and tailwater levels, as well as coincident flood flows are discussed here

Staff Positions:

- If the dam failed under the 10^{-4} annual exceedance probability seismic hazard (ground motion), assume that failure coincides with the peak water level from a 25-year flood in the watershed above the dam.
- If the dam failed at half of the 10^{-4} annual exceedance probability seismic hazard (ground motion), assume that failure coincides with the peak water level from a 500-year flood (or ½ PMF, whichever is less) in the watershed above the dam.
- Water level estimates at the site should include effects of a 2-year wind speed from the critical direction.
- In view of the uncertainties involved in estimating reservoir levels that might reasonably be expected to prevail at the time of failure, the default starting water surface elevation used in flood routings for evaluation of seismic failure consequences should be the maximum normal pool elevation (i.e. top of active storage pool). Other starting water surface elevations may be used, with appropriate justification. Justification should be based on operating rules and operating history of the reservoir. The operating history used should be of sufficient length to support any conclusions drawn. However, consideration should be given to possible instances where the operating history and/or rules have been influenced by anomalous conditions such as drought.
- Reservoir and downstream tributary inflows should be consistent with the selected reservoir level. Hydrologically consistent headwater or tailwater relationships should be used in routing the flood wave.

5.7 Detailed Site Specific Seismic Hazard Analysis

When the analysis using readily available information (as described above) indicates that a dam cannot be screened out based upon its design-basis information, a more detailed, site-specific seismic hazard evaluation is required if failure is not assumed. Approaches for such a detailed analysis are discussed below.

Staff Position:

Because each dam and its immediate environment form a unique system, it is not feasible to provide detailed guidance that will be applicable in all cases. Therefore, detailed, site-specific seismic hazard analyses will be reviewed on a case-by-case basis. The following discussion is meant to provide a general overview of the pieces that would normally be part of a detailed seismic hazard evaluation.

5.7.1 Ground Shaking

Detailed estimates for ground shaking at the dam site can be developed using the same PSHA procedures discussed earlier, only using more detailed, site-specific information. This may take the form of more detailed seismic source characterization, attenuation relations or site response functions.

5.7.1.1 Seismic Source Characterization

Seismic source characterization is concerned with the identification of all the relevant potential earthquake sources. Earthquake sources typically consist of: 1) faults, and 2) areal or background seismic source zones. Fault sources are usually modeled as planar surfaces, where the parameters such as activity (expressed by either slip rate or recurrence interval), geometry (location, length, dip, and down-dip extent), sense of slip, segmentation (some segments may be more active than others), maximum magnitude (Mmax), and recurrence model (characteristic earthquake or maximum magnitude) are specified. For areal source zones, the parameters of interest are maximum magnitude, associated rate of activity, and recurrence model (e.g., exponentially truncated Gutenberg-Richter relationship, or just truncated exponential). The fault sources are primarily characterized using geological data, while the areal or background source zones are characterized using historical seismicity occurrence and magnitude data.

Many definitions exist for what constitutes an "active" or "potentially active" fault and hence what faults should be considered potential seismic sources. Simply put, most faults considered in a PSHA to be potential seismic sources are faults with evidence of Quaternary activity; i.e., faults that have documented or suspected evidence of displacement during roughly the last 1.6 million years. For most studies, all faults within approximately 50 km of the site are characterized, but for some sites, faults as far as 1000 km or more distant are included if they can have a significant impact on the hazard (such as large earthquakes originating on the Cascadia Subduction zone of western Washington, Oregon, and northern California, or the San Andreas fault in California). The U.S. Geological Survey (USGS), many state geological surveys, and various consulting companies have conducted studies of potentially active faults in the United States. The USGS has produced catalogues describing the faults and other seismic sources used to develop the National Seismic Hazard Maps (e.g., the Quaternary Fault and Fold Database of the United States).

Seismic Sources for the CEUS In addition to the USGS seismic source catalogues (USGS, 2008), for the region designated as the central and eastern U.S. (CEUS), a regional study was jointly conducted by the U.S. Nuclear Regulatory Commission (NRC), EPRI, and DOE during the period 2009-2011, to develop a comprehensive representation of seismic sources for nuclear plant seismic evaluation purposes. The results were published by the NRC in 2012, and provide an acceptable source characterization model for use in seismic hazard studies.

Seismic Sources for the WUS In the Western United States (WUS), numerous studies have been conducted over the last 30 years or so that identify and characterize in one form or another most of the known or suspected Quaternary faults. For example, the U.S. Bureau of Reclamation has a large inventory of studies in the Western United States that include detailed seismic source characterizations. Much of this data has been compiled by the USGS and can be accessed at http://earthquake.usgs.gov/hazards/qfaults/imsintro.php.

It should be emphasized that the USGS Quaternary fault compilation was done primarily to facilitate the development of the national seismic hazard maps, and many faults are not in the database, such as those that have low slip rates, were characterized recently, or were characterized for private companies. In addition, the quality of the data and level of study varies greatly for many faults. Finally, it also must be emphasized that for a critical facility, such as a dam, even faults with fairly low slip rates (~0.01 mm/year) can be important if the fault is located close to the site and if the downstream consequences are significant. Therefore, in the WUS, development of seismic sources and the adequacy of the USGS National Seismic Hazard Map (USGS, 2008) should be determined on a site-specific basis.

5.7.1.2 Ground Motion Attenuation

Ground-motion prediction equations (GMPEs) or attenuation relations relate the source characteristics of the earthquake and propagation path of the seismic waves to the ground motion at a site. The predicted ground motion is typically quantified in terms of a median value (a function of magnitude, distance, style of faulting, and other factors) and a probability density function of peak horizontal ground acceleration or spectral accelerations. GMPEs are statistical models developed by combining geophysical attenuation models with regression analysis of recorded strong motion databases. Ground motion parameters are typically expressed as functions of earthquake magnitude, distance from the rupture zone, the type of faulting, and site conditions. Because they are based on recorded strong motion data, GMPEs change with time as more strong motion data becomes available.

The state of the art in GMPEs for shallow crustal earthquakes is well represented by relationships developed as part of the Next Generation Attenuation (NGA) Project sponsored by the Pacific Earthquake Engineering Research (PEER) Center Lifelines Program. The NGA models are based on an extensive database of strong ground motion recordings and were developed through the efforts of five selected attenuation relationship developer teams working in a highly interactive process with other researchers. These relationships have a substantially better scientific basis than earlier ground motion attenuation relationships. In order to model site conditions, most of the NGA ground motion attenuation relationships incorporate the input parameter V_{S30}, which is the average shear-wave velocity in the upper 30 m at the site. Development of the database of strong motion recordings is discussed in Chiou et al. (2008) and the attenuation relationships are available on the PEER website: http://peer.berkeley.edu/products/rep_nga_models.html.

In 2004, EPRI published a set of GMPEs for the CEUS, which was subsequently updated in 2006. A second update is scheduled to be released in late 2015.

Staff Position:

> Ground motion prediction equations approved by the NRC for Recommendation 2.1 Seismic are acceptable for use in dam failure analysis for Recommendation 2.1 Flooding.

5.7.1.3 Site Response

Local site conditions can profoundly influence important characteristics of strong ground motion (e.g., amplitude, frequency content, direction). The extent of the influence depends upon the geometry and the material properties of subsurface materials, site topography, and the characteristics of the input motion. In particular, the characteristics of local soil deposits can have a significant impact on the ground motions experienced at the surface. Critical parameters that determine which frequencies of ground motion might experience significant amplification (or de-amplification) are the layering of soil and/or soft rock, the thicknesses of these layers, the initial shear modulus and damping of these layers, their densities, and the degree to which the shear modulus and damping change with increasing ground motion. The site response is typically addressed by developing amplification functions or amplification factors that relate the bedrock ground motions to motions at the ground surface. Methods to calculate possible site amplification are well established, but at some sites, the characterization of the subsurface may be limited.

It is well known that topographic irregularities and alluvial basin geometry can have significant effects on ground motions (Kramer, 1996). Due to where they are typically sited, this effect should be considered in developing the site response for dams. In fact, the best-known example of apparent topographic effects was observed at a dam site during the 1971 San Fernando earthquake (Trifunac and Hudson, 1971). Evaluation of these effects requires two- and in some cases three-dimensional analysis. At this time, only linear 2-D and 3D analyses are standard practice.

5.7.1.4 Development of Uniform Hazard Spectra

Uniform hazard spectra (UHS) are computed or developed from the seismic hazard curves. This is done by developing hazard curves (i.e. spectral acceleration vs. exceedance probability) for several vibration periods to define the response spectra. Then, for a given exceedance probability or return period, the ordinates are taken from the hazard curves for each spectral acceleration, and an "equal hazard" response spectrum is generated. Thus, the response spectra curves are generated for specified annual exceedance frequencies of interest.

5.7.1.5 Development of Acceleration Time-histories

For higher-level studies, acceleration time histories are developed for the sites that represent the seismic hazard at the return periods of interest. The selected ground motions are then used for dynamic analyses. A wavelet-based method is currently employed by Reclamation to produce acceleration time-histories through spectral matching to the 5% damping mean UHS at the return period of interest. Because the UHS calculated from the PSHA curves is only available for the horizontal component of the ground motion, the vertical-component response spectra used for spectral matching is found by scaling the UHS using estimated V/H ratios.

In addition to the response spectra, additional characteristics of the time history, such as phase spectra and strong shaking duration, are also needed to produce a time history using spectral matching. A suitable recording from an historic earthquake is used as the initial time history to provide the required characteristics for the spectral matching. This earthquake and the recording station should be of similar magnitude and distance as the earthquake event dominating the UHS. In some cases, the records from several different events should be used because of the differences in the ground motions produced by earthquakes with even similar magnitudes and distances. If there are no near-field strong ground motion recordings from historical earthquakes, synthetic accelerations generated by stochastic methods are used.

5.8 Detailed Analysis of Seismic Capacity of the Dam

Once the earthquake ground motions or displacements have been determined, the impact they have on the dam and its appurtenances must be determined. The extent and type of analysis required for the seismic evaluation of a dam depends on the hazard potential classification, level of seismic loading, the site conditions, type and height of dam, construction methods, as-built as well as current material properties, and engineering judgment. Consistency should be maintained between the level of analysis and level of effort given to the development of seismotectonic data, the ground motion parameters, and the site investigation. For example, a highly refined structural analysis based on an assumed earthquake loading is not reasonable in most cases. Likewise, a highly refined structural analysis should use site-specific ground motions, not assumed values.

In general, it is the most cost-effective for seismic analyses to begin with the simplest conservative method appropriate to the problem. If the structure is judged able to resist the earthquake loading within certain safety margins from the initial analysis, then further analysis should not be necessary. If further studies are needed, they would be progressively more detailed and the structure evaluated accordingly. Regardless of the method of analysis, the final evaluation of the seismic safety of the dam will include engineering judgment and experience, not just numerical results of the analyses.

In some cases, the analyses may indicate that the dam is either clearly safe or clearly unsafe. Frequently, however, judgments concerning the safety of the dam must take into account not only the results of the analyses but also the level of confidence that can be put in those analyses and underlying assumptions and, to some extent, the consequences of misjudging the level of uncertainty.

Staff Position:

Seismic capacity studies should consider seismic capacity for both the maximum normal operating pool level (i.e. top of active storage) and average pool level (i.e. 50% exceedance duration pool level calculated using average daily water levels for the period of record). The average non-flood tailwater level should be used with both headwater conditions above.

5.8.1 Concrete Dams

Concrete dams should be analyzed for the effects of ground shaking and surface displacements. Cracking in the dam as well as displacement of foundation materials, leading to sliding or overturning are common concerns.

5.8.1.1 Sliding and Overturning Stability

Concrete dams (gravity, arch, or buttress) and sites are very unique and should be evaluated for stability under earthquake loading.

Excessive cracking is a safety concern for concrete gravity dams subjected to earthquakes, which can lead to potential instability of the dam from sliding or overturning. Sliding can be on an existing plane of weakness in the dam or foundation or along planes of weakness formed by excessive cracking of the concrete above or at the foundation-dam interface. For concrete dams, sliding instability is possible due to an earthquake-induced vibratory motion on a plane of weakness at, above, or below the foundation-dam interface.

For an arch dam, sliding instability is more likely to occur by failure of the abutment support because the arching effect provides additional resistance to sliding within the dam. In general, instability of gravity and arch dams caused by excessive cracking of the concrete is most likely to occur in the upper half of the dam.

Buttress dams are also particularly vulnerable to cross-valley shear motions that can result in tipping of the buttresses and loss of support for the reinforced concrete slab.

Of the two possible types of instability discussed above, historical experience shows that foundation (abutment) induced failure is often the chief source of concern for concrete dams. In contrast to the dam itself, the supporting medium consists of natural materials of varying composition, irregular joints, and planes of weakness. The strength of this medium is generally estimated from exploratory borings and tests on only a small fraction of the material present. Key zones of weakness are critical and often difficult to detect.

In the past, pseudostatic methods were commonly used to analyze dam stability. However, given the widespread availability of structural dynamics software, pseudostatic methods are generally discouraged today and should only be used for screening from further consideration those dams where a seismic stability failure is highly improbable. An example of such a screening analysis is given in FEMA (2005):

Structures that fail to meet the prescribed pseudostatic stability requirements (i.e., sliding safety factors and resultant location) should be subjected to in-depth study using dynamic analyses to assess the demands placed on the dam and foundation during an earthquake. Dynamic time-history analyses are used to determine the displacements and stresses experienced by the dam and foundation. Evaluation of the results is used to determine if there is a risk of a stability failure.

<u>Staff Positions:</u>

- Pseudostatic methods are generally discouraged from use in stability analysis of structures.
- Structures that fail to meet prescribed pseudostatic stability requirements (i.e., sliding safety factors and resultant location) should be subjected to in-depth study using dynamic analyses to assess the performance of the dam and foundation during an earthquake.
- Detailed evaluation of the seismic performance of a concrete dam should be performed using (as appropriate) linear-elastic response spectrum analysis, linear-elastic time-history analysis, or nonlinear time-history methods. Guidance provided on these methods in FEMA Dam Safety Guidelines (FEMA, 2005) should be used to perform the evaluation.

5.8.1.2 Dynamic Analysis

Results of dynamic analyses are generally evaluated in terms of compressive and tensile strengths of the concrete. The compressive stresses resulting from the combination of static and earthquake loads usually remain below the dynamic strength of the concrete. However, since the mere occurrence of tensile stresses does not necessarily lead to failure, the significance of predicted tensile stresses is not evaluated as easily. The number and amplitudes of stress cycles that exceed the dynamic tensile strength are taken into account for this purpose in linear analysis.

To evaluate the effects of stresses that exceed the tensile strength, sound engineering judgment is required and should be based on the expected effects of nonlinear behavior and the past performance of dams under similar earthquake loadings. To estimate the extent of cracking, one must consider nonlinear behavior leading to stiffness degradation and energy absorption. Nonlinear behavior from cracking reduces the stiffness of the dam and shifts the dam’s response into other frequency ranges of the ground motion. The energy level of the earthquake corresponding to the frequency of the cracked structure may or may not be more severe than when uncracked. As a result, the peak values of tensile stress and the extent of tensile zones may increase or decrease, and large tensile stresses given by linear elastic analysis may or may not necessarily indicate an unsafe condition. They may, in fact, be “artifacts” of the analysis rather than real behavior.

A nonlinear analysis should be performed if the response of the dam would be influenced significantly by nonlinearity from material behavior or changes in geometry. For an arch dam, this might include 1) cantilever tensile stresses larger than the tensile strength of the concrete over significant areas of the dam; 2) a long duration earthquake; 3) opening and closing of contraction joints indicated by simultaneous arch tensions on the upstream and downstream faces; and 4) large displacements or distortions of the arch.

<u>Staff Positions:</u>

- A nonlinear analysis should be performed if the response of the dam would be influenced significantly by nonlinearity from material behavior or changes in geometry.
- Detailed evaluation of the seismic performance of a concrete dam should be performed using (as appropriate) linear-elastic response spectrum analysis, linear-elastic time-history analysis, or non-linear time-history methods. Guidance provided

on these methods in FEMA Dam Safety Guidelines (FEMA, 2005) should be used to perform the evaluation.

5.8.2 Embankment Dams

The most common concern for embankment dams is deformation and/or liquefaction of the embankment or foundation. These may fail the dam directly due to overtopping or indirectly due to cracking and subsequent internal erosion. The deformation may be due to surface displacement, but ground shaking is a more common concern.

If a dam is deformed by earthquake excitation or fault displacement, the deformations can cause cracks in the dam and/or disrupt internal filters, either of which could lead to failure of the dam by erosion. Cracks are most likely to occur at interfaces with concrete structures (e.g., spillway walls) or at abrupt changes in the embankment's cross section. There is also evidence that shaking could precipitate piping even without formation of a crack if the dam is already on the verge of piping. The amount of deformation a dam can withstand without risk of failure by erosion through cracks depends on the materials in the dam and foundation, the details of internal zoning (filters, drains, and cutoff), the reservoir elevation at the time of the earthquake, and the nature and location of appurtenant structures. Should there be conduits through the embankment, deformation of the dam can rupture them or cause joints to separate, leading to erosive failure by either creating an unfiltered exit for seepage or exposing the embankment or foundation to full reservoir head where not intended. Erosion along intact conduits has also caused dam failures.

5.8.2.1 Deformation Analyses

Direct methods of assessing deformation model the design earthquake, the dam, and foundation to calculate the expected deformation. There are also indirect methods to predict the response of the embankment and foundation based on empirical observations. Post-earthquake stability analysis can, in a sense, also be considered an indirect prediction of deformation – if the post-earthquake factor of safety is high, the deformations should be limited to a few feet except under very severe loading. The magnitude of deformations is very dependent on the strengths of the materials involved. During strong shaking, permanent deformations (usually small) may occur simply because the dynamic stresses temporarily exceed the available strength. In saturated soils, there is frequently some loss of shearing resistance due to an increase in pore water pressure when shaken. This increases the dynamic deformations over what they would be with no strength loss. In very loose, contractive soils, the strength may become a small fraction of its static, drained value due to excess pore-water pressure (liquefaction). Very large deformations can result, driven by gravity even after the shaking ends. There is an intermediate condition known as "cyclic mobility," in which the shearing resistance is initially very low due to excess pore pressure, but increases with larger shear strains, helping to prevent gross instability but still permitting significant deformation.

Deformation analyses can be made for three conditions: 1) liquefaction would not occur; 2) liquefaction may occur but instability would not; and 3) liquefaction may occur, resulting in instability. In the first two conditions, judgment is required to determine whether the predicted deformations along the critical failure surfaces are small enough that cracking of the embankment/foundation materials that could eventually cause a piping failure of the dam does not occur. A determination also must be made whether the post-earthquake sliding factors of safety and available freeboard are adequate to ensure the dam would not be overtopped and would be able to safely retain the reservoir. If there are no potentially liquefiable materials present, this can usually be done by the simple Newmark sliding-block approach. In situations where excess pore pressure could develop, it may be necessary to conduct more rigorous finite-element or finite-difference analyses.

If the results of post-earthquake sliding stability analyses for critical failure surfaces indicate a safety factor well above 1.0 (e.g., 1.25 or greater) using the strengths expected after the earthquake, experience from past earthquakes suggests that deformations will be small and the dam will perform satisfactorily (FEMA, 2005). Confidence in the safety of the dam decreases when the factor of safety against triggering of liquefaction is 1.0 or less and a post-earthquake sliding factor of safety less than or approaching 1.0 is calculated using residual shear strengths for materials assumed to be liquefied. In general, many analyses have shown that when a wedge or circular sliding surface has a low post-earthquake sliding factor of safety, the deformations on these sliding planes will be excessive. If these failure planes are critical to the overall integrity of the dam, deformations may lead to failure of the dam by overtopping or internal erosion.

FEMA (2005) provides a screening-level analysis for embankment dams similar to that discussed for concrete dams above. FEMA (2005) suggests that for a dam and foundation not subject to liquefaction, minor deformation may take place but should not lead to failure if all of the following conditions are satisfied:

- Dam and foundation materials are not subject to liquefaction and do not include loose soils or sensitive clays.
- The dam is well built and compacted to at least 95% of the laboratory maximum dry density (modified Proctor test), or to a relative density greater than 80%.
- The slopes of the dam are 3:1 (H:V) or flatter, and/or the phreatic line is well below the downstream slope of the embankment.
- The peak horizontal acceleration at the base of the embankment is no more than 0.2 g.
- The static factors of safety for all potential failure surfaces (other than shallow surficial slides) are greater than 1.5 under loading and pore-pressure conditions expected immediately prior to the earthquake.
- The freeboard at the time of the earthquake is at least 3% to 5% of the embankment height and not less than 3 feet (0.9 m). (Freeboard requirements to accommodate reservoir seiche waves or coseismic movement of faults at the dam site or in the reservoir must be considered as a separate issue.)
- There are no critical appurtenant features that would be harmed by small movements of the embankment, or that have the potential to cause cracks that would allow internal erosion.

If these conditions are not satisfied, more detailed study is required. This may include assessment of liquefaction potential, post-earthquake stability analysis, and/or deformation analysis.

A comprehensive review of the factors to be considered in the earthquake resistance design of dams, as well as a review and commentary on the field performance of dams during earthquakes, can be found in Seed (1979). Regulatory Guide 1.198, "*Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plants*" (USNRC, 2003) provides guidance on acceptable methods for evaluating the potential for earthquake-induced instability of soils resulting from liquefaction and strength degradation.

Staff Positions:

- Detailed seismic evaluation of embankment dams should include the following (as appropriate): post-earthquake stability analysis, deformation analysis, and assessment of liquefaction potential.
- If there are no potentially liquefiable materials present, evaluation can usually be done using the Newmark sliding-block approach. In situations where excess pore pressure could develop, more rigorous finite-element or finite-difference analyses should be conducted.
- Embankment dams should be evaluated to ensure sufficient factors of safety against sliding of critical failure surfaces.
- Embankment dams should be evaluated to ensure sufficient factors of safety against triggering of liquefaction.

6. OTHER (SUNNY DAY) FAILURES

Dam failures not associated with a concurrent extreme flood or seismic event may arise from a variety of causes. These failures are often referred to as sunny day or fair weather failures. These dam failures may occur because of failures of embankment material, foundations, or appurtenances such as floodgates, valves, spillways, conduits, and other components. The potential for these failures to occur should be carefully evaluated. American National Standards Institute/American Nuclear Society Standard 2.8-1992, “Determining Design Basis Flooding at Nuclear Power Plant Sites” (ANSI/ANS, 1992), lists the potential nonhydrologic and nonseismic causes for partial or complete dam failure. That list, with minor modifications, is:

- Deterioration of concrete (e.g., weathering, cracking, chemical growth)
- Deterioration of embankment protection (e.g., grass cover, riprap, or soil cement)
- Excessive saturation of downstream face or toe of embankment.
- Excessive embankment settlement.
- Cracking of embankment due to uneven settlement.
- Excessive pore pressure in structure, foundation, or abutment.
- Excessive loading due to buildup of silt load against dam.
- Excessive leakage through foundation.
- Embankment slope failure.
- Leakage along conduit in embankment.
- Channels from tree roots or burrowing.
- Landslide in reservoir.

More detailed discussion of failure modes by dam type is provided below.

Staff Positions:

- Sunny-day failures cannot be screened-out. If no other failure mechanisms exist, sunny-day failures should be the default failure scenario for the purposes of this ISG (see discussion in Section 1.4).
- An exception to the preceding staff position is that dams failed due to hydrologic and seismic events shown to have negligible impacts at the site do not require evaluation for the sunny-day scenario since the sunny-day scenario is bounded by the other two events. The level of effort required for evaluating sunny-day failure is typically lower since it only involves identifying the worst-case individual or cascading failure scenario.
- Sunny-day failures such as those listed above should be carefully evaluated to ensure that all plausible mechanisms for flooding from dam breaches and failures at and near a site are considered.
- A sunny-day breach can be used to model piping failures for hydrologic, geologic, structural, seismic, and human-influenced failure modes.

6.1 Overview of Sunny Day Failures by Structure Type

6.1.1 Concrete Dams

Several potential failure initiators are common to all types of concrete dams. These include plugging of drains (leading to increased uplift pressures), gradual creep that reduces the shear strength on potential sliding surfaces, and degradation of the concrete from alkali-aggregate reaction, freeze-thaw, or sulfate attack.

For concrete gravity dams founded on bedrock, the leading cause of dam failures has been related to sliding on planes of weakness within the foundation, most typically weak clay or shale layers within sedimentary rock formations. For concrete gravity dams founded on alluvial soils, the leading cause of failure is piping or “blowout” of the soil material from beneath the dam. Failures have also occurred along weak lift joints within dams.

Historically, arch dam failures have resulted primarily from foundation deficiencies. The predominant mode of failure is sliding of large blocks (bounded by geologic discontinuities) within the foundation or abutments. Typically, these failures have been sudden, brittle, and have occurred on first filling of the reservoir.

Plugging of the drains may lead to an increase in the foundation uplift pressure. This may lead to creep along sliding surfaces. Additionally, degradation of the concrete may occur from alkali-aggregate reactions, freeze-thaw cycles, or sulfate attack. All of these may lead to dam failure. Some of these mechanisms may be difficult to detect. A review of instrumentation results can be helpful. For example, if piezometers or uplift pressure gauges indicate a rise in pressures, and weirs indicate a reduction in drain flows, the drains may be plugging leading to higher uplift and potentially unstable conditions. If conditions appear to be changing, risk estimates are typically made for projected conditions as well as current conditions.

Because of high unit loads underneath the buttresses, concrete buttress dams are subject to failures initiated by weakness within the foundation. Such weakness may cause the foundation to undergo unacceptable settlement or shearing. Sliding on planes of weakness within the foundation may also occur. Concrete buttress dams founded on alluvial soils are subject to failure initiated by piping or “blowout” of the soil material from beneath the dam. Deformation of the abutment can also lead to failure since unforeseen movement in the abutment will induce stresses in a buttress that may not have been considered in its design. Particular attention must be paid to the quality and performance of the concrete in the face slab. Because of its relative thinness it cannot withstand excessive deterioration, pitting, or spalling that will decrease the strength of the slab. Exposure and corrosion of reinforcing steel can reduce the capacity of reinforced concrete elements.

6.1.2 Embankment Dams

Historically, the most common failure modes for embankment dams are initiated by or heavily influenced by various seepage-related internal erosion phenomena. Internal erosion phenomena are the predominant mechanism for sunny-day (nonhydrologic, nonseismic) failures of embankment dams. The term internal erosion is used here as a generic term to describe erosion of particles by water passing through a body of soil. “Piping” is often used generically in the literature, but actually refers to a specific internal erosion mechanism. Several types of internal erosion have been observed in embankments.

Classical piping occurs when soil erosion begins at a seepage exit point, and erodes backwards, supporting a “pipe” or “roof” along the way. Progressive erosion can occur when the soil is not capable of sustaining a roof or a pipe. Soil particles are eroded and a temporary void grows until a roof can no longer be supported, at which time the void collapses. This mechanism is repeated progressively until the core is breached or the downstream slope is over-steepened to the point of instability. "Suffosion" or internal instability occurs when the finer particles of a soil are eroded through the coarser fraction of that soil, leaving behind a coarsened and more permeable soil skeleton. The loss of material can lead to voids and sinkholes.

"Scour" occurs when tractive seepage forces along a surface (i.e. a crack within the soil, adjacent to a wall or conduit, or along the dam foundation contact) are sufficient to move soil particles into an unprotected area. Once this begins, a process similar to piping or seepage erosion could result.

"Heave" can occur where an impervious layer overlies more pervious material near the downstream toe of a dam. A buildup of hydrostatic pressure beneath the impervious layer can lead to high uplift forces capable of moving material from and breaching of the impervious layer. This in turn can lead to rapid development of piping or seepage erosion (unless the pressure is relieved to the point where the seepage velocities are insufficient to move soil particles). This is sometimes referred to as "blowout," especially if it occurs in a local area.

The various internal erosion phenomena discussed above may affect the embankment (including spillway walls), the foundation or both. The zone of contact between earth materials and conduits through the embankment or its foundations and around drains is an area prone to internal erosion phenomena. More detailed discussion of internal erosion and piping for earthen dams is provided in USBR (2011).

6.2 Analysis of Sunny Day Failures

Analysis of sunny day failure can be organized into three basic steps:

Step 1. Assessment of potential failure modes

Step 2. Breach modeling

Step 3. Flood wave routing

Failure modes and assumptions regarding initial water surface elevation (Step 1) used in breach modeling (Step 2) and flood routing (Step 3) are discussed below. The details of breach modeling are discussed in Section 7 and details of flood routing are discussed in Section 9.

Base flow conditions for a sunny day failure are typically ignored because of the small discharge and volume compared to that of a dam breach. As a general guidance, base flow can be ignored if the dam breach flow is two times greater than the base flow. Where base flow is considered, the discharge is typically estimated based on reported base flows through the dam's outlet works or from stream gage records. Additional inflow (e.g. from a storm event) is not required when analyzing a sunny-day breach.

6.2.1 Sunny- Day Failure Modes

An essential element in evaluating the potential for sunny-day failure is assessment of credible failure modes. Common sunny-day failure modes for various dam types are discussed in Section 6.1. That discussion is not meant to be exhaustive. The purpose of the discussion is to inform the process of identifying potential failure modes. In general, identifying potential failure modes will require a thorough review of all relevant background information on a dam, including geology, design, analysis, construction, operations, dam safety evaluations, and performance monitoring documentation.

6.2.2 Initial Water Surface Elevation

Breaching should be assumed and breaching scenario(s) should be assessed, if sunny day failure modes cannot be ruled out. Section 7 discusses breach modeling in detail, but does not discuss assumptions regarding initial water surface elevations used in the breach modeling.

Staff Position:

In view of the uncertainties involved in estimating reservoir levels that might reasonably be expected to prevail at the time of failure, the default starting water surface elevation used in flood routings for evaluation of overtopping should be the maximum normal pool elevation (i.e. top of active storage pool). Other starting water surface elevations may be used, with appropriate justification. Justification should be based on operating rules and operating history of the reservoir. The operating history used should be of sufficient length to support any conclusions drawn (e.g., 20 years or more). However, consideration should be given to possible instances where the operating history and/or rules have been influenced by anomalous conditions such as drought.

7. DAM BREACH MODELING

The breach is the opening formed in a dam when it fails, the aim of a breach analysis is to estimate the resulting reservoir outflow hydrograph. Modeling of the breach formation (or development) process has typically been one of the greatest sources of uncertainty in dam failure analysis, and is especially important when the dams distance from the location(s) or population(s) of interest is small, and routing effects are minimized (Gee, 2008; Wahl 2004, 2010).

The simplest approach to breach modeling is to assume that the dam fails completely and instantaneously. While this assumption is convenient when applying simplified analytical techniques for analyzing dam-break flood waves, and is somewhat appropriate for concrete arch dams, it is not considered realistic for either earthen or concrete gravity dams, which tend to fail partially and/or progressively. Concrete gravity dams tend to have a partial breach (as one or more monolith concrete sections are forced apart by the escaping water), although the time for breach formation is in the range of a few minutes. Earthen dams do not tend to fail completely, nor do they tend to fail instantaneously. Dam breach analysis of composite dams (dams that include both concrete and earthen sections) should consider the failure of the portion or portions of the dam that would produce the largest peak outflow.

7.1 Breach Modeling for Concrete Dams

In most cases where breach of a concrete dam is considered, one or more sections of the dam deemed most susceptible to failure (based on engineering analysis) are assumed to fail instantaneously. The breach size and shape are determined by considering the size and shape of the failed section(s), and then a weir formula or hydraulic simulation software package is used to compute the outflow hydrograph and/or peak outflow.

Concrete gravity dams tend to have a partial breach as one or more monolith sections formed during construction of the dam are forced apart and overturned by the escaping water. The time for breach formation depends on the number of monoliths that fail in succession, but is typically on the order of minutes. The challenge of modeling breach of concrete dams is in predicting the number of monoliths that may be displaced or fail. However, by using a dam-breach flood prediction model and running several trials of the model, wherein the breach width parameter representing the combined lengths of assumed failed monoliths is varied in each trial, the resulting reservoir water surface elevations can be used to indicate the extent of reduction in the loading pressures on the dam. Because the hydraulic loadings diminish as the breach width increases, a limiting-safe loading condition, which would not cause further failure, may be estimated (Fread, 2006).

Unlike concrete gravity dams, concrete arch dams tend to fail completely and are assumed to require only a few minutes for the breach formation. The geometry of the breach is usually approximated as a rectangle or a trapezoid. Buttress and multi-arch dams can be modeled in a similar fashion, where sections are assumed to fail completely.

7.2 Breach Modeling of Embankment Dams

For earthen dams, the failure process often begins when appreciable amounts of water flow over or around the dam face and begin to scour the face of the dam. In general, the most erosive flow occurs on the downstream slope, where the velocity is highest and where the slope makes it easier to remove material.

On dams that have been overtopped by floods, severe erosion has often been observed to begin where sheet flow on the slope meets an obstacle, such as a structure, a large tree, or groin, creating local turbulent flow. Erosion generally continues in the form of "headcutting," upstream progression of deep eroded channel(s) that can eventually reach the reservoir. Pavement on the crest of the dam may be of some value in slowing head cutting once the gullies reach the crest, but should not be expected to affect initiation.

For cohesive soils, the failure mechanism is typically headcut initiation and advance. A small headcut is typically formed near the toe of the dam and then advances upstream until the crest of the dam is breached. For cohesionless soils, the failure process typically initiates because of tractive stresses from the flow removing material from the downstream face, but then progresses as headcut advance once a surface irregularity is formed. Predicting whether breach initiation and formation will occur can be a complicated procedure. Several factors have been shown to be important, including:

- Depth and duration of overtopping
- Potential concentration of overtopping flows at dam crest due to camber or low spots
- Potential concentration of overtopping flows on the dam face, along the groins or at the toe of the dam
- Erosional resistance of materials on the downstream face and in the downstream zones of the embankment
- Whether the dam crest is paved
- Whether a parapet wall is provided and the potential for the wall to fail before or after the dam is overtopped

For embankment dams, failure typically begins at a point on the top of the dam and expands in a generally trapezoidal shape. The water flow through the expanding breach acts as a weir; however, depending on conditions such as headwater and tailwater, various flow characteristics can be observed during a breach development including weir flow, converging flow, and channel flow.

Breach analysis for earthen and rockfill embankment dams is typically more complex than for other dam types. As mentioned above, failure of embankment dams is typically progressive. Failure progression differs for overtopping and piping failures (the two most common failure modes for earthen dams), as described below.

- In the case of overtopping, once a breach has been initiated, the discharging water will progressively erode the breach until either the reservoir water is depleted or the breach resists further erosion. Erosion processes typically result in the progressive widening and deepening of the breach. In some cases, the breach will deepen until bedrock or some other erosion resistant strata is encountered. At this point, the breach depth stays approximately constant while the breach continues to widen. The final breach shape is often modeled as trapezoidal (see Figure 21).
- Piping is a term used to describe an array of internal erosion failure modes that can be very different in their initial stages, but all cause the breach opening in the dam to initially form at some point below the top of the dam. As erosion proceeds, a “pipe" through the dam enlarges until the top of the dam collapses, or the breach becomes large enough that open channel flow occurs. Beyond this point, breach enlargement is similar to the overtopping case.

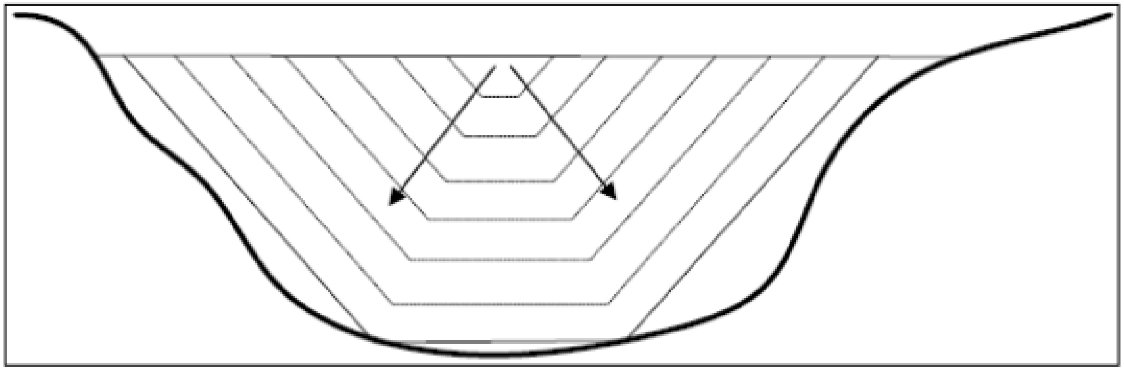

Figure 21. Generalized Trapezoidal Breach Progression (Gee, 2008)

Breach widths for earthen dams are usually much less than the total length of the dam. The breach also requires a finite interval of time for its formation through erosion of the dam materials by the escaping water. The total time of failure may range from a few minutes to a few hours, depending on the height of the dam, the type of materials used in construction, and the magnitude and duration of the flow of escaping water.

There are two widely used approaches to breach modeling of embankment dams, both based on regression analysis of data from dam failures. The first approach is direct estimation of the breach outflow hydrograph by simple equations that relate the peak outflow discharge and time for breach development to basic reservoir and embankment parameters. Once the peak outflow is estimated, it can be used to complete the analysis of flooding impacts. The second approach uses regression equations to predict parameters of the breach opening (e.g., size, shape, and rate of development) when given input data such as reservoir volume, initial water height, dam height, dam type, configuration, failure mode, and material erodibility. These breach parameters are then used in a computational model that determines the breach outflow through the parameterized opening using a weir or orifice flow equation (e.g., Gee and Brunner, 2005; Xiong, 2011).

7.2.1 Regression Equations for Peak Outflow from the Breach

For screening-level analyses, direct estimation of the breach outflow hydrograph by simple equations that relate the peak outflow discharge and time for breach development to basic reservoir and embankment parameters is often adequate. The equations are developed by regression of case-study data. A number of such regression equations have appeared in the literature in the past 35 years. Some attempt to provide conservative estimates by developing equations that envelop the case study data, while others provide a best fit to the data. The number and types of dams included in the studies vary, but typically the studies have been small (~10-20 dams) and skewed towards small dams. The following list provides a description of some, but not all, existing equations for peak outflow:

- Kirkpatrick (1977) presented data from 13 embankment dam failures and six additional hypothetical failures and provided a best-fit relation for peak discharge as a function of depth of water behind the dam at failure. This study included data from the failure of one concrete gravity dam (St. Francis Dam, in California) because, at the time of the study, this dam was thought to have failed due to piping in the abutment. More recent studies have questioned that explanation.
- The Soil Conservation Service (SCS, 1981) used the 13 case studies cited by Kirkpatrick to develop a power law equation relating the peak dam failure outflow to

the depth of water behind the dam at time of failure. This was meant to be an enveloping relation although three data points are slightly above the curve.

- USBR (1982) extended this work and proposed a similar relation for peak outflow using case study data from 21 dams. This study also proposed a relation for attenuation of the peak flow with distance downstream. USBR (1983) later analyzed six case studies with large storage-to-height ratios and proposed a modification to the USBR (1982) peak discharge equation that included reservoir storage as a parameter.
- Singh and Snorrason (1982, 1984) used 10 real dam failures and eight simulated failures to develop peak dam failure outflow as functions of dam height and reservoir storage.
- MacDonald and Langridge-Monopolis (1984) developed best fit and envelope curves for peak outflow from studies of 42 breached earthfill dams.
- Costa (1985) developed envelope curves and best-fit regression equations for peak flow from 31 breached dams as functions of dam height, storage volume at time of failure and the product of these two parameters. Costa's study included both constructed and natural dams, as well as the St. Francis Dam failure.
- Froehlich (1995a) developed a best-fit regression equation for peak discharge based on reservoir volume and head, based on 22 case studies. He also presented a procedure for determining confidence intervals for the estimates.
- Pierce et al. (2010) conducted a study that compares several of the regression equations described above to each other and to a database developed by the U.S. Bureau of Reclamation (USBR, 1998). This study provides insights into the degree of conservatism in the equations studied.

Staff Position:

- For screening-level analysis, the use of simple equations that relate the peak outflow discharge to basic reservoir and embankment parameters is acceptable with adequate justification. The list above describes several available regression equations for peak outflow. Selection of methods should consider the assumptions inherent in the models and their applicability to the dam failure scenario being considered. Sensitivity studies should be performed on a reasonable variation of input parameters, when applicable. If there are multiple applicable models available for use, a study should be performed to evaluate the effect of model selection (and input parameter sensitivity, when applicable) on the results of the analysis. Justification for the preferred model and input parameters should be documented, including results of sensitivity studies.

7.2.2 Regression Equations for Breach Parameters

Regression equations have been developed to predict parameters of the breach opening (e.g., size, shape, and rate of development) when given certain input data are available such as reservoir volume, initial water height, dam height, dam type, configuration, failure mode, and material erodibility. These parameters are then used in a computational model that determines the breach outflow through the parameterized opening using a weir or orifice flow equation. Wahl (2010) suggests that one of the main advantages of using empirical parametric regression equations is that the analyst can exert some control over the breach parameters used in the model, and thus account for site-specific factors. A large number of relationships have been published in the last 35 years. The U.S. Bureau of

Reclamation, the U.S. Army Corps of Engineers, and others have compiled extensive reviews of the most widely used regression-based approaches for breach parameter estimation, including discussions of uncertainties in the methods [Gee, 2008; Wahl, 2004,2010; Washington State Department of Ecology (WSDE), 2007; Colorado Department of Natural Resources (CODNR), 2010].

The list below provides descriptions of some, but not all, of the available regression models for breach parameters:

- Johnson and Illes (1976) published a classification of failure configurations for earthfill, gravity, and arch dams. The breach shape for earthen dams was described as varying from triangular to trapezoidal as the breach progressed.
- Singh and Snorrason (1982) conducted a study of 20 dam failures and noted that breach width was generally between two and five times the dam height. They also found that the breach formation time was generally 15 minutes to 1 hour and the maximum overtopping depth prior to failure (for overtopping failures) ranged from approximately 0.5 foot to 2 feet.
- Based on 42 dam failure case studies, MacDonald and Langridge-Monopolis (1984) proposed a "breach formation factor," defined as the product of the volume of breach outflow and the depth of water above the dam. They related this factor to the volume of material eroded from the dam's embankment. The amount of water that passes through the breach is not known before breach analysis occurs; however, the entire volume of the reservoir can be used as a starting estimate (Gee, 2008). Based on their study, MacDonald and Langridge-Monopolis also concluded that the breach could be assumed to be trapezoidal with a side slope of 0.5H:1V. The study further presented an envelope equation for the breach formation time for earthfill dams.
- Based on a study of 52 earthen embankment dam breaches, Singh and Scarlatos (1988) determined that the ratio of top and bottom breach widths ranged from 1.06 to 1.74, with an average value of 1.29. They also noted that the majority of breach formation times were less than 1.5 hours and most were less than 3 hours.
- In 1995, Froehlich developed equations for breach width and breach formation time based on 63 case studies (Froehlich, 1995a). Froehlich suggested using a breach side slope factors of 1.4 for overtopping failures and 0.9 for other failure modes. Froehlich (2008) provided updated breach parameter equations based on data collected from 74 embankment dam failures. He also used the new equations and their uncertainties in a Monte Carlo simulation to estimate the degree of uncertainty in predictions (Froehlich, 2008).
- Xu and Zhang (2009) used a database of 182 earth and rockfill dam failure cases. A multiparameter nonlinear regression was used to develop empirical relationships between five breaching parameters (breach depth, breach top width, average breach width, peak outflow rate, and failure time) and five selected dam and reservoir control variables (dam height, reservoir shape coefficient, dam type, failure mode, and dam erodibility). A significant feature of this study is that nearly one-half of the 182 case studies focused on dams higher than 15 m (a commonly used metric for large dams). However, the majority of these case studies are of dam failures in China, which have not been subject to independent review by the U.S. dam safety community. Another feature is that Xu and Zhang's relations explicitly include erodibility of embankment soils. Their study showed that breach parameters are

very sensitive to the selected erodibility index (dam erodibility was found to be the most important factor, influencing all five breaching parameters).

- Review of the Xu and Zhang paper by NRC staff has revealed several concerns. The Xu and Zhang paper does not provide clear criteria for selecting the erodibility index. Their paper also has an internally inconsistent treatment of breach formation time and time to failure, which leads to uncertainty in how these definitions were applied in the case studies supporting their regression analysis. In addition, discussions with staff from USBR and USACE indicate that the Xu and Zhang treatment of failure time is not consistent with how this parameter is applied in widely used hydraulic models (e.g., HEC-RAS). Use of the Xu and Zhang failure time could thus lead to invalid results. There is also concern that the Xu and Zhang relation for failure time may be biased in favor of longer times (Wahl, 2013; Brunner, 2013).

Staff Positions:

- The state of practice in dam breach modeling shows a clear preference for regression-based approaches. The preferred approach uses regression equations to predict final parameters of the breach opening (.e.g. size, shape, time to fully develop) when given input data such as reservoir volume, initial water height, dam height, dam type, failure mode, and material erodibility.
- Based on discussions with technical experts at USACE and USBR, as discussed above, and the lack of independent review of the case studies for Chinese dam failures, there is concern about the use of breach parameters computed using Xu and Zhang (2009). Given the time necessary to conduct a sufficient review of the model in conjunction with other Federal agencies, and the uncertainty regarding its ultimate acceptability, the NRC staff does not recommend use of the breach parameter model developed in this paper (Xu and Zhang 2009) for the purposes of conducting the Recommendation 2.1 hazard review. This position applies to Category 2 and 3 plants, while Category 1 sites that used the Xu and Zhang relationship (and submitted their Recommendation 2.1 flood hazard reevaluation report in March 2013) will be reviewed on a case-by-case basis (see Section 1.2).

7.2.2.1 Uncertainty in Predicted Breach Parameters and Hydrographs

Predicting the reservoir outflow hydrograph remains a great source of uncertainty, especially for embankment dams in which dam failure is usually a complex progressive process that is difficult to model (Wahl, 2010). Since the scale of estimated consequences associated with a dam failure can be sensitive to the choice of breach parameters, careful consideration should be given to the selection of the proper method(s) of determining breach parameters and the associated uncertainty, not only with the parameters themselves, but also of the overall result of the breach modeling efforts.

Numerous regression equations, summarized in the preceding sections of this ISG, have been developed for peak discharge and breach parameters. The available equations vary widely depending on the analyst and the types of dam failures studied. Regression equations suffer from a lack of well-documented case study data as well as a high level of uncertainty in the data used to develop the equations. Approximately 75 percent of the dam failures used to develop the equations are less than 50 feet in height; therefore, these equations may not be very representative of dams greater than 50 feet in height. According to Wahl (2010), the best methods of breach width prediction are empirically derived parametric equations (e.g., USBR, 1988; Von Thun & Gillette, 1990; and Froehlich, 1995a).

These methods were found to have uncertainties of about ± one-third of an order of magnitude.

In general, predictions of the side slope of dam breach openings have a high uncertainty, although this is of secondary importance since breach outflows are relatively insensitive to the selection of side slopes. In the case studies used to develop most regression equations, observed final breach openings are generally sloped. However, based on laboratory and scale model studies, researchers believe that side slopes are nearly vertical while the breach is actively eroding and enlarging.

Predictions of breach formation time may also be subject to great uncertainty due to a lack of reliable case study data; many dams fail without eyewitnesses, and the problem of distinguishing between breach initiation and breach formation phases has likely tainted much of the data (USBR, 1998). An analysis by Wahl (2004) found that most of the empirically developed equations for predicting time of failure had uncertainties of about ±1 order of magnitude; the best predictions were obtained with the equation by Froehlich (1995b). Newer equations by Froehlich (2008) and Xu and Zhang (2009) which are based on more case studies and additional parameters (e.g., erodibility) may be marginally improved, but breach failure time predictions should still be considered highly uncertain.

7.2.2.2 Performing Sensitivity Analyses to Select Final Breach Parameters

With a wide range of methods available that can produce a wide range of results for breach width and breach formation time, a sensitivity analysis should be performed prior to selecting the final breach parameters. The sensitivity analysis should not be restricted to identifying the impact of varying the breach parameters on the peak discharge and breach hydrograph at the dam. It should also identify the effect of breach parameters on the calculated water surface elevations at locations of interest downstream from the dam. While a model may indicate that the stage and outflow at the dam vary greatly depending on the selected breach parameters, the effect on the stage, flow, and travel time to an area of interest downstream of the dam may be smaller due to routing effects (e.g., flood attenuation and floodplain hydraulics).

Significant engineering judgment must be exercised in interpreting breach parameter and/or breach peak flow results. The sensitivity analysis could involve using several widely used predictor equations to establish breach parameters. However, it should be noted that the importance attached to details of the breach model are highest for those sites that are closest to the dam. As the distance between the dam and the site of interest increases, the attenuation of the flood wave reduces the influence of the breach details. Sensitivity analysis can be used to study this effect.

<u>Staff Positions:</u>

- Because of the large uncertainties, inconsistencies and potential biases associated with breach modeling, licensees should not rely on a single modeling method. Instead, licensees should compare the results of several models judged appropriate. Justification should be provided for the selection of the candidate models used as well as the value(s) for the specific model. Model and parameter uncertainty as well as parameter sensitivity in final results should be explicitly addressed.
- Studies have shown that failure time uncertainties can be quite large. Contributions to uncertainty include: (a) observations of failure time in case studies generally originate from non-professional eyewitness; and (b) lack of clear and consistent definition of failure time across (and sometimes within) studies. Therefore, licensees

should describe how failure time is defined and discuss how failure time was appropriately applied in the numerical model selected to simulate the breach formation and outflow hydrograph.

7.2.3 Physically-Based Combined Process Breach Models

Another approach to dam breach analysis for earthen dams is to use a combined process model that simulates specific erosion processes and the associated hydraulics of flow through the developing breach to yield a breach outflow hydrograph. These combined process models attempt to simulate the progression of a dam breach using sediment detachment and/or sediment transport equations that in turn rely on estimated erosion rates and soil mechanics relations to predict mass slope failures. Several models have been developed (e.g., Fread, 1991; Mohamed, 2002; Temple et al., 2006; Hanson et al. 2011; Visser et al. 2012; Wu, et al. 2010), but they are not nearly as widely used as the previously described regression approaches. The work required to develop site-specific parameters needed for application of these models is likely to be significant.

Staff Position:

The state of practice in dam breach modeling tends to emphasize regression-based approaches. However, use of physically based breach modeling will be considered on a case-by-case basis. If used, the parameters describing erosion and hydraulic properties should be developed from site-specific studies. Generic values or values obtained from the literature are, in general, not sufficient. Uncertainty and sensitivity studies should be performed to evaluate the effect of model and input parameter selection on the results of the analysis. Justification for the preferred model and values for input parameters should be provided, including documentation of uncertainty and sensitivity studies.

8. LEVEE BREACH MODELING

The breaching process for levees can be quite different from that of earthen dams. The principal differences include: (a) breach sensitivity to upstream and downstream conditions; (b) dimensionality of the outflow; and (c) flow direction relative to the structure.

One of the most significant differences is the effect of the upstream and downstream water conditions. In a dam breaching event, the upstream reservoir water level drops and the breach outflow discharge increases as the breach enlarges. At some point, the discharge will decrease as water level decreases and storage volume in the reservoir is depleted. The dam breach size and outflow are thus usually limited by reservoir characteristics, and downstream tailwater conditions are generally of secondary importance. In a scenario involving a levee failure bordering a large lake, only a minimal drop in water level can be expected. The breach size and outflow continue to increase until the tailwater downstream from the breach rises to reduce hydraulic stresses on the breach opening below the threshold for continued erosion. Thus, downstream tailwater rise is much more important than for dams. Tailwater rise has a similar effect on a riverine levee breach, but upstream river inflow (and hence catchment size) also affects the breach size and outflow by sustaining the water level in the river.

Outflow from a dam breach often flows into a narrow valley and is often as a one-dimensional flow. However, outflow from a levee breach usually spreads into a relatively flat plain. The diverging flow as water inundates land behind the levee is essentially two-dimensional in nature. This two-dimensional flow can be important when the levee is close to the site. When the levee breach is distant from the site, simplified modeling approaches are more appropriate.

In a coastal context, the presence of waves in the incipient breach increases sediment mobilization and transport. The breach flow may be affected by the tidal cycle, and water conditions on both sides of the embankment. In addition, a barrier breach may be closed naturally by the sediments transported from adjacent beaches and shores attributable to littoral drift, or it may increase in size and become a new inlet or estuary

In contrast to earthen embankment dams, very few studies have been carried out to derive regression equations for levee breach parameters. The parametric dam breach models discussed above may not be strictly applicable to levee breaches because water conditions upstream and downstream from levee breaches may be very different from those at dam breaches.

<u>Staff Positions:</u>

- In general, earthen embankment levees should be assumed to fail when overtopped. The case for nonfailure must be developed using detailed engineering analysis supported by site-specific information, including material properties of the embankment and foundation soils, material properties of embankment protection (if any), levee condition, etc. Other forms of levees (e.g., pile walls, concrete flood walls) should be evaluated for potential failures applicable to the particular type of levee.

- Levees are generally not designed to withstand high water levels for long periods. However, no generally accepted method currently exists for predicting how long a levee will continue to function under high loading conditions. Therefore, historical information is the best available basis for predicting levee performance. The historical information should be from levees that have similar design and construction characteristics as the levee being analyzed.

- Because there is no widely accepted method for modeling breach development in the case of levees, conservative assumptions regarding the extent of the breach and the failure time should be used.
- In general, inundation mapping of a NPP site from an onsite or nearby levee will require two-dimensional modeling.

9. FLOOD WAVE ROUTING

Regardless of the type of dam failure, the dam-break flood hydrographs represent dynamic, unsteady flow events. Therefore, a dynamic hydraulic model should generally be used to route the dam failure flood wave to the plant. Sensitivity of flood stage and water velocity estimates to reservoir levels, reservoir inflow conditions, and tailwater conditions before and after dam failure should be examined. Transport of sediment and debris by the floodwaters should be considered. However, as discussed in Section 3, there may be situations where using a simplified approach is appropriate.

This section describes hydrologic routing methods that are appropriate for use in modeling dam breach in hydrologic modeling software packages. Several commonly used methods are:

- Muskingum
- Modified Puls (also known as storage routing)
- Muskingum Cunge

Each of these models computes a downstream hydrograph, given an upstream hydrograph as a boundary condition. Each does so by solving the continuity equation and simplified version of the momentum equations.

An important consideration in selecting a routing method is the nature of the flood wave exiting the reservoir. The flood wave will rise from a fairly low value, very quickly to a value much greater than the initial flow. The extremely large flows will generally overflow the channel and enter the floodplain. Therefore, the routing method selected must be capable of nonlinear routing.

While some of the hydrologic routing methods include attenuation, none of them include acceleration. Acceleration can be a significant source of attenuation during a large floodwave in a flat channel. The attenuation by hydrologic methods is approximate. A hydraulic model may be necessary to accurately predict the attenuation.

9.1 Applicability and Limitations of Hydrologic Routing Models

Each routing model discussed above involves solving both the momentum and continuity equations. However, each omits or simplifies certain terms of those equations to arrive at a solution. To select a routing model, one must consider the routing method's assumptions and reject those models that fail to account for critical characteristics of the flow hydrographs and the channels through which they are routed. These include (but are not limited to) the effects discussed in the following subsections.

9.1.1 Backwater Effects

Tidal fluctuations, significant tributary inflows, dams, bridges, culverts, and channel constrictions can cause backwater effects. A flood wave that is subjected to the influences of backwater will be attenuated and delayed in time.

Practically, none of the hydrologic routing models will simulate channel flow well if the downstream conditions have a significant impact on upstream flows. The structure of the methods is such that the computations progress from upstream watersheds and channels to those downstream. Thus, downstream conditions are not yet known when routing computations begin. Only a full unsteady-flow hydraulic system model can accomplish this.

9.1.2 Floodplain Storage

If flood flows exceed the channel carrying capacity, water flows into overbank areas. Depending on the characteristics of the overbanks, the overbank flow can be slowed greatly, and often ponding will occur. This can significantly affect the translation and attenuation of a flood wave.

To analyze the transition from main channel to overbank flows, the model should account for varying conveyance between the main channel and the overbank areas. For one-dimensional flow models, this is normally accomplished by calculating the hydraulic properties of the main channel and the overbank areas separately, then combining them to formulate a composite set of hydraulic relationships. The Muskingum model parameters are assumed constant. However, as flow spills from the channel, the velocity may change significantly, so the Muskingum *K* should change. While the Muskingum model can be calibrated to match the peak flow and timing of a specific flood magnitude, the parameters cannot easily be used to model a range of floods that may remain in bank or go out of bank. Similarly, the kinematic wave model assumes constant celerity, an incorrect assumption if flows spill into overbank areas.

In fact, flood flows through extremely flat and wide flood plains may not be modeled adequately as one-dimensional flow. Velocity of the flow across the floodplain may be just as large as that of flow down the channel. If this occurs, a two-dimensional flow model will better simulate the physical processes.

9.1.3 Interaction of Channel Slope and Hydrograph Characteristics

As channel slope approaches zero assumptions in some of the hydrologic models will be violated (e.g., momentum-equation terms that were omitted become significant).

For example, the simplification for the kinematic-wave model is appropriate only if the channel slope exceeds 0.002. The Muskingum-Cunge model can be used to route slow-rising flood waves through reaches with flat slopes. However, it should not be used for rapidly rising hydrographs in the same channels, because it omits acceleration terms of the momentum equation that are significant in that case. Ponce et al. (1978) established a numerical criterion to judge the likely applicability of various routing models. He suggested that the error due to the use of the kinematic wave model is less than 5 percent if:

$$\frac{TS_0u_0}{d_0} \geq 171$$

where *T* = hydrograph duration, S_0 is the bottom slope u_0 is the reference mean velocity, and d_0 = reference flow depth. (These reference values are average flow conditions of the inflow hydrograph.) He suggested that the error with the Muskingum- Cunge model is less than 5 percent:

$$TS_0\sqrt{\frac{g}{d_0}} \geq 30$$

where *g* = acceleration of gravity.

9.1.4 Configuration of Flow Networks

In a dendritic stream system, if the tributary flows or the main channel flows do not cause significant backwater at the confluence of the two streams, any of the hydraulic or hydrologic routing methods can be applied. However, if significant backwater does occur at confluences, then models that can account for backwater must be applied. For networks, where the flow divides and possibly changes direction during the event, none of the simplified hydrologic models are recommended.

9.1.5 Occurrence of Subcritical and Supercritical Flow

During a flood, flow may shift between subcritical and supercritical regimes. If the supercritical flow reaches are short, this shift will not have a noticeable impact on the discharge hydrograph. However, if the supercritical-flow reaches are long, these should be identified and treated as separate routing reaches. If the shifts are frequent and unpredictable, then a hydraulic model is recommended.

9.1.6 Availability of Calibration Data Sets

In general, if observed data are not available, the physically based routing models will be easier to set up and apply with some confidence. Parameters such as the Muskingum *X* can be estimated, but the estimates should be verified with observed flows. Thus, these empirical models should be avoided if the watershed and channel are ungauged.

Staff Position

- The use of simplified hydrologic routing must justified and shown to be appropriate for use on a case-by-case basis.
- When available, records from the largest observed floods should be used to calibrate hydrologic and hydraulic models. Flood records from nearby hydrologically similar watersheds may also be useful.
- When flood records are not available, USGS regression equations for ungauged watersheds may be used to inform modeling.

9.2 Hydraulic Models

As stated above, hydraulic routing methods are preferred when routing flood waves from dam breach. Hydraulic routing models provide more accuracy when modeling flood waves from a dam breach. These models include terms that hydrologic models neglect. Typically, a dynamic hydraulic model should be used to route the dam failure flood wave to the plant.

There are many readily available dynamic (unsteady flow) hydraulic models that have been used for dam breach outflow hydrograph computation and downstream routing. Recent case studies of dam-break flood routing using hydraulic models developed by Federal agencies are available in the hydraulic engineering literature. Models to route the flood can be one- or two-dimensional or can be a combination of both. In general, as the flood plain widens, one-dimensional analysis becomes less reliable. Accurate estimates of flood elevation in areas of changing topography and near large objects (i.e. buildings and other structures) in the flow field will typically require localized two-dimensional analysis, in areas of particular interest or sensitivity.

Staff Positions:

- For estimating inundation at or near a NPP site, two-dimensional models are generally preferred by the NRC staff. However, use of one-dimensional models may be appropriate in some cases. Therefore, use of one-dimensional models will be accepted on a case-by-case basis, with appropriate justification.
- Large uncertainty exists in relationships between water elevation and discharge (rating curves), especially at high river discharges. Typically, observed data are extrapolated well beyond field-observed data when discussing dam breach

scenarios. Some estimation of the likely variation in maximum water surface elevation at a NPP site should be reported to account for this uncertainty in the rating curve.

9.3 Sediment Transport Modeling

Sediment transport effects such as erosion and sedimentation should be considered. Ignoring sediment deposition on or near the site may result in underestimates of water level elevations. Conversely, ignoring sediment erosion may mean that potentially dangerous scouring around structures is not analyzed. However, detailed guidance on sediment transport modeling is beyond the scope of this ISG. The reader should consult one or more standard references in this discipline (Yalin 1977, Julien 2010, Lick 2008). Some hydrologic and hydraulic modeling packages include sediment transport modules (e.g., HEC-RAS, SRH-1D, SRH-2D, FLO-2D). In many cases, simplified conservative estimates for sediment transport, erosion and sedimentation may be used in place of detailed analysis.

Staff Position:

Transport of sediment and debris by the flood waters should be considered.

9.4 Inundation Mapping

Inundation maps have a variety of uses including EAP's, mitigation planning, emergency response, and consequence assessment. Each map application has unique information requirements tailored to a specific end need. For the purpose of this ISG, inundation maps provide assistance in identifying SSCs important to safety that may require protective and/or mitigation measures from flooding due to dam breach.

The use of Geographic Information Systems (GIS) has emerged as the most common method to develop mapping products associated with hydrologic/ hydraulic engineering, including inundation maps from dam breach modeling. Several software packages are commercially available and some are free of charge. It should be noted that the NRC does not endorse a particular software package.

Several Federal agencies have developed guidance documents on producing inundation maps (USDOI, 2010; USACE, 2011d). These documents provide guidance based on their specific mission and are mentioned for information only.

Staff Positions:

- Inundation map(s) should be developed for NPP sites that are flooded because of dam failure The inundation map(s) should reflect the bounding flood scenario (greatest depth of inundation at the NPP site) from the dam breach analysis. The inundation map(s) should contain the following features:
    - Topographic elevation / contour information related to a stated datum.
    - Streets and highways bordering and within the NPP site
    - SSCs important to safety and other key structures and landmarks.
    - Orthophotographic imagery of the NPP site and nearby areas.
    - Cross sections / Grid (if applicable) from hydraulic model(s)
    - Water Surface elevations for the NPP site and nearby areas related to a stated datum.
    - Local water depths across the site and nearby areas.

- Velocity data across the NPP site and nearby areas.

10. TERMS AND DEFINITIONS

Abutment. That part of the valley side against which the dam is constructed. An artificial abutment is sometimes constructed, as a concrete gravity section, to take the thrust of an arch dam where there is no suitable natural abutment. The left and right abutments of dams are defined with the observer viewing the dam looking in the downstream direction, unless otherwise indicated.

Appurtenant structure. Ancillary features of a dam such as outlets, spillways, power plants, tunnels, etc.

Attenuation. A decrease in amplitude of the seismic waves with distance caused by geometric spreading, energy absorption, and scattering, or decrease in the amplitude of a flood wave caused by channel geometry and energy loss.

Average reservoir level. The 50% exceedance duration pool level calculated using average daily water levels for the period of record.

Axis of dam. The vertical plane or curved surface, chosen by a designer, appearing as a line, in plan or in cross-section, to which the horizontal dimensions of the dam are referenced.

Base thickness. Also referred to as base width. The maximum thickness or width of the dam measured horizontally between upstream and downstream faces and normal to the axis of the dam, but excluding projections for outlets or other appurtenant structures.

Bedrock. Any sedimentary, igneous, or metamorphic material represented as a unit in geology; being a sound and solid mass, layer, or ledge of mineral matter; and with shear wave threshold velocities greater than 2500 feet/second.

Borrow area. The area from which natural materials, such as rock, gravel or soil, used for construction purposes is excavated.

Breach. An opening through a dam that allows the uncontrolled draining of a reservoir. A controlled breach is a constructed opening. An uncontrolled breach is an unintentional opening allowing discharge from the reservoir. A breach is generally associated with the partial or total failure of the dam. A breach opening could be formed by many processes.

Channel. A general term for any natural or artificial facility for conveying water.

Compaction. Mechanical action that increases the density by reducing the voids in a material.

Conduit. A closed channel to convey water through, around, or under a dam.

Construction joint. The interface between two successive placements or pours of concrete where bond, and not permanent separation, is intended.

Core. A zone of low permeability material in an embankment dam. The core is sometimes referred to as central core, inclined core, puddle clay core, rolled clay core, or impervious zone.

Core wall. A wall built of relatively impervious material, usually of concrete or asphaltic concrete in the body of an embankment dam to prevent seepage.

Crest length. The measured length of the dam along the crest or top of the dam.

Crest of dam. See top of dam.

Critical damping. The minimum amount of damping that prevents free oscillatory vibration.

Cross section. An elevation view of a dam formed by passing a plane through the dam perpendicular to the axis.

Cutoff trench. A foundation excavation to be filled later with impervious material in order to limit seepage beneath a dam.

Cutoff wall. A wall of impervious material usually of concrete, asphaltic concrete, or steel sheet piling constructed in the foundation and abutments to reduce seepage beneath and adjacent to the dam.

Cyclic mobility. A phenomenon in which a cohesionless soil loses shear strength during earthquake ground vibrations and acquires a degree of mobility sufficient to permit intermittent movement up to several feet, as contrasted to liquefaction where continuous movements of several hundred feet are possible.

Dam. An artificial barrier that has the ability to impound water, wastewater, or any liquid-borne material, for the purpose of storing or controlling the material.

> **Critical dam.** A dam (or set of dams) that is shown to have flooding impacts at a NPP site (i.e., flood elevations at or above systems, structures, and components important to safety).
>
> **Inconsequential dam.** A dam identified by federal or state agencies as having minimal or no adverse failure consequences beyond the owner's property. Also, a dam that can be shown to have minimal or no adverse downstream failure consequences.
>
> **Noncritical dam.** A dam (or set of dams) that can be shown to have no flooding impacts at a NPP site (i.e., flood elevations below systems, structures, and components.
>
> **Potentially critical dam.** A dam (or set of dams) that is under evaluation to determine if it is either a non-critical or critical dam for a NPP site (i.e., flood evaluations below systems, structures, and components important to safety).

Dam failure. Catastrophic type of failure characterized by the sudden, rapid, and uncontrolled release of impounded water or the likelihood of such an uncontrolled release. It is recognized that there are lesser degrees of failure and that any malfunction or abnormality outside the design assumptions and parameters that adversely affect a dam's primary function of impounding water is properly considered a failure. These lesser degrees of failure can progressively lead to or heighten the risk of a catastrophic failure. They are, however, normally amenable to corrective action. Dams may be classified according to the broad level of importance in estimating the flooding hazard at a NPP site:

Damping. Resistance that reduces vibrations by energy absorption. There are different types of damping such as viscous, Coulomb, and geometric damping.

Damping ratio. The ratio of the actual damping to the critical damping.

Design water level. The maximum water elevation, including the flood surcharge that a dam is designed to withstand.

Design wind. The most severe wind that is reasonably possible at a particular reservoir for generating wind setup and run-up. The determination will generally include the results of meteorologic studies that combine wind velocity, duration, direction and seasonal distribution characteristics in a realistic manner.

Dike. See saddle dam.

Drain, blanket. A layer of pervious material placed to facilitate drainage of the foundation and/or embankment.

Drain, chimney. A vertical or inclined layer of pervious material in an embankment to facilitate and control drainage of the embankment fill.

Drain, toe. A system of pipe and/or pervious material along the downstream toe of a dam used to collect seepage from the foundation and embankment and convey it to a free outlet.

Drainage area. The area that drains to a particular point on a river or stream.

Drainage curtain. A line of vertical wells or boreholes to facilitate drainage of the foundation and abutments and to reduce water pressure.

Drainage wells or relief wells. Vertical wells downstream of or in the downstream shell of an embankment dam to collect and control seepage through and under the dam. A line of such wells forms a drainage curtain.

Duration of strong ground motion. The "bracketed duration" or the time interval between the first and last acceleration peaks that are equal to or greater than 0.05g.

Dynamic routing. Hydraulic flow routing based on the solution of the St.-Venant Equation(s) to compute the changes of discharge and stage with respect to time at various locations along a stream.

Earthquake. A sudden motion or trembling in the earth caused by the abrupt release of accumulated stress along a fault.

Emergency gate. A standby or reserve gate used only when the normal means of water control is not available for use.

Energy dissipator. A device constructed in a waterway to reduce the kinetic energy of fast flowing water.

Epicenter. The point on the earth's surface located directly above the point where the first rupture and the first earthquake motion occur.

Erosion. The wearing away of a surface (bank, streambed, embankment, or other surface) by floods, waves, wind, or any other natural process.

Failure. See Dam, Failure.

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

Fetch. The-straight-line distance across a body of water subject to wind forces. The fetch is one of the factors used in calculating wave heights in a reservoir.

Filter (filter zone). One or more layers of granular material graded (either naturally or by selection) so as to allow seepage through or within the layers while preventing the migration of material from adjacent zones.

Flashboards. Structural members of timber, concrete, or steel placed in channels or on the crest of a spillway to raise the reservoir water level but intended to be quickly removed, tripped, or fail in the event of a flood.

Flood. A temporary rise in water surface elevation resulting in inundation of areas not normally covered by water. Hypothetical floods may be expressed in terms of average probability of exceedance per year such as one-percent-chance-flood, or expressed as a fraction of the probable maximum flood or other reference flood.

Flood, Inflow Design (IDF). The flood flow above which the incremental increase in downstream water surface elevation due to failure of a dam or other water impounding structure is no longer considered to present an unacceptable threat to downstream life or property. The flood hydrograph used in the design of a dam and its appurtenant works particularly for sizing the spillway and outlet works and for determining maximum storage, height of dam, and freeboard requirements.

Flood, Probable Maximum (PMF). The flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the drainage basin under study.

Flood plain. An area adjoining a body of water or natural stream that may be covered by floodwater. Also, the downstream area that would be inundated or otherwise affected by the failure of a dam or by large flood flows. The area of the flood plain is generally delineated by a frequency (or size) of flood.

Flood routing. A process of determining progressively over time the amplitude of a flood wave as it moves past a dam or downstream to successive points along a river or stream.

Flood storage*.* The retention of water or delay of runoff either by planned operation, as in a reservoir, or by temporary filling of overflow areas, as in the progression of a flood wave through a natural stream channel.

Flume. An open channel constructed with masonry, concrete or steel of rectangular or U shaped cross section and designed for medium or high velocity flow. Also, a channel in which water is conveyed through a section of known size and shape for purposes of measurement.

Foundation. The portion of the valley floor that underlies and supports the dam structure.

Freeboard. Vertical distance between the surface elevation of specified stillwater (or other) reservoir and the top of the dam, without camber.

Gallery. A passageway in the body of a dam used for inspection, foundation grouting, and/or drainage.

Gate. A movable water barrier for the control of water.

Bascule gate. See flap gate.

Bulkhead gate. A gate used either for temporary closure of a channel or conduit before dewatering it for inspection or maintenance or for closure against flowing water when the head difference is small, e.g., for diversion tunnel closure.

Crest gate (spillway gate). A gate on the crest of a spillway to control the discharge or reservoir water level.

Drum gate (roller drum gate). . A type of spillway gate consisting of a long hollow drum. The drum may be held in its raised position by the water pressure in a flotation chamber beneath the drum.

Emergency gate. A standby or auxiliary gate used when the normal means of water control is not available. Sometimes referred to as guard gate.

Fixed wheel gate (fixed roller gate) (fixed axle gate). A gate having wheels or rollers mounted on the end posts of the gate. The wheels bear against rails fixed in side grooves or gate guides.

Flap gate. A gate hinged along one edge, usually either the top or bottom edge. Examples of bottom-hinged flap gates are tilting gates and fish belly gates so called from their shape in cross section.

Flood gate. A gate to control flood release from a reservoir.

Outlet gate. A gate controlling the flow of water through a reservoir outlet.

Radial gate (Tainter gate). A gate with a curved upstream plate and radial arms hinged to piers or other supporting structures.

Regulating gate (regulating valve). A gate or valve that operates under full pressure flow conditions to regulate the rate of discharge.

Roller drum gate. See drum gate.

Roller gate (stoney gate). A gate for large openings that bears on a train of rollers in each gate guide.

Skimmer gate. A gate at the spillway crest whose prime purpose is to control the release of debris and logs with a limited amount of water. It is usually a bottom hinged flap or Bascule gate.

Slide gate (sluice gate). A gate that can be opened or closed by sliding in supporting guides.

Gate chamber (valve chamber). A room from which a gate or valve can be operated, or sometimes in which the gate is located.

Hazard. A situation that creates the potential for adverse consequences such as loss of life, property damage, or other adverse impacts.

Hazard potential. The possible adverse incremental consequences that result from the

release of water or stored contents due to failure of the dam or misoperation of the dam or appurtenances. Impacts may be for a defined area downstream of a dam from floodwaters released through spillways and outlet works of the dam or waters released by partial or complete failure of the dam. There may also be impacts for an area upstream of the dam from effects of backwater flooding or landslides around the reservoir perimeter.

Head, static. The vertical distance between two points in a fluid.

Head, velocity. The vertical distance that would statically result from the velocity of a moving fluid.

Headwater: The water immediately upstream from a dam. The water surface elevation varies due to fluctuations in inflow and the amount of water passed through the dam.

Heel. The junction of the upstream face of a gravity or arch dam with the ground surface. For an embankment dam, the junction is referred to as the upstream toe of the dam.

Height, above ground. The maximum height from natural ground surface to the top of a dam.

Height, hydraulic. The vertical difference between the maximum design water level and the lowest point in the original streambed.

Height, structural. The vertical distance between the lowest point of the excavated foundation to the top of the dam.

Hydrograph, breach or dam failure. A flood hydrograph resulting from a dam breach.

Hydrograph, flood. A graph showing, for a given point on a stream, the discharge, height, or other characteristic of a flood with respect to time.

Hydrograph, unit. A hydrograph with a volume of one inch of runoff resulting from a storm of a specified duration and areal distribution. Hydrographs from other storms of the same duration and distribution are assumed to have the same time base but with ordinates of flow in proportion to the runoff volumes.

Hydrology*.* One of the earth sciences that encompasses the natural occurrence, distribution, movement, and properties of the waters of the earth and their environmental relationships.

Hydrometeorology. The study of the atmospheric and land-surface phases of the hydrologic cycle with emphasis on the interrelationships involved.

Hypocenter. The location where the slip responsible for an earthquake originates; the focus of an earthquake.

Inflow Design Flood (IDF). See Flood.

Intake. A component placed at the beginning of an outlet-works waterway (power conduit, water supply conduit), the intake establishes the ultimate drawdown level of the reservoir by the position and size of its opening(s) to the outlet works. The intake may be vertical or inclined towers, drop inlets, or submerged, box-shaped structures. Intake elevations are determined by the head needed for discharge capacity, storage reservation to allow for siltation, the required amount and rate of withdrawal, and the desired extreme drawdown level.

Landslide. The unplanned descent (movement) of a mass of earth or rock down a slope.

Leakage. Uncontrolled loss of water by flow through a hole or crack.

Length of dam. The length along the top of the dam. This also includes the spillway, power plant, navigation lock, fish pass, etc., where these form part of the length of the dam. If detached from the dam, these structures should not be included.

Lining. With reference to a canal, tunnel, shaft, or reservoir, a coating of asphaltic concrete, reinforced or unreinforced concrete, shotcrete, rubber or plastic to provide water tightness, prevent erosion, reduce friction, or support the periphery of the outlet pipe conduit.

Liquefaction. A condition whereby soil undergoes continued deformation at a constant low residual stress or with low residual resistance, due to the buildup and maintenance of high pore water pressures, which reduces the effective confining pressure to a very low value. Pore pressure buildup leading to liquefaction may be due either to static or cyclic stress applications and the possibility of its occurrence will depend on the void ratio or relative density of a cohesionless soil and the confining pressure.

Logboom. A chain of logs, drums, or pontoons secured end-to-end and floating on the surface of a reservoir to divert floating debris, trash, and logs.

Low-level outlet (bottom outlet). An opening at a low level from a reservoir generally used for emptying or for scouring sediment and sometimes for irrigation releases.

Maximum flood control level. The highest elevation of the flood control storage.

Maximum wind. The most severe wind for generating waves that is reasonably possible at a particular reservoir. The determination will generally include results of meteorologic studies that combine wind velocity, duration, direction, fetch, and seasonal distribution characteristics in a realistic manner.

Meteorological homogeneity. Climates and orographic influences that are alike or similar.

Meteorology. The science that deals with the atmosphere and atmospheric phenomena, the study of weather, particularly storms and the rainfall they produce.

Minimum operating level. The lowest level to which the reservoir is drawn down under normal operating conditions. The lower limit of active storage.

Non-overflow dam (section). A dam or section of dam that is not designed to be overtopped.

Orographic. Physical geography that pertains to mountains and to features directly connected with mountains and their general effect on storm path and generation of rainfall.

Outlet. An opening through which water can be freely discharged from a reservoir to the river for a particular purpose.

Outlet works. A dam appurtenance that provides release of water (generally controlled) from a reservoir.

Overflow dam (section). A section or portion of a dam designed to be overtopped.

Parapet wall. A solid wall built along the top of a dam (upstream or downstream edge) used for ornamentation, for safety of vehicles and pedestrians, or to prevent overtopping caused by wave run-up.

Peak flow. The maximum instantaneous discharge that occurs during a flood. It is coincident with the peak of a flood hydrograph.

Pervious zone. A part of the cross section of an embankment dam comprising material of high permeability.

Phreatic surface. The free surface of water seeping at atmospheric pressure through soil or rock.

Piezometer. An instrument used for measure water levels or pore water pressures in embankments, foundations, abutments, soil, rock, or concrete.

Piping. The progressive development of internal erosion by seepage.

Probability. The likelihood of an event occurring.

Probable. Likely to occur; reasonably expected; realistic.

Probable Maximum Flood (PMF). See Flood.

Probable Maximum Precipitation (PMP). Theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location during a certain time of the year.

Reservoir. A body of water impounded by a dam for storage.

Reservoir regulation procedure (Rule Curve): The compilation of operating criteria, guidelines, and specifications that govern the storage and release function of a reservoir. It may also be referred to as operating rules, flood control diagram, or water control schedule. These are usually expressed in the form of graphs and tabulations, supplemented by concise specifications and are often incorporated in computer programs. In general, they indicate limiting rates of reservoir releases required or allowed during various seasons of the year to meet all functional objectives of the project.

Reservoir rim. The boundary of the reservoir including all areas along the valley sides above and below the water surface elevation associated with the routing of the IDF.

Reservoir surface area. The area covered by a reservoir when filled to a specified level.

Response spectrum. A plot of the maximum values of acceleration, velocity, and/or displacement response of an infinite series of single-degree-of-freedom systems subjected to a time-history of earthquake ground motion. The maximum response values are expressed as a function of natural period for a given damping.

Riprap. A covering layer of large chunks of rock or concrete used to protect abutments, shorelines, and other vulnerable surfaces from erosion or scour.

Scaling. An adjustment to an earthquake time-history or response spectrum where the amplitude of acceleration, velocity, and/or displacement is increased or decreased, usually without change to the frequency content of the ground motion.

Seepage. The internal movement of water that may take place through the dam, the foundation or the abutments.

Seiche. An oscillating wave in a reservoir caused by a landslide into the reservoir or earthquake-induced ground accelerations or fault offset or meteorological event.

Sensitivity analysis. An analysis in which the relative importance of one or more of the variables thought to have an influence on the phenomenon under consideration is determined.

Settlement. The vertical downward movement of a structure or its foundation.

Slope. Inclination from the horizontal. Sometimes referred to as batter when measured from vertical.

Slope protection. The protection of a slope against wave action or erosion.

Sluice. An opening for releasing water from below the static head elevation.

Smooth response spectrum. A response spectrum devoid of sharp peaks and valleys that specifies the amplitude of the spectral acceleration, velocity, and/or displacement to be used in the analyses of the structure.

Spillway. A structure over or through which flow is discharged from a reservoir. If the rate of flow is controlled by mechanical means, such as gates, it is considered a controlled spillway. If the geometry of the spillway is the only control, it is considered an uncontrolled spillway.

> **Spillway, auxiliary (spillway, emergency).** Any secondary spillway that is designed to be operated infrequently, possibly in anticipation of some degree of structural damage or erosion to the spillway that would occur during operation.
>
> **Spillway, emergency.** See Spillway, auxiliary.
>
> **Spillway, service (spillway, principal).** A spillway that is designed to provide continuous or frequent regulated or unregulated releases from a reservoir, without significant damage to either the dam or its appurtenant structures. This is also referred to as principal spillway.

Spillway capacity: The maximum spillway outflow that a dam can safely pass with the reservoir at its maximum level.

Spillway channel. An open channel or closed conduit conveying water from the spillway inlet downstream.

Spillway chute. A steeply sloping spillway channel that conveys discharges at supercritical velocities.

Spillway crest. The lowest level at which water can flow over or through the spillway.

Spillway Design Flood. See Flood, Inflow Design.

Spillway, fuse plug. A form of auxiliary spillway with an inlet controlled by a low embankment designed to be overtopped and washed away during an exceptionally large flood.

Spillway, shaft. A vertical or inclined shaft into which water spills and then is conveyed through, under, or around a dam by means of a conduit or tunnel. If the upper part of the shaft is splayed out and terminates in a circular horizontal weir, it is termed a bellmouth or morning glory spillway.

Stability. The condition of a structure or a mass of material when it is able to support the applied stress for a long time without suffering any significant deformation or movement that is not reversed by the release of the stress.

Stilling basin. A basin constructed to dissipate the energy of rapidly flowing water, e.g., from a spillway or outlet, and to protect the riverbed from erosion.

Stillwater level. The elevation that a water surface would assume if all wave actions were absent.

Stoplogs. Large logs, timbers, or steel beams placed on top of each other with their ends held in guides on each side of a channel or conduit to provide a cheaper or more easily handled means of temporary closure than a bulkhead gate.

Storage. The retention of water or delay of runoff by planned operation, as in a reservoir, or by temporary filling of overflow areas, as in the progression of a flood wave through a natural stream channel. Definitions of specific types of storage in reservoirs are:

> **Active storage.** The volume of the reservoir that is available for some use such as power generation, irrigation, flood control, water supply, etc. The bottom elevation is the minimum operating level. The top elevation is the maximum normal operating level.
>
> **Dead storage.** The storage that lies below the invert of the lowest outlet and that, therefore, cannot readily be withdrawn from the reservoir.
>
> **Flood surcharge.** The storage volume between the top of the active storage and the design water level.
>
> **Inactive storage.** The storage volume of a reservoir between the crest of the invert of the lowest outlet and the minimum operating level.
>
> **Live storage.** The sum of the active-and the inactive storage.
>
> **Reservoir capacity.** The sum of the dead and live storage of the reservoir.

Surcharge. The volume or space in a reservoir between the controlled retention water level and the maximum water level. Flood surcharge cannot be retained in the reservoir but will flow out of the reservoir until the controlled retention water level is reached.

Surface waves. Waves that travel along or near the surface and include Rayleigh (Sv) and Love (SH) Waves of an earthquake.

Tailwater. The water immediately downstream from a dam. The water surface elevation varies due to fluctuations in the outflow from the structures of a dam and due to downstream influences of other dams or structures. Tailwater monitoring is an important consideration because a failure of a dam will cause a rapid rise in the level of the tailwater.

Toe of the dam. The junction of the downstream slope or face of a dam with the ground surface; also referred to as the downstream toe. The junction of the upstream slope with ground surface is called the heel or the upstream toe.

Topographic map. A detailed graphic delineation (representation) of natural and manmade features of a region with particular emphasis on relative position and elevation.

Top thickness (top width). The thickness or width of a dam at the level of the top of dam (excluding corbels or parapets). In general, the term thickness is used for gravity and arch dams, and width is used for other dams.

Trashrack. A device located at an intake to prevent floating or submerged debris from entering the intake.

Tributary. A stream that flows into a larger stream or body of water

Unit Hydrograph. See Hydrograph, unit.

Valve. A device fitted to a pipeline or orifice in which the closure member is either rotated or moved transversely or longitudinally in the waterway to control or stop the flow.

Watershed. The area drained by a river or river system or portion thereof. The watershed for a dam is the drainage area upstream of the dam.

Watershed divide. The divide or boundary between catchment areas (or drainage areas).

Wave protection. Riprap, concrete, or other armoring on the upstream face of an embankment dam to protect against scouring or erosion due to wave action.

Wave run-up. Vertical height above the stillwater level to which water from a specific wave will run up the face of a structure or embankment.

Weir. A notch of regular form through which water flows.

> **Weir, broad-crested.** An overflow structure on which the nappe is supported for an appreciable length in the direction of flow.
>
> **Weir, sharp-crested.** A device for measuring the rate of flow of water. It generally consists of a rectangular, trapezoidal, triangular, or other shaped notch, located in a vertical, thin plate over which water flows. The height of water above the weir crest is used to determine the rate of flow.

Wind setup. The vertical rise in the stillwater level at the face of a structure or embankment caused by wind stresses on the surface of the water.

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