

# U.S. NUCLEAR REGULATORY COMMISSION

## REGULATORY GUIDE 1.256, REVISION 0



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Technical Lead: Joseph Kanney

## GUIDANCE FOR ASSESSMENT OF FLOODING HAZARDS DUE TO WATER CONTROL STRUCTURE FAILURES AND INCIDENTS

### A. INTRODUCTION

#### Purpose

This regulatory guide (RG) provides guidance to applicants for new nuclear power plants (NPPs) on acceptable methods for evaluating design-basis flooding hazards due to failure or other incidents at manmade water control structures including, but not limited to, dams and levees.

#### Applicability

This RG applies to applicants for NPPs subject to Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, “Domestic Licensing of Production and Utilization Facilities”; 10 CFR Part 52, “Licenses, Certifications, and Approvals for Nuclear Power Plants”; and 10 CFR Part 100, “Reactor Site Criteria.” Though the guidance primarily reflects past reviews of large light water nuclear power plant applications, this RG may also provide useful guidance to other types of power reactors (i.e., small Light-Water Reactors (LWRs), non-LWRs, and micro-reactors).

#### Applicable Regulations

- 10 CFR Part 50 provides for the licensing of production and utilization facilities.
  - 10 CFR 50.34, “Contents of applications; technical information,” provides the requirements for the content of applications submitted under 10 CFR Part 50. Under its provisions, an application for a construction permit must include the principal design criteria for a proposed facility.
  - 10 CFR Part 50, Appendix A, “General Design Criteria for Nuclear Power Plants,” provides minimum requirements for the principal design criteria that establish the necessary design, fabrication, construction, testing, and performance requirements for structures, systems, and components (SSCs) important to safety to provide reasonable assurance that the facility can

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Electronic copies of this RG, previous versions of DGs and RGs are available through the NRC’s public website in the NRC Library at <https://www.nrc.gov/reading-rm/doc-collections/reg-guides/index.html> under Document Collections, in Regulatory Guides. This RG is also available through the NRC’s Agencywide Documents Access and Management System (ADAMS) at <https://www.nrc.gov/reading-rm/adams.html>, under ADAMS Accession Number (No.) ML25129A075. The regulatory analysis may be found in ADAMS under Accession No. ML22278A111. The associated draft guide DG-1417 may be found in ADAMS under Accession No. ML22278A110, and the staff responses to the public comments on DG-1417 may be found under ADAMS Accession No. ML26258A151.

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- be operated without undue risk to the health and safety of the public. The general design criteria (GDC) applicable to this RG include the following:
  - GDC 2, “Design Bases for Protection against Natural Phenomena,” requires, in part, that SSCs important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches, with the appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena without loss of capability to perform their safety function.
- 10 CFR Part 52 governs the issuance of early site permits (ESPs), standard design certifications (DCs), combined licenses (COLs), standard design approvals, and manufacturing licenses. It contains application requirements similar to the requirements in 10 CFR Part 50.
  - 10 CFR Part 52, Subpart A, “Early Site Permits,” Section 52.17, “Contents of applications; technical information,” requires that the site safety analysis report include the following:
 

...seismic, meteorological, hydrologic, and geologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
  - 10 CFR Part 52, Subpart B, “Standard Design Certifications,” Section 52.47, “Contents of applications; technical information,” references the GDC of 10 CFR Part 50, Appendix A.
  - 10 CFR Part 52, Subpart C, “Combined Licenses,” Section 52.79, “Contents of applications; technical information in final safety analysis report,” contains the same requirements as 10 CFR 52.17.
  - 10 CFR Part 52, Subpart E, “Standard Design Approvals,” Section 52.137, “Contents of applications; technical information,” references the GDC of 10 CFR Part 50 Appendix A.
  - 10 CFR Part 52, Subpart F, “Manufacturing Licenses,” Section 52.157, “Contents of applications; technical information in final safety analysis report,” references the GDC of 10 CFR Part 50, Appendix A.
- 10 CFR Part 100 establishes approval requirements for proposed sites for stationary power and test reactors subject to 10 CFR Part 50 or 10 CFR Part 52.
  - 10 CFR 100.20, “Factors to be considered when evaluating sites,” requires that physical characteristics of the site, including seismology, meteorology, geology, and hydrology, be considered when determining the suitability of a site for a nuclear power reactor.
  - 10 CFR 100.23, “Geologic and seismic siting criteria,” requires that the potential for seismically induced floods and water waves be considered and incorporated into the design bases for NPPs.
  - 10 CFR Part 100, Appendix A, “Seismic and Geologic Siting Criteria for Nuclear Power Plants,” Section IV(c), requires that the potential for seismically induced floods and water waves be considered and incorporated into the design bases for NPPs.

## Related Guidance

- RG 1.59, “Design Basis Floods for Nuclear Power Plants” (USNRCb), provides an overview of technical approaches, available data sources, and analysis methods acceptable to the staff of the U.S. Nuclear Regulatory Commission (NRC) for determining design-basis floods. The guide discusses floods resulting from natural hydrometeorological, geologic, and seismic phenomena. The guide also discusses flood hazards resulting from combined events. Appendix K, “Considerations for Applying Guidance to Advanced Reactors and Small Modular Reactors,” discusses general considerations related to evaluating design-basis flooding hazards for advanced reactor and microreactor designs which, by virtue of their unique engineering features, may have SSCs important to safety that are unaffected by exposure to external flood waters. These hazards include failure or other incidents at manmade water control structures including, but not limited to, dams and levees that are the subject of this RG.
- RG 1.70, “Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants (LWR Edition)” (USNRCh), provides detailed guidance for preparing applications for construction permits and operating licenses for new NPPs submitted to the NRC under 10 CFR Part 50. It provides general guidance on the hydrologic setting information and flooding hazard assessments that should be included.
- RG 1.102, “Flood Protection for Nuclear Power Plants” (USNRCc), describes the types of flood protection that the NRC staff finds acceptable for the SSCs important to safety.
- RG 1.198, “Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites” (USNRCg), provides guidance to license applicants on acceptable methods for evaluating the potential for earthquake-induced instability of soils resulting from liquefaction and strength degradation. It discusses conditions under which safety analysis reports should address the potential for such response. The guidance includes procedures and criteria currently applied to assess the liquefaction potential of soils ranging from gravel to clays.
- RG 1.201, “Guidelines for Categorizing Structures, Systems, and Components in Nuclear Power Plants According to Their Safety Significance” (USNRCe), describes a risk-informed process for categorizing SSCs according to their safety significance. This process can remove SSCs of low safety significance from the scope of certain identified special treatment requirements.
- RG 1.206, “Applications for Nuclear Power Plants” (USNRCa), provides guidance on the format and content of applications for light-water reactor NPPs submitted to the NRC under 10 CFR Part 52 with general applicability to other types of power reactors. The NRC staff considers this guidance acceptable to support preparation of applications for ESPs, standard DCs, and COLs and generally acceptable to support its review of other types of applications under 10 CFR Part 52.
- RG 1.208, “A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion” (USNRCf), provides guidance on the development of the site-specific ground motion response spectrum. This response spectrum represents the first part of the development of the safe-shutdown earthquake ground motion for a site as a characterization of the regional and local seismic hazard. This RG provides an alternative for use in satisfying the requirements in 10 CFR 100.23.

- RG 4.7, “General Site Suitability Criteria for Nuclear Power Stations” (USNRCd), assists applicants in the initial stage of selecting potential sites for a nuclear power station. The safety issues discussed include geological, seismic, hydrological, and meteorological characteristics of proposed sites as they relate to protecting the general public from the potential hazards of serious accidents.
- NUREG-0800, “Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition” (USNRCi), provides guidance to the NRC staff in performing safety reviews under 10 CFR Part 50 and 10 CFR Part 52. Chapter 2, “Site Characteristics and Site Parameters,” contains general review guidance related to site characteristics and site parameters, together with site-related design parameters and design characteristics, as applicable.

### **Purpose of Regulatory Guides**

The NRC issues RGs to describe methods that are acceptable to the staff for implementing specific parts of the agency’s regulations, to explain techniques that the staff uses in evaluating specific problems or postulated events, and to describe information that the staff needs in its review of applications for permits and licenses. Regulatory guides are not substitutes for regulations and compliance with them is not required. Methods and solutions that differ from those set forth in RGs are acceptable if the applicant provides sufficient basis and information for the NRC staff to verify that the alternative methods comply with the applicable NRC regulations.

### **Paperwork Reduction Act**

This RG provides voluntary guidance for implementing the mandatory information collections in 10 CFR Parts 50, 52, and 100 that are subject to the Paperwork Reduction Act of 1995 (44 U.S.C. 3501 et. seq.). These information collections were approved by the Office of Management and Budget (OMB), under control number 3150-0011, 3150-0151, and 3150-0093, respectively. Send comments regarding this information collections to [Infocollects.Resource@nrc.gov](mailto:Infocollects.Resource@nrc.gov), and to the OMB reviewer at: OMB Office of Information and Regulatory Affairs (3150-0011, 3150-0151, and 3150-0093), Attn: Desk Officer for the Nuclear Regulatory Commission, 725 17<sup>th</sup> Street, NW, Washington, DC, 20503.

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## **B. DISCUSSION**

### **B1. Reason for Issuance**

This RG is being issued to formally incorporate interim staff guidance (ISG) “Guidance for Assessment of Flooding Hazards Due to Dam Failure,” dated July 29, 2013 (JLD-ISG-2013-01, NRC 2013) into the NRC’s regulatory framework. JLD-ISG-2013-01 was prepared to aid completion of flood hazard reevaluations performed by NPP licensees in response to the NRC’s 10 CFR 50.54(f) information request issued following the 2011 Fukushima Dai-ichi accident (NRC 2012). The flood hazard reevaluations were to be performed using analysis methods current in 2012 and guidance used by the NRC staff for reviewing external flooding analyses for ESP and COL applications submitted under 10 CFR Part 52. The site-specific flooding reevaluations performed by affected owners and operators in response to the 2011 Fukushima event are publicly available in ADAMS. ISGs were prepared to clarify or address issues not fully discussed in the NRC’s Standard Review Plan (NUREG-0800), which was to be used as the basic reference for reviewing the post-Fukushima flood hazard reevaluations. ISGs are meant to be withdrawn after their immediate purpose has been fulfilled or else integrated formally into the NRC’s regulatory guidance framework.

The guidance in this document is meant to be generally consistent with guidelines developed by other Federal and State agencies that regulate, operate, build, or own dams and levees, or have roles in emergency planning and response for failures and incidents. Therefore, this RG draws from guidelines developed by the Federal Emergency Management Agency (FEMA), Federal Energy Regulatory Commission (FERC), the U.S. Bureau of Reclamation (USBR), the U.S. Army Corps of Engineers (USACE), and other Federal agencies. Some portions of this guidance draw from dam safety guidelines developed by States, including California, Colorado, and Washington.

Although this guide is broadly consistent with the Federal and State guidelines discussed above, there may be differences. Guidelines are not uniform across all agencies. In some cases, variance between this RG and other agencies’ guidelines is due to differences in risk tolerance levels between the nuclear power sector and sectors such as water resources and flood control.

This RG complements RG 1.59, which provides guidance on assessment of flooding hazards other than those associated with failures or other incidents at water control structures.

### **B2. Background**

In the evaluation of flooding hazards for NPPs, floods resulting from failures or other incidents involving water control structures are among the factors considered. In this RG, “failure” refers to flooding caused by any uncontrolled release of water that threatens to impact SSCs important to safety at the NPP site. In the context of other incidents, it should be noted that even controlled releases can potentially impact SSCs at the NPP site. Examples include but are not limited to (1) releases performed to meet required operational criteria during a flood, (2) releases performed to rapidly draw down a reservoir to prevent incipient failure after a seismic event, and (3) releases performed to rapidly draw down a reservoir to prevent incipient sunny day failure.

In some cases, the elevation of the site provides the principal protection from flooding hazards. Some SSCs important to safety are protected by passive (e.g., structures) or active (e.g., equipment) flood protection features. In other cases, manual actions and procedures provide flood protection. 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 could be a potential flooding mechanism. Consideration of upstream dams should include all water-impounding structures, whether or not 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.

Failures of 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 safety-related equipment are potential flooding mechanisms. In addition, for plants that use water as an ultimate heat sink, failure of a structure that impounds the ultimate heat sink could constitute a hazard to the plant.

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

### **B3. Consideration of International Standards**

The International Atomic Energy Agency (IAEA) works with member states and other partners to promote the safe, secure, and peaceful use of nuclear technologies. The IAEA develops Safety Standards and Safety Guides for protecting people and the environment from harmful effects of ionizing radiation. This system of safety fundamentals, safety requirements, safety guides, and other relevant reports, reflects an international perspective on what constitutes a high level of safety. In developing or updating RGs, the NRC has considered IAEA Safety Standards and Safety Guides in order to benefit from international perspectives.

The basic safety principles in this RG are consistent with the following IAEA Safety Guides:

- IAEA Safety Standards Series SSR-2/1, “Safety of Nuclear Power Plants: Design,” Revision 1 (IAEA, 2016), establishes design requirements for the SSCs of an NPP, as well as for procedures and organizational processes important to safety, that are required to be met for safe operation and for preventing events that could compromise safety, or for mitigating the consequences of such events were they to occur.
- IAEA Specific Safety Requirement SSR-1, “Site Evaluation for Nuclear Installations,” (IAEA 2019), encompasses site-related factors relating to operational states and accident conditions, including those that could warrant emergency response actions, and natural and human-induced events external to the installation that are important to safety. The external human-induced events considered in this Safety Requirements publication are all of accidental origin.
- IAEA Safety Standards Series NS-G-1.5, “External Events Excluding Earthquakes in the Design of Nuclear Power Plants,” (IAEA 2008), provides guidance on design for the protection of NPPs from the effects of external events (excluding earthquakes) that are not directly involved in the operation of NPP units.
- IAEA Safety Standards Series SSG-18, “Meteorological and Hydrological Hazards in Site Evaluation for Nuclear Installations,” IAEA (2011), provides guidance on how to comply with

the safety requirements for assessing hazards associated with meteorological and hydrological phenomena. It also recommends ways to determine the corresponding design basis for these natural hazards and measures for protecting the site of a nuclear installation against hazards of this type.

The International Commission on Large Dams (ICOLD) is a nongovernmental international organization, which provides a forum for the exchange of knowledge and experience in dam engineering. ICOLD seeks to lead the profession in ensuring that dams are built safely, efficiently, economically, and without detrimental effects on the environment. Its original aim was to encourage advances in the planning, design, construction, operation, and maintenance of large dams and their associated civil works, by collecting and disseminating relevant information and by studying related technical questions. Since the late 1960s, the focus has been on subjects of current concern such as dam safety, monitoring of performance, reanalysis of older dams and spillways, effects of aging, and environmental impact. The U.S. Society on Dams (USSD) is the member organization representing the United States in ICOLD. ICOLD has published an extensive series of publications (e.g., bulletins and position papers) on dam design and dam safety topics, some of which are publicly available.

The following publicly available ICOLD publications are consistent with the basic safety principles considered in developing this RG:

- ICOLD Bulletin 170, “Flood Evaluation and Dam Safety,” (ICOLD 2019a), provides guidance for estimation of flood hydrographs, evaluation of extreme floods, and risk analysis. It also includes a summary of inflow design flood (IDF) guidelines by country.
- ICOLD Bulletin 158, “Dam Surveillance,” (ICOLD 2019b), provides guidance on monitoring systems and surveillance programs to reduce risks by early detection of an undesirable event.
- ICOLD Position Paper “Dam Safety and Earthquakes,” (ICOLD 2021), deals with the seismic safety of large dams, based on experiences with large earthquakes in Japan (Tohoku earthquake, 2011), in China (Wenchuan earthquake, 2008), and in Chile (Maule earthquake, 2010).

#### **B4. Documents Discussed in Staff Regulatory Guidance**

This RG endorses, in part, the use of one or more codes or standards developed by external organizations, and other third-party guidance documents. These codes, standards, and third-party guidance documents may contain references to other codes, standards, or third-party guidance documents (“secondary references”). If a secondary reference has itself been incorporated by reference into NRC regulations as a requirement, then licensees and applicants must comply with that standard as set forth in the regulation. If the secondary reference has been endorsed in an RG as an acceptable approach for meeting an NRC requirement, then the standard constitutes a method acceptable to the NRC staff for meeting that regulatory requirement as described in the specific RG. If the secondary reference has neither been incorporated by reference into NRC regulations nor endorsed in an RG, then the secondary reference is neither a legally binding requirement nor a “generic” NRC-approved acceptable approach for meeting an NRC requirement. However, licensees and applicants may consider and use the information in the secondary reference, if appropriately justified, consistent with current regulatory practice, and consistent with applicable NRC requirements.

## **C. STAFF REGULATORY GUIDANCE**

This section describes in detail the methods, approaches, or data that the NRC staff considers acceptable for meeting the requirements of the regulations cited in Section A that pertain to the estimation of flooding caused by dam failure.

### **C1. Framework for Dam Failure Flood Hazard Estimation**

#### **C1.1. General Considerations**

The term “dam” used in this RG should be understood to include all manmade water storage or water control structures whose failure may cause flooding at the NPP site. In general, all dams upstream of the plant site should be considered for potential failures. Dams that are not upstream of the plant but whose failure may potentially impact the plant because of backwater effects or loss of cooling water should also be considered. Water storage or water control structures (such as onsite cooling or auxiliary water reservoirs and onsite levees) that may be located at or above the grade of safety-related equipment should also be evaluated. 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 of its failure.

In engineering terms, dams and levees fail when they do not deliver the services for which they are designed (e.g., flood protection, water supply, hydropower). However, this RG defines failure from the point of view at an NPP. Therefore, this RG considers dam failure flooding to be any release of water that threatens to impact SSCs important to safety at the NPP site. This may also include controlled releases. Examples of controlled releases that might lead to inundation at the NPP site include, but are not limited to, (1) releases performed to prevent dam failure during flood conditions, (2) releases performed to rapidly draw down a reservoir to prevent incipient failure after a seismic event, and (3) releases performed to rapidly draw down a reservoir to prevent incipient sunny day failure.

An initial screening exercise may identify some dams that can be eliminated from detailed consideration because of the low differential head, the small volume of water stored, the dam’s distance from the plant site, and the major intervening natural or reservoir detention capacity. Note that the distance of a dam from the plant site may not be a sufficient exclusion criterion by itself. A sufficient distance will depend on the volume of water stored. The screening process should be quantitative and well documented.

For any dam not eliminated from consideration by the initial screening, available records should be examined to assess the likelihood of failure. The analysis of dam failure is a complex task, and many dam failures are not completely understood even after substantial forensic investigations. The principal uncertainties involve the likely initiating mode of failure, failure progression, and degree of failure. These uncertainties can be circumvented in situations where it can be shown that the complete and sudden disappearance of a dam or dams will not endanger the NPP. Otherwise, reasonable failure postulations should be used, and their consequences should be examined. The potential for multiple dam failures and the domino failure of a series of dams should be considered. Plausible permutations of cascading failures should be described, and their consequences should be evaluated.

Dam failures can be broadly categorized according to the following predominant modes of failure:

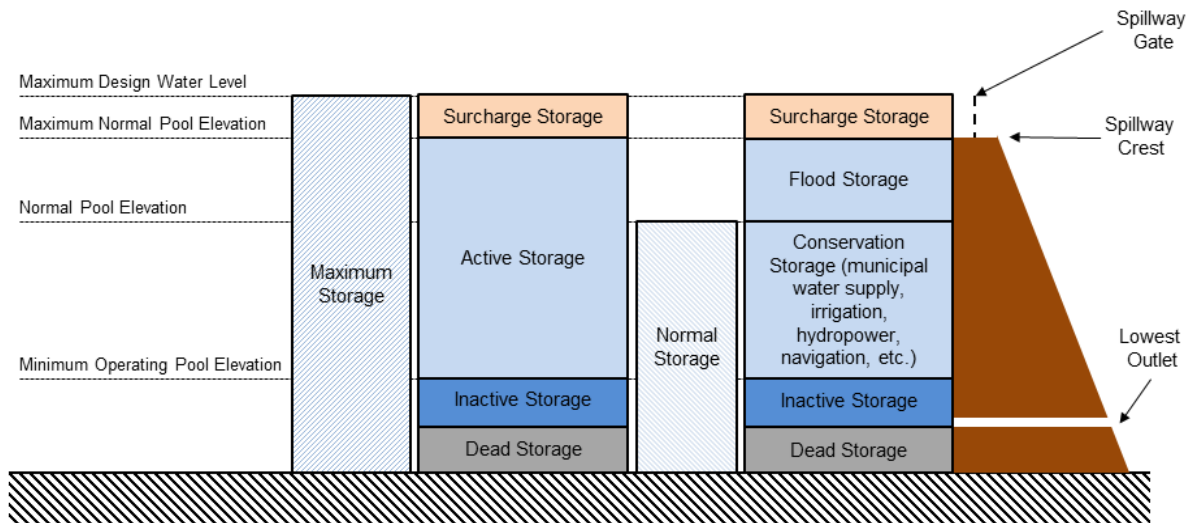
- hydrologic dam failure,
- seismic dam failure, and
- dam failure from other causes.

However, these categories are not mutually exclusive. Dam design and foundation type strongly affect predominant failure modes. Historically, the leading cause of failure for concrete arch dams and concrete gravity dams founded on rock 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 the piping of soil material from beneath the dam. Overtopping with subsequent embankment erosion is a leading cause of failure for earthen and rockfill embankment dams. Failure resulting from piping erosion is also a very significant failure mode for embankment dams. Modes and causes of failure are varied, often complex, and interrelated. For example, a seismic event might cause spillway gates to become inoperable, thus leading to overtopping and subsequent dam failure. Excessive seepage that erodes fine sandy soils may cause the slope of an embankment dam to fail. Therefore, all reasonable failure modes and scenarios (e.g., initiation, progression, and ultimate failure) should be assessed. Training materials jointly developed by the USBR and the USACE for the analysis of dam safety risks discuss the identification, description, and screening of potential failure modes in detail (USBR & USACE, 2019).

If it cannot be demonstrated that the likelihood of dam failure over the normal life of the NPP is extremely low (see section C1.4) or if the consequences of dam failure are negligible, the failure of the dam should be postulated, and the flooding consequences should be estimated. This effort will generally include estimating the reservoir outflow hydrograph (discharge hydrograph) resulting from dam failure (i.e., a 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. The transport of sediment and debris by floodwaters should be considered.

The terminology used to describe storage volumes and corresponding water levels (pool levels) in the water resources and dam safety literature vary among different agencies and practitioners. Figure 1 illustrates the terminology that has been adopted for use in this RG (see also Appendix A, “Terms and Definitions”).

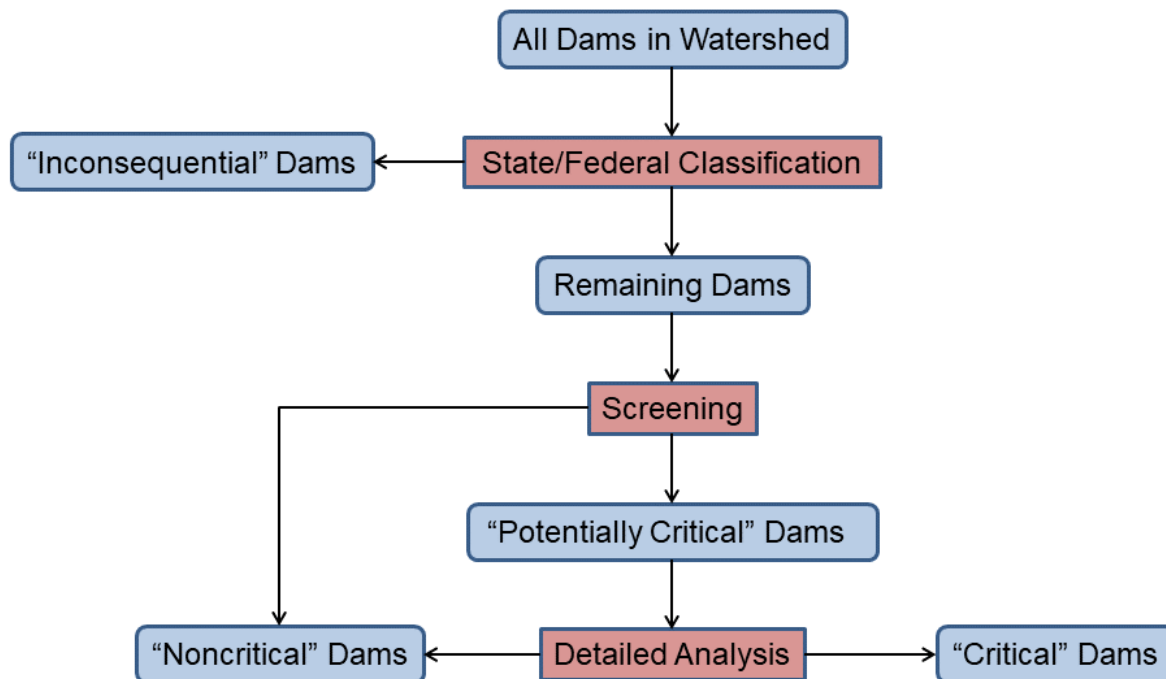
Reservoir storage consists of dead storage, inactive storage, active storage, and flood surcharge storage. Dead storage is the volume in a reservoir below the lowest controllable level. Inactive storage is the capacity below which the reservoir is not normally drawn. Active (or usable) storage is the 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 maximum normal pool elevation (or full 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 full pool elevation of the reservoir with the crest gates in the closed position).



**Figure 1. Reservoir Water Levels and Corresponding Storage Volumes**

## C1.2. Screening

Most large watersheds in the United States contain 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, many dams in the watershed will likely have no impact on flooding at an 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 C2 describes several procedures for identifying the small or distant dams whose failure would likely have negligible impacts on flooding at the NPP site. The approach identifies several classes of dams for the purposes of this guide. Dams that can be removed from consideration without analysis because they meet criteria described in section C2 (e.g., dams not owned by an NPP applicant 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 the NPP site using simplified analyses (as described in section C2) 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 the 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 an NPP site. Figure 2 illustrates the screening concept and the various dam classes and analysis levels. Section C2 presents details of the screening methods.



**Figure 2. Analysis Levels**

### **C1.3. Detailed Analysis**

For potentially critical dams (i.e., those not screened out as discussed in the preceding section), the first step in a detailed estimation of the flood hazard from dam failure is determining the demand or loading cases that will be applied to the dam failure under hydrologic loadings associated with extreme floods. Ground motions associated with earthquakes should also be considered. In addition, failure due to non-hydrologic, non-seismic causes (i.e., sunny day failures) should be considered. Sunny day failures encompass a wide variety of mechanisms (e.g., geologic or structural defects, improper operation).

Dam failure flood hazard estimation will require collecting data, to the extent it is obtainable, on the dam(s) to be analyzed (e.g., design documents, construction records, maintenance and inspection program, and 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, the regulator, or both. In the United States, there is no single entity responsible for regulation of dams. Instead, dam regulation is distributed among various Federal 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., USACE, USBR, Tennessee Valley Authority). Information on the physical characteristics and flooding history of watersheds can be obtained from federal agencies (e.g., U.S. Geological Survey [USGS]), 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 or approved by Federal, State or other agencies may be used. However, the applicant should exercise due diligence and examine the record of extreme storms and floods in the region of interest to ensure 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 the NPP site versus a very small dam or one that is very far away), availability of design/construction information, and availability of recent studies to support capacity estimates (e.g., spillway capacity ratings, seismic capacity ratings, inspection, and maintenance records). In lieu of a detailed analysis, the analyst can simply assume that the dam fails under appropriate loading and move on to estimation of the consequences.

Failures of 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 safety-related equipment are potential flooding mechanisms. In addition, flood-induced failure of a dam or levee that impounds the ultimate heat sink constitutes a hazard to the plant.

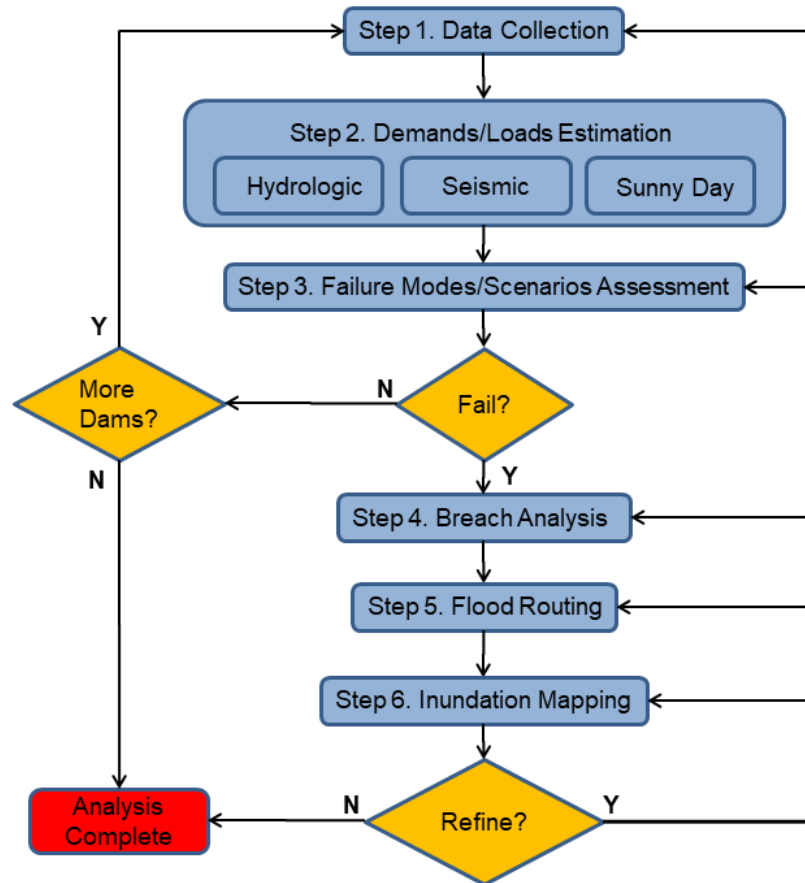
The dam failure itself may be due to flooding or some other cause such as a seismic event, a structural defect, or issues related to human performance. The potential for these mechanisms to initiate dam failure, as well as the potential failure modes, should be evaluated to fully characterize the dam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

The estimated capacities are compared to the applied loads 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 NPP is extremely low (see section C1.4) or 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 or quantitative probabilistic analysis or both. 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 RG. The USBR and the USACE have jointly developed guidance on this topic (USBR & USACE, 2019).

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 flood waters should be considered.

In summary, the detailed dam failure flood hazard analysis for potentially critical dams will comprise the following steps (see also figure 3):

1. Data collection (to the extent that it is obtainable)
  - a. Compile information on dam(s) (design, construction, inspection, maintenance, etc.).
  - b. Compile information on the river basin upstream and downstream from dam (topography, bathymetry, reservoir volumes, reservoir flood inflows, etc.).
2. Estimation of demand/loads, including the following:
  - a. flooding case,
  - b. seismic case, and
  - c. sunny day case.
3. Assess credible failure modes or scenarios under the various loading cases (flooding, seismic, sunny day), including the potential for multiple or cascading failures.
  - a. Compare loadings to estimated capacities, considering uncertainties as well as factors of safety.
  - b. For each credible failure, perform steps 4–6.
  - c. If failure is not considered credible, 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.
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 3. Detailed Dam Failure Flood Hazard Analysis Overview**

#### **C1.4. Failure Probability**

The current state of practice in dam safety analysis uses a combination of deterministic and probabilistic approaches. Probabilistic seismic hazard analysis (PSHA) 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 RG (e.g.,  $1 \times 10^{-4}$  per year or lower annual exceedance probability) exist. However, current industry consensus standards and federal guidance provide guidelines, but not detailed guidance. The NRC has established probabilistic screening criteria for man-related hazards (e.g., between  $1 \times 10^{-7}$  and  $1 \times 10^{-6}$  annual exceedance probability) that are, in theory, applicable to dam failures. However, no widely accepted methodology exists for estimating dam failure probabilities on the order of  $1 \times 10^{-7}$  to  $1 \times 10^{-6}$  annual exceedance probability.

#### **Key Point:**

- Due to lack of detailed guidance, dam failure probability estimates will be reviewed on a case-by-case basis. The expectation is that dam failure probability estimates will need to be based on a dam-specific, interdisciplinary engineering analysis, requiring detailed staff review.

#### **C1.4.1. 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, Engineering and Medicine (NASEM), 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 (NASEM, 2012). Expressed in terms of dam years, numerous studies of dam failures in the United States and worldwide have indicated an average failure rate on the order of  $10^{-4}$  per dam year (e.g., Baecher et al., 1980).

A comprehensive list of levee failures in the United States is not readily accessible. Therefore, there are no widely accepted failure rates for levees.

##### **Key Point:**

- Historical rates for dam failure provide background information about generic failure probabilities. However, each dam and its environment are unique, and failure probability estimates, if used, should be developed based on information specific to the site and dam.

#### **C1.4.2. Hydrologic Failure**

Probabilistic approaches for estimating the extreme rainfall and flood events of interest in this RG (e.g.,  $1 \times 10^{-4}$  per year or lower annual exceedance probability) exist, but there is very little industry experience with their application, and the NRC has not yet developed guidance or endorsed an industry consensus standard. Therefore, and to maintain consistency with RG 1.59, a deterministic approach based on the PMP and PMF is acceptable for the purpose of this RG. 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 C3 for additional detail on hydrologic failure.

##### **Key Points:**

- A dam should be assumed to fail due to a hydrologic hazard if it cannot withstand its basin-specific PMF, with associated effects.
- When considering hydrologic failure due to floods it should not be assumed that deterministic estimates such as PMP or PMF equate directly to a specific AEP. Extreme extrapolation of distributions such as log-Pearson and others based on limited data to assign AEPs to deterministic concepts such as PMP or PMF is seldom justified. Applicants should engage the NRC staff in pre-application discussions before using such approaches
- Probabilistic rainfall and flood hazard analyses will be reviewed on a case-by-case basis.

#### **C1.4.3. Seismic Failure**

PSHA is current state of practice 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  $1 \times 10^{-4}$  per year is routine. Widely accepted earthquake source characterization datasets, ground motion prediction equations, and site amplification factors are publicly available. Section C4 provides additional detail on analysis of seismic dam failures. The seismic capacity of dams

and other water control structures varies considerably depending on the design criteria in place at the time they were built, construction practices, and subsequent maintenance and/or retrofits. The magnitude of potential flooding events in the watershed also covers a wide range. So, although expected to be rare, seismic failure combined with flooding is credible and potential impacts of such combined flooding events should be considered. In order to simplify the analysis, two scenarios are provided below: that of failure due to a large earthquake with moderate flooding and that of failure due to a smaller earthquake with a larger flood.

#### **Key Points:**

- When considering seismic dam failure and PSHA, it is important to note that the hazard of interest to the NPP is a catastrophic failure resulting in uncontrolled release of the reservoir, not lower levels of damage that may degrade the services that the dam provides. It is also recognized that the seismic design of dams typically includes significant margins and factors of safety. To account for this level of margin before failure, it is acceptable to use the  $1 \times 10^{-4}$  annual frequency ground motions, at spectral frequencies important to the dam, for seismic evaluation of dams, instead of  $1 \times 10^{-6}$ , as discussed above. However, appropriate engineering justification should be provided to show that the dam has sufficient seismic margin. Otherwise, the  $1 \times 10^{-6}$  ground motions should be used.
- 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  $1 \times 10^{-4}$  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  $1 \times 10^{-4}$  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, or
  - half of the  $10^{-4}$  ground motion, combined with a 500-year flood.

#### **C1.4.4. 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 can be considered a predominantly man-related hazard.

Sections 2.2.1–2.2.3 of NUREG-0800 describes the NRC's approach to assessing impacts from manmade hazards (such as dams). The NRC considers design-basis events resulting from the presence of hazardous materials or activities in the vicinity of the plant to be acceptable based on estimated annual frequency. If a postulated accident type meets the NRC staff objective (order of magnitude of  $1 \times 10^{-7}$  per year), then the potential exposures are considered to meet the requirements of 10 CFR 50.34(a)(1) as it relates to the requirements of 10 CFR Part 100.

When data are not available to confirm that the NRC staff objective has been met, a higher calculated event threshold ( $1 \times 10^{-6}$  per year) is acceptable when combined with reasonable qualitative evidence that the best estimate is lower than  $1 \times 10^{-6}$ . This exception is made since data are often not available to enable the accurate calculation of probabilities because of 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 in the introductory paragraph of this section, current engineering practice has no widely accepted method for estimating dam or levee failure rates on the order of  $1 \times 10^{-6}$  per year.

**Key Point:**

- Due to lack of detailed guidance, dam failure probability estimates will be reviewed on a case-by-case basis. The expectation is that dam failure probability estimates will need to be based on a dam-specific, interdisciplinary engineering analysis, requiring detailed staff review.

**C1.4.5. Dam Safety Governance**

There are over 90,000 dams (USACE, 2022a) and over 100,000 miles of levees (National Committee on Levee Safety (NCLS), 2009) in the United States, constructed by a variety of public sector agencies (local, State, and Federal), as well as many private sector entities (e.g., individuals, groups, and corporations). Shaped by laws, policies, and practice, dam and levee safety program governance in the United States is like the governance that has evolved for emergency response in this country (NASSEM, 2012).

Most of the direct responsibility for dam safety governance is in the hands of local and State governments. Almost all States have formal dam safety programs tied to federal guidelines. FEMA has published a summary of individual State dam safety programs (FEMA, 2012).

Federal regulatory authority for non-federal dams is limited to the roughly 2,100 dams that are part of hydropower projects regulated by 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 some hydropower dams).

Federally owned dams are regulated according to the policies and guidance of the individual federal agencies that own or operate the dams. Table 1 summarizes federal dam ownership and dam safety roles.

**C1.4.6. Other Agencies' Dam Safety Guidance**

At the federal level, FEMA has been charged with encouraging the establishment and maintenance of effective Federal and State programs, policies, and guidelines to enhance dam safety and security. It implements this charge through leadership of the National Dam Safety Program, the National Dam Safety Review Board, and the Interagency Committee on Dam Safety (ICODS). ICODS serves as the permanent forum for the coordination of federal activities in dam safety and security. FEMA chairs this committee. ICODS has prepared and approved a series of federal guidelines for federal agency dam owners and regulators (e.g., FEMA, 2004a; 2004b; 2005; 2009; 2011; 2013). These guidelines may also be used by non-federal dam owners, regulators, and operators. The goal of the guidelines is to foster a uniform and consistent dam safety framework for Federal, State, and private dam owners and regulators. However, adherence to the guidelines is not mandatory (FEMA and ICODS have responsibility for developing guidelines, but no regulatory authority for implementing safety). Other federal agencies, such as the USACE, Natural Resources Conservation Service (NRCS), USBR, FERC, Tennessee Valley Authority, Bureau of Indian Affairs, U.S. Forest Service, and U.S. Fish and Wildlife Service, have also developed dam safety guidelines. FEMA has published a summary of existing dam safety guidance that provides information on federal dam safety programs (FEMA, 2012).

**Table 1. Federal Agencies' Dam Safety Roles**

| <b>Agency</b>                                                                    | <b>Primary Roles</b>                                                                                                                            | <b>Dams under Jurisdiction</b>                                                                                                                                                                                 |
|----------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| U.S. Department of Homeland Security, Federal Emergency Management Agency (FEMA) | Lead agency for National Dam Safety Program.<br><br>Chairs National Dam Safety Review Board and Interagency Committee on Dam Safety.            | Does not own or operate 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                                                       | Plans, designs, finances, constructs, owns, operates, and permits dams; limited to military lands with exception of USACE civil works programs. | Over 200 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 the USACE but operated by others, and other flood control dams subject to federal regulation.<br><br>Over 600 dams in the NID are associated with the 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 the Bureau of Reclamation.                                                                                                                              |
| U.S. Department of Labor                                                         | Regulates safety and health-related aspects of miners.                                                                                          | About 1,400 dams under the Mine Safety and Health Administration.                                                                                                                                              |
| Federal Energy Regulatory Commission (FERC)                                      | Issues licenses, inspects, and regulates non-Federal dams with hydroelectric capability.                                                        | Over 2,000 dams in the NID are associated with FERC.                                                                                                                                                           |
| Tennessee Valley Authority                                                       | Plans, designs, constructs, operates, and maintains dams.                                                                                       | Approximately 49 major dams in the Tennessee River Valley.                                                                                                                                                     |

<sup>1</sup> The NRC regulates dams providing ultimate heat sink at NPPs, as well as tailings dams at uranium mill tailings sites. Source: FEMA (2009, 2012)

### **C1.4.7. 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. Security concerns related to sharing of detailed dam information may also contribute to the challenges. In most cases, applicants will not own or operate dams or levees that potentially may contribute to flooding hazards at the 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 federal and non-Federal dams (USACE, 2022a) and the National Levee Database (NLD), which provides information on thousands of federal and non-federal levee systems (USACE, 2022b). These databases and inventories are useful sources of basic geographic and physical information on dams and levees in the United States.

#### **Key Points:**

- The applicant should consult national and State databases to identify dams and levees within the NPP watershed.
- The applicant should contact the relevant owner, operator, or regulator to obtain information needed to assess the flooding hazard to the NPP site. However, the NRC staff acknowledges that obtaining detailed information from these entities to support a dam failure flood hazard evaluation may be challenging. Applicants are advised to discuss potential challenges with NRC staff early (i.e., during the pre-application phase). NRC staff can, on a case-by-case basis, assist in interfacing with State or Federal agencies that own, operate, or regulate dams, to facilitate obtaining information to assess the flooding hazards.

### **C2. Screening and Simplified Modeling Approaches for Watersheds with Many Dams**

Section C1.2 and figure 2 provided an overview of a screening approach intended to reduce the analysis burden for watersheds with many dams. This section discusses criteria used to identify those dams that may be removed from further analysis (i.e., “inconsequential” dams). This section also discusses simplified approaches based on both empirical and theoretical methods intended to reduce the effort required to show that failure of certain upstream dams does not result in water levels above the flood protection level of SSCs important to safety, or plant grade, if appropriate (i.e., to 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.

These screening methods are intended to be used with publicly available information (e.g., USACE NID and NLD). The USGS provides online access to data used to delineate and describe watersheds, such as topographic maps, digital elevation datasets, and watershed boundaries (USGS 2022a, b, c).

A justification for using simplified methods should be developed on a site-specific basis and included in the safety analysis report. Other methods can be used and will be reviewed on a case-by-case basis.

#### **C2.1. 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, 2014), 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 cause damage only to the property of the dam owner may be removed. In some cases, State dam safety programs have identified dams in this category. For example, the State of Colorado identifies such dams as “No Public Hazard,” while the Commonwealth of Virginia uses the term “low hazard with special criteria.” This RG refers to these dams as “inconsequential dams.” Removal of dams based on damage being limited to the owner’s property does not apply to any NPP applicant-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.

### **Key Points:**

- Dams identified by Federal or State agencies as having minimal or no adverse failure consequences beyond the owner’s property may be removed from consideration. Dams owned by applicants may not be removed. Other inconsequential dams may be removed with appropriate justification (e.g., 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 their collective failure will exceed the removal criteria.

## **C2.2. Simplified Modeling Approaches**

Several optional methods discussed below provide a quantitative basis for simplified modeling of upstream dams. The methods are presented in a hierarchical-hazard-assessment (HHA)-type gradation of conservatism (see NUREG/CR-7046, “Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America” (NRC, 2011), and are applicable to all initiating events (hydrologic, seismic, and sunny day).

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

The following methods may be applied sequentially in an 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 4 through figure 7.

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 4):
  - 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 top of dam).
  - b. The 500-year flood is used to capture antecedent flood conditions at an 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 500-year water surface elevation estimates nor stage-discharge functions exist, then they should be developed using appropriate methods (e.g., using hydrologic and hydraulic models).

- c. Using available topographic data (e.g., light detection and ranging (LiDAR) datasets or USGS digital elevation models (DEMs), 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 a 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 5):
- a. Use applicable regression equations to estimate 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 top of 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 USBR (USBR, 1982), is recommended.
  - b. Sum the peak failure outflows for all upstream dams (i.e., assume flows from all the upstream dams reach the site simultaneously, ignoring attenuation). As in method 1, “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 an NPP site corresponding to the 500-year return period flood. If stage-discharge functions do not exist, they may be developed using appropriate methods (see method 1, step b).
  - d. Using the stage-discharge function developed in step c, estimate the flood stage corresponding to the summed peak failure outflows from 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 the largest dams, to segregate potentially critical dams from dams with small cumulative effect of failure at the site (see method 1, step d). 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 6):
  - a. Same as method 2, step a.
  - b. Sum the peak failure outflows for all upstream dams (i.e., assume flows from all the upstream dams reach the site simultaneously, taking into account attenuation based on distance). As in method 1, “inconsequential” dams may be excluded. The distance from the dam(s) to the site can be determined using geographic information system (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 or studies to justify their applicability to the river or 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 7): Use an available rainfall-runoff-routing software package (e.g., USACE Hydrologic Engineering Center Hydrologic Modeling System (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 should be justified and shown to be appropriate for use (section C9). This method also requires additional basin-specific inputs (e.g., watershed topography, roughness, unit hydrographs, 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 8). 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 DEMs). See section C2.3 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 in methods 1–3, it may be necessary to iteratively remove dams (hypothetical or real), larger to smaller, to the point where the resulting 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.

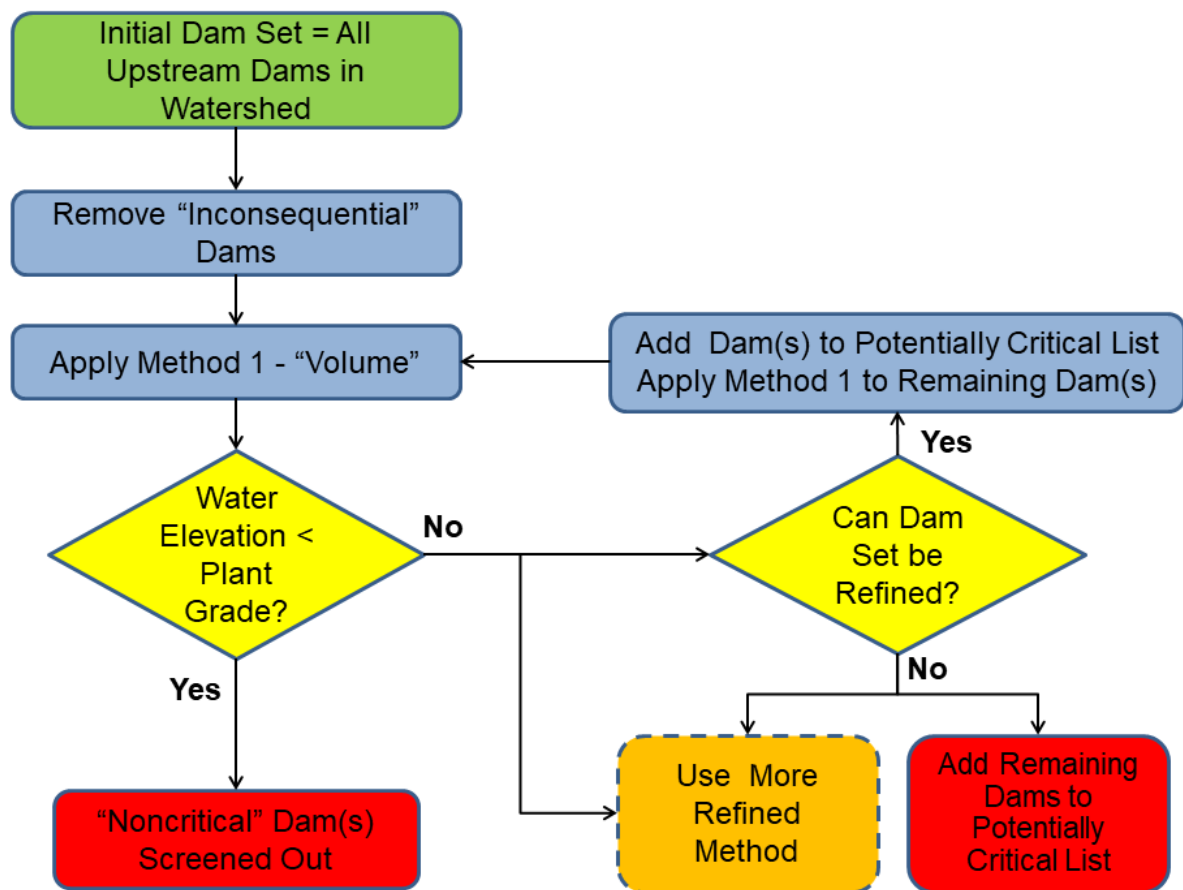
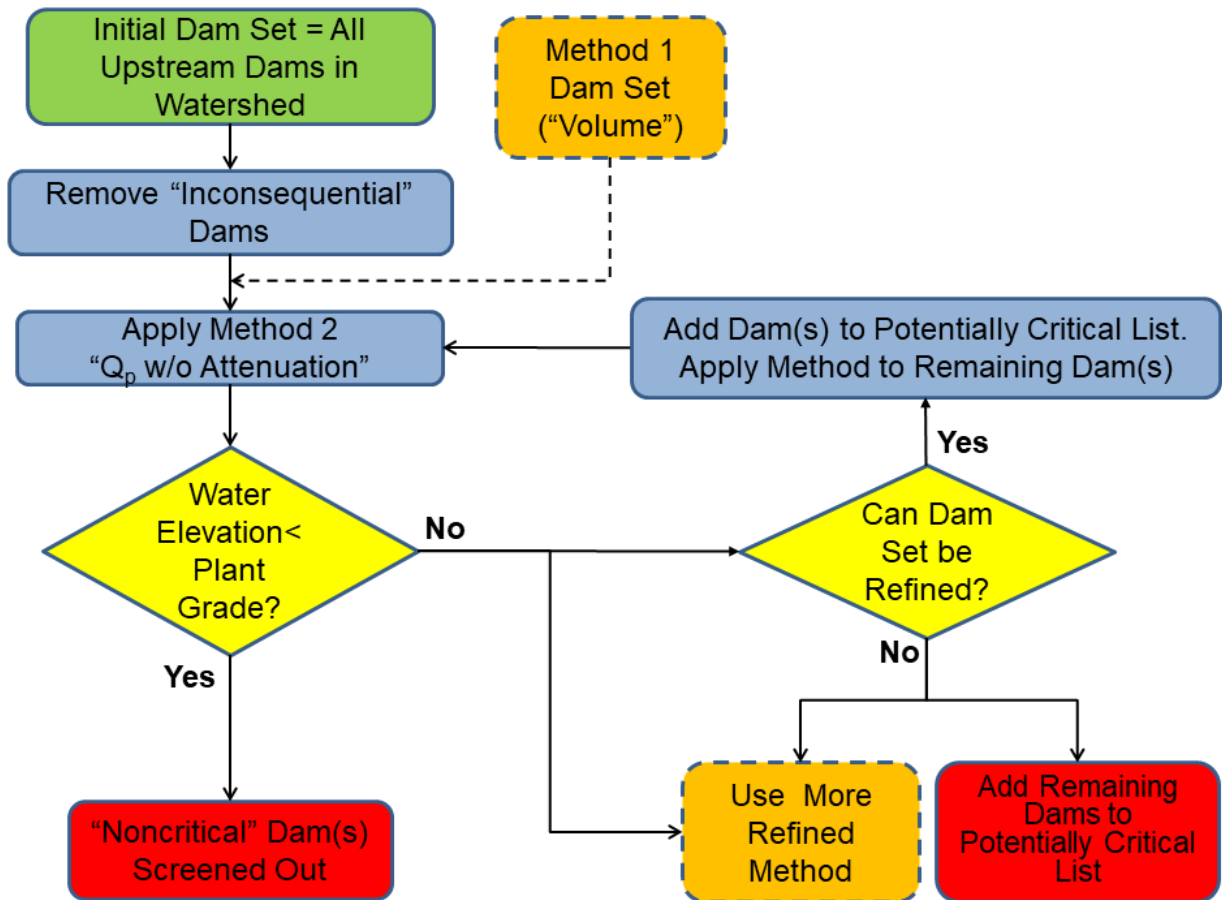


Figure 4. Screening Method 1 (Volume) Flowchart



**Figure 5. Screening Method 2 (Peak Flow without Attenuation) Flowchart**

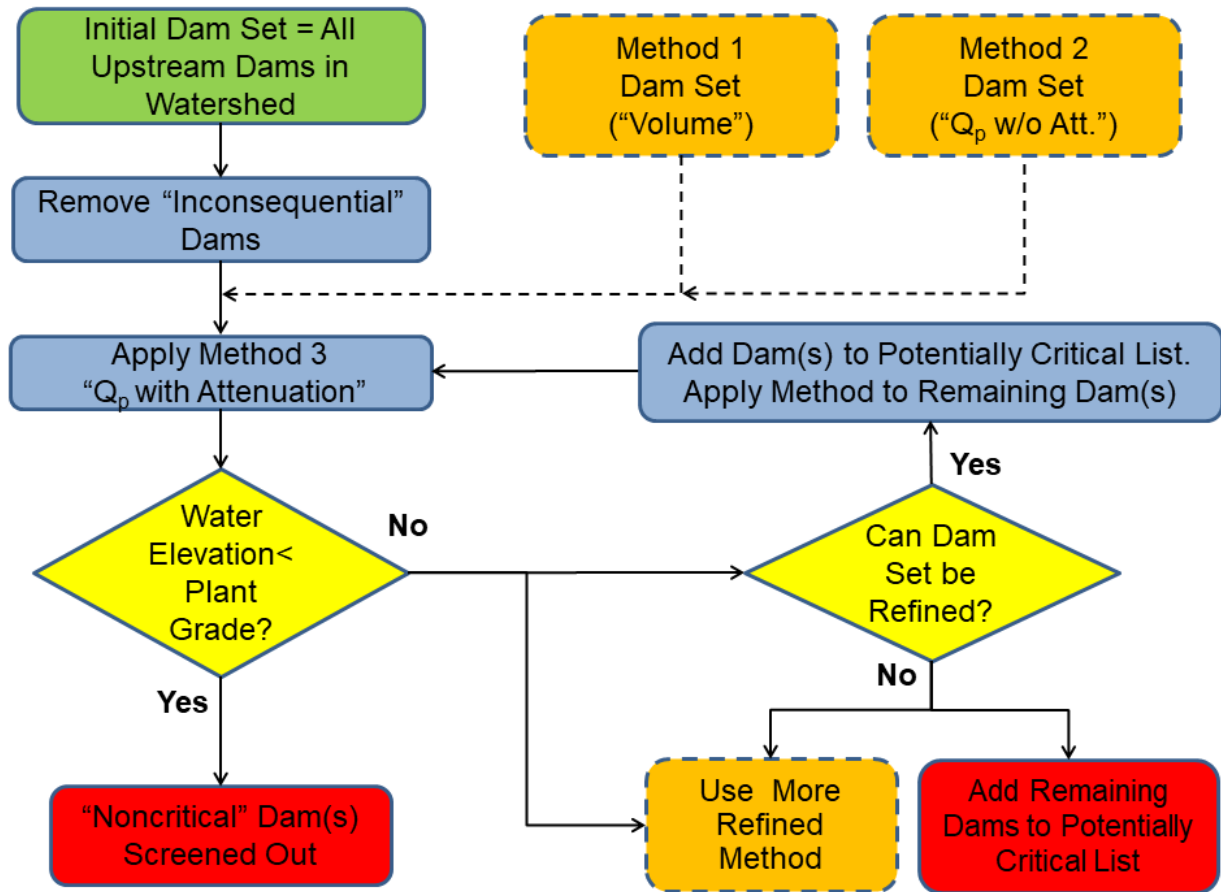
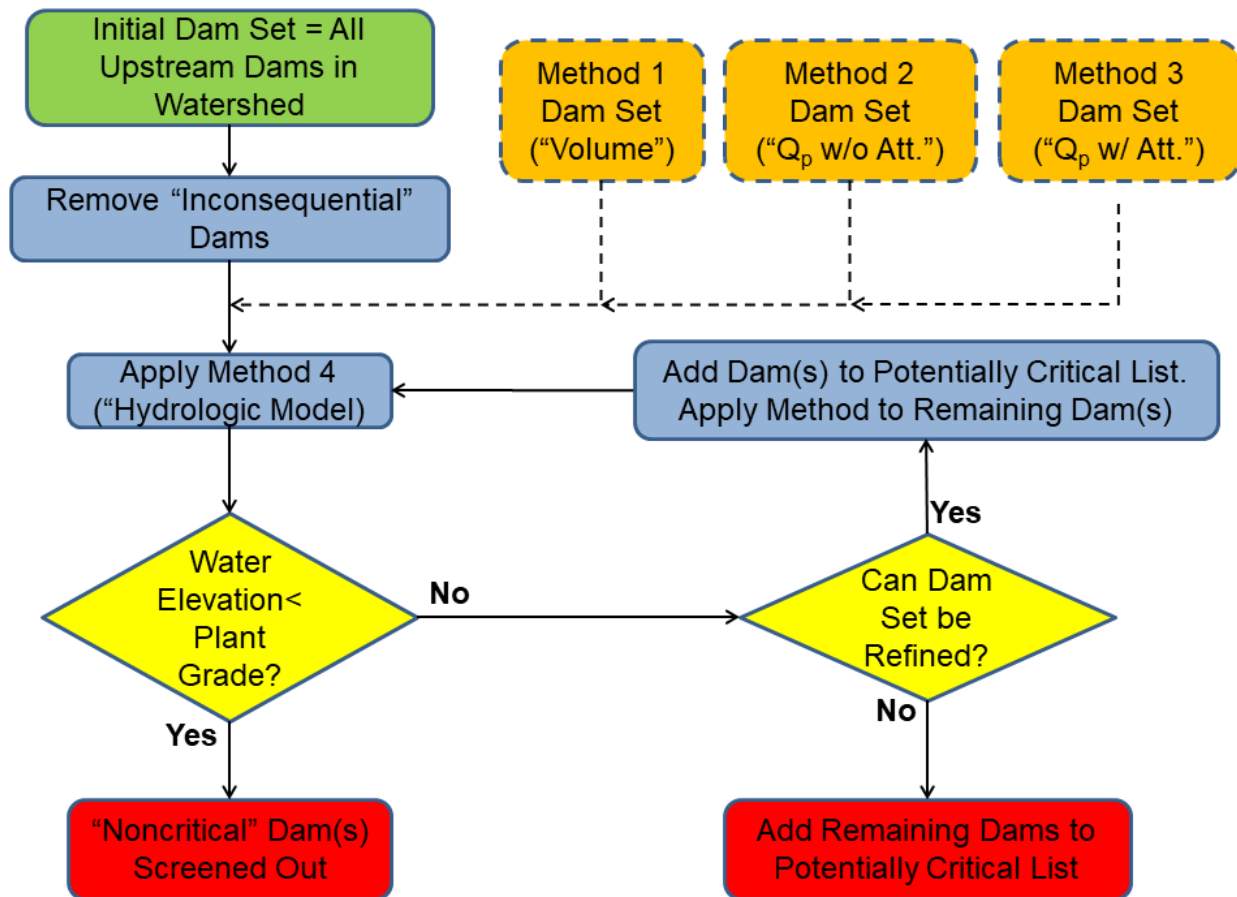


Figure 6. Screening Method 3 (Peak Flow with Attenuation) Flowchart



**Figure 7. Screening Method 4 (Hydrologic Method) Flowchart**

### C2.3. Representing Clusters of Dams

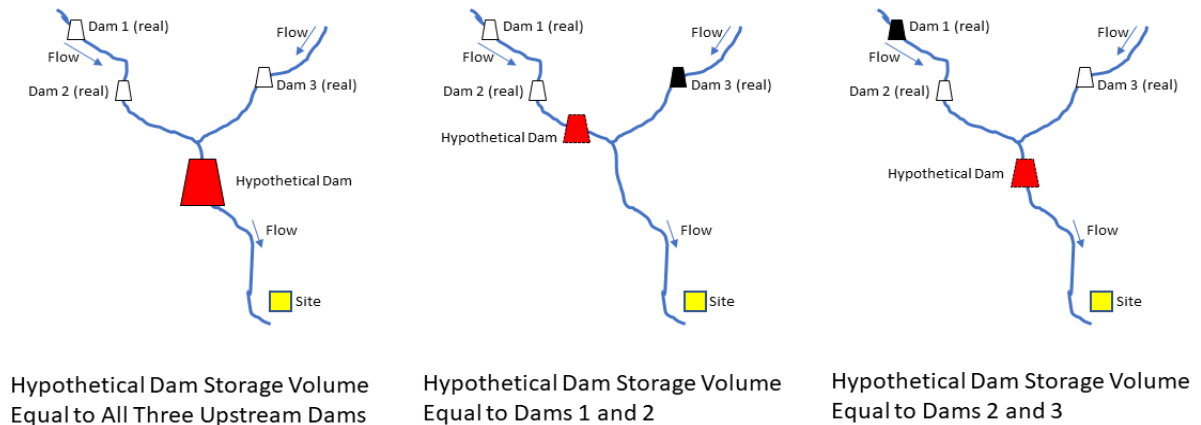
To reduce the level of effort needed to evaluate the flood levels occurring due to dam breach, dams may be grouped together or clustered and represented as a larger hypothetical dam (see figure 8).

#### **Key Points:**

- The volume of the hypothetical dam should be the cumulative volume of the real dams it is intended to represent.
- The location of the hypothetical dam should be at either the most downstream dam in the cluster or even further downstream toward the site. Topographic information from LiDAR or a 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 dam, the analyst should keep in mind that the clustering should make hydrologic sense.



**Figure 8. Dam Clustering Examples**

### **C3. 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. The most common scenario for hydrologic dam failure is a large flood that overwhelms the dam spillway-discharge capacity with floodwaters overtopping the dam crest, which consequently leads to the erosion of the downstream dam face (for embankment dams) or foundation materials (for concrete dams) and eventual failure (breach). Although the likelihood of large floods is obviously a significant component of the hydrologic dam failure hazard, other factors (e.g., operational failures) can play a role. Section C3.1 provides an overview of hydrologic failure by dam type. Section C3.2 presents more detail on analysis of various hydrologic failure modes.

#### **C3.1. Hydrologic Failure by Structure Type**

#### **C3.2. Concrete Dams**

Concrete dams are generally perceived to be relatively resistant to overtopping failure. Non-overflow sections of concrete dams (i.e., sections not designed to be overtopped) are typically able to withstand some overtopping due to the inherent strength of the concrete. However, the foundation or abutments may be susceptible to significant erosion during overtopping flows (e.g., due to weak or 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, (1) erosion of an unlined tunnel or spillway chute, (2) erosion of a channel downstream from a stilling basin due to flow exceeding

capacity, (3) erosion of the spillway foundation where floor slabs have been damaged or lost, and (4) 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 this 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.).

**Key Point:**

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

- overtopping of the main dam, and 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 exceeding capacity;
- erosion of the spillway foundation where floor slabs have been damaged or lost;
- overstressing of the dam, foundation, or abutments; and
- cavitation damage to spillway and outlet flow surfaces.

**C3.1.1. Embankment Dams**

Hydrologic loadings on embankments associated with flooding mainly fall into two categories: (1) increased internal seepage pressures and (2) overtopping which initiates embankment erosion. Overtopping may be due to stillwater elevation alone or occur 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.

**Key Point:**

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

- overtopping and
- increases in internal seepage pressures.

**C3.1.2. Spillways, Gates, Outlet Works, and Other Appurtenances**

There are several dam features, not unique to 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 via overtopping, erosion, or some combination of these. Chief among these are spillways, gates, and other outlet works. Sections C3.2.5 and C3.2.6 further discuss the treatment of spillways, gates, and outlet works in the analysis of hydrologic failures.

**Key Point:**

- Analysis of hydrologic failure modes should consider the potential for loss or degraded function of spillways, gates, outlet works, and other appurtenances.

**C3.1.3. Levees**

Levees that provide flood protection to an NPP site should be evaluated. If they are overtopped in 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. Earthen embankment levees are often 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 by 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 1 foot (0.3048m) 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.

**Key Points:**

- 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), and levee condition.
- Other forms of levees (e.g., pile walls, concrete floodwalls) should be evaluated for potential failures applicable to the particular levee type.
- Levees are generally not designed to withstand high water levels for long periods. However, there is no generally accepted method for predicting how long a levee will continue to function under high loading conditions. Therefore, historical information is often the best available basis for predicting levee performance. The historical information should be from levees that have design and construction characteristics similar to those of the levee being analyzed.
- 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 whole.
- Levees should not be assumed to fail in a beneficial manner without appropriate justification.

#### **C3.1.4. Analysis of Hydrologic Failure Modes**

Overtopping is the most widely recognized hydrologic failure mode. Other common modes include overstressing of the dam or abutments due to hydrologic loads, erosion of embankments due to wave action, and erosion or cavitation in spillways. Analysis of these and other potential failure modes associated with flooding is discussed in more detail below. In addition, the potential failure of multiple dams due to a single storm event is discussed.

#### **C3.1.5. 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.

##### **Key Points:**

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.
- The potential for static liquefaction should be considered.
- Evaluation should generally include reviewing the dam design to ensure that appropriate filters, drains, and monitoring points are included. Monitoring records from piezometers, observation wells, or other observation methods can be used to infer the absence of unremediated deficiencies.

#### **C3.2.1. Overtopping**

Overtopping occurs when the water surface elevation in the reservoir exceeds the height of the dam. Water can then 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. Section C7 discusses details of breach modeling of dams.

Dams are typically designed to accommodate the IDF. In many cases, the IDF is the PMF developed by analyzing the impacts of the 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 IDF is the most common cause of overtopping (IDF estimates often change over time as more data are 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.

**Key Points:**

- 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 detailed engineering analysis supported by site-specific information, including material properties of the embankment and foundation soils, material properties of embankment protection (if any), and dam condition.
- 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 the appropriate failure assumptions with appropriate engineering justification.

**C3.2.2. Reservoir Capacity**

The reservoir capacity will influence the maximum water surface elevation, as well as the rate of change in elevation during floods. The potential for reductions in reservoir capacity over the life of an NPP should be considered. 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 coal ash impoundments, and failure of upstream coal ash and mine tailings impoundments.

**Key Point:**

- The potential for reductions in reservoir capacity due to sedimentation over the life of an NPP should be considered. Records from periodic bathymetric surveys of the reservoir, records of sediment production in upstream reaches, or estimates of sediment production rates for the upstream watershed can be used to support modeling assumptions.

**C3.2.3. Starting Reservoir Elevation**

The starting reservoir water surface elevation at the beginning of the 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 because of the additional surcharge space within the reservoir. Some reservoirs are operated to provide more surcharge storage during flood season.

**Key Point:**

- 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, then 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.

#### **C3.2.4. Reservoir Surge Capacity**

Reservoir surge capacity will also affect the ability to pass large floods at a dam. Reservoir surge 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 surge volume in comparison to the flood volume, in addition to the spillway type and capacity. If the reservoir surge volume is large in comparison to the flood volume, significant attenuation will occur.

##### **Key Point:**

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

#### **C3.2.5. 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 includes spillway discharge capacity and any additional release capacity that would be available through other release structures at the dam. 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 surge capacity of the reservoir, dam overtopping could occur. The likelihood of these floods and the erodibility of the dam or foundation materials control the risk.

Existing federal guidance is not consistent on crediting release capacity through appurtenances other than the spillway (e.g., outlets or turbines). For example, USACE Engineering Manual EM 1110-2-1603, “Hydraulic Design of Spillways,” (USACE 1992) states that a powerhouse should not be considered as a reliable discharge facility when determining the safe conveyance of the spillway. Conversely, FERC’s “Engineering Guidelines for the Evaluation of Hydropower Projects”, Chapter 2 “Selecting and Accommodating Inflow Design Floods for Dams,” (FERC 2015) 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, 2014) suggest that at least one turbine should always be assumed to be down (e.g., for maintenance or other reasons) in performing flood routing.

The operational history of 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, 2022), which can be used if site-specific information is not available. Site-specific records from past flooding events, if available, should be reviewed.

### **Key Points:**

- Release capacity through appurtenances other than the spillway (e.g., outlets, 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. However, at least one turbine should always be assumed to be down (e.g., for maintenance or other reasons) in performing flood routings.

### **C3.2.6. Potential for Debris to Block Reservoir Spillway**

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 9 shows debris buildup at Lake Lynn Dam on the Cheat River (West Virginia) during a large flood in 1985. The spillway capacity was reduced by approximately 35 percent from the theoretical flow, and 13 out of 26 Tainter gates were almost fully blocked (Schadinger et al., 2012).



**Figure 9. 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 on debris production in the watershed (or similar watersheds) can be used to gauge the potential for debris blockage. Dam owners, dam regulators, or river basin commissions often perform periodic debris studies.

**Key Points:**

- 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 percent reduction in capacity should be performed. The study should 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. The reduction used should be justified (e.g., by debris studies for the watershed or similar watersheds).

**C3.2.7. 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 windspeeds, and the fetch of the reservoir).

Parapet walls are sometimes used 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.

**Key Point:**

- 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 windspeed and added to the stillwater elevation.

**C3.2.8. 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).

**Key Point:**

- 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, failure of the appurtenance should be assumed, and the impact of its failure on stability of the dam should be estimated. If the dam stability is not impacted, the analysis should still consider the downstream impact of uncontrolled release (if any) associated with appurtenance failure.

**C3.2.9. Surface Erosion from High Flow Velocity and Wave Action**

Surface erosion can occur along earthen spillways, along the upstream or downstream embankment slopes, or along other appurtenant structures such as inlet and outlet channels. 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 toward the spillway crest, eventually leading to a full dam breach (FEMA, 2004a).

**Key Point:**

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

**C3.2.10. Spillway Failure**

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 or flow into the foundation. If no drainage exists, or if the drainage 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 surface, there may be damage to the surface material. Cracks, offsets, surface irregularities, 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 (1) failure of the grass or vegetation cover in the spillway, (2) concentrated erosion that initiates a headcut, and (3) 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 (NRCS, 2007) and the Windows Dam Analysis Modules (WinDAM, Hunt et al., 2021) are publicly available. Both computer programs implement similar technology for evaluating spillway integrity. They can indicate whether breach of a spillway due to headcutting is likely, but they 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 U.S. Society on Dams publication contains a detailed discussion of causative mechanisms and predictive models for erosion of unlined soil- or grass-covered spillways (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 these models to a rock channel, because the models 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). A U.S. Society on Dams publication presents a detailed discussion of causative mechanisms and predictive models for erosion of unlined spillways excavated in rock (USSD, 2006).

### **Key Points:**

- 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 (nonconcrete) 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.

### **C3.2.11. Failure of Gates**

A variety of gates are used to control spillways. Gates range in complexity from simple slide gates (e.g., fixed wheel gates, roller gates), to float-type gates (e.g., drum gates, ring gates), to gates that 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 most of the 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, the 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 in danger, the fuse plug breaches, and the floodwaters safely spill over the dam.

Gates may fail to operate because of mechanical or power failures. Gates may also fail to operate when needed in flooding situations because of 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 because of downstream flooding concerns).

Fuse plugs are generally considered to be 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.

### **Key Points:**

- The evaluation should consider the potential for gate failure under flooding conditions to lead to an uncontrolled release of the reservoir.
- Regarding fuse plugs, the evaluation 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.

### **C3.2.12. Operational Failures and Controlled Releases**

Certain operational failures and even certain controlled releases can lead to flooding at an 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.

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. Examples of these types of failures include the following:

- 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 because of excessive friction or corrosion. This is more common with gates that are not maintained or are 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 because of 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, a large earthquake or flood may cut power and phone lines. This may result in an 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).

**Key Point:**

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

There may be instances in which controlled releases can lead to inundation at an NPP site. Examples include but are not limited to (1) releases performed to prevent dam failure during flood conditions, (2) releases performed to rapidly draw down a reservoir to prevent incipient failure after a seismic event, and (3) releases performed to rapidly draw down a reservoir to prevent incipient sunny day failure. Consideration of the potential for controlled releases to cause flooding at an 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.

**Key Point:**

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

**C3.2.13. Waterborne Debris**

Waterborne debris (e.g., trees, logs, or other objects) produces drag and impact loads that may damage or destroy buildings, structures, or their parts. The magnitude of these loads is very difficult to predict, yet some reasonable allowance should be made for them in evaluating dam performance. The loads are influenced by the location of the structure 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), or
- among closely spaced buildings.

Building standard ASCE/SEI 7-22, developed by the American Society of Civil Engineers (ASCE, 2022), describes a methodology for determining impact loads based on the momentum impulse method. The methodology differs from the classic impulse-momentum approach (USACE, 1995; 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/SEI 7-22 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 (i.e., those structures that pose a substantial hazard in the event of failure), is appropriate. The ASCE/SEI 7-22 methodology also uses a depth coefficient meant to consider 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 7-22 also provides a method for estimating drag loads on structures.

ASCE/SEI 7-22 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 pounds are appropriate for riverine floodplains in most areas of the United States. In the Pacific Northwest, larger tree and log sizes suggest a typical 4,000 pound debris weight. Debris weights in riverine areas subject to floating ice typically range from 1,000 to 4,000 pounds. ASCE/SEI 7-22 considers the 1,000 pound 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 local conditions before the final debris weight is selected.

#### **Key Points:**

- 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/SEI 7-22, with the caveats described above, are acceptable to the NRC staff.
- Applicants should consider regional and 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.

#### **C3.2.14. Multiple Dam Failure Due to a Single Storm**

Some NPP sites may have a potential for flooding due to multiple dam failures (e.g., dams on different reaches or tributaries above the plant) or the domino failure of a series of dams on the same reach. For example, the site may be in a watershed where dams are close enough to one another so 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 into the reservoir impounded by a downstream dam and may cause failure by overtopping of 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).

### **Key Points:**

- Those dams unable to be removed as “inconsequential” or screened out as “noncritical” (see section C2) are considered potentially critical dams. These dams should be evaluated for hydrologic failures that lead to cascading failures of downstream dams or simultaneous dam failures causing flood conditions at the site. Operational rules may be considered but the starting water surface elevation should be as specified in section C3.2.2.1. Flood waves from multiple dam failures should be assumed to reach the NPP site simultaneously unless appropriate justification for differing flood arrival times is provided.
- River flows should be based on the precipitation and runoff from the basin encompassing the multiple dam scenario(s) under consideration. Flood waves from multiple dam failures should be assumed to reach an NPP site simultaneously unless appropriate justification for differing flood arrival times is provided.
- Three cases of multiple dam failure should be considered: (1) failure of individual dams on separate tributaries upstream from the site, (2) cascading or domino-like failures of dams upstream from the site, and (3) combination of cases (1) and (2).
  - 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 into the reservoir impounded by a downstream dam and may cause failure by overtopping of 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. In 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 C9. 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.

### **C3.2.15. Levee Failures**

Failure of levees that provide flood protection to an NPP site should be considered. Such levees should be considered to fail when overtopped. Their stability when they are not overtopped should be addressed 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 under-seepage resurfaces on the land side, 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, with the purpose of protecting other areas. In most cases, an intentional breach is not initiated without significant planning and notification.

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

### **Key Points:**

- 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), and levee condition. Other forms of levees (e.g., pile walls and concrete flood walls) should be evaluated for potential failures applicable to the particular levee type.
- 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 design and construction characteristics similar to those of the levee being analyzed.
- If the performance of levees is potentially important to estimation of inundation at an 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 because of large uncertainties in implementing such plans (e.g., decisions about such actions are often political).
- Assumptions about 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.

#### **C4. Seismic Dam Failure**

Consideration of seismically induced floods should include the same range of seismic events as that postulated for the design of the nuclear plant.

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 the 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 where “load combinations” come into play (e.g., a more frequent earthquake combined with a flood event), as discussed in section C5.6. 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 C5.6, 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.

Detailed guidance on methods for engineering analyses of seismic hazard potential and seismic dam failure scenarios is beyond the scope of this guide. Federal guidelines for dam safety developed by FEMA, USACE, and USBR should be consulted (e.g., FEMA, 2005; USBR & USACE, 2019). The NRC has published regulatory guides that focus on seismic hazard assessments (e.g., RG 1.208). A number of geotechnical engineering texts address several aspects of seismic hazard analysis (e.g., Duncan et al., 2014; Kramer, 1996).

#### **Key Points:**

- 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).
- Consequences of dam failure caused by an earthquake should be considered for both the maximum normal operating (full-pool) and average reservoir levels. Reservoir and downstream tributary inflows should be seasonally consistent with the selected reservoir level.

## **C4.1. 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 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 knowledgeable seismic engineers, following the “state of the practice” in the profession, conduct seismic investigations.

### **C4.1.1. Seismic Hazard Characterization**

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

Preferably, seismic evaluation parameters should be specified based on 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 because of differences in earthquake focal depths, transmission paths, and tectonic settings. It is important to indicate if seismic parameters predicted by attenuation relationships account for 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 contains soils that are prone to accumulate excess pore water 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.

#### **C4.1.1.1. Use of USGS National Seismic Hazard Maps**

The USGS National Seismic Hazard Maps (USGS, 2014) 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 estimates ground motions from sources that are up to 1,000 kilometers from the site. In the Western United States, the USGS calculates ground motion from crustal sources less than 300 kilometers and subduction sources less than 1,000 kilometers from the site. The USGS also maintains a website where the maps, as well as the data and software used in their creation, are available (<http://earthquake.usgs.gov/hazards/?source=sitenav>).

#### **C4.1.2. 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 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.

#### **C4.1.3. Probabilistic Seismic Hazard Analysis**

A PSHA involves relating a ground motion parameter and 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) is 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 accounts for 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, or spectral amplitudes at specified periods.

Deterministic approaches have been favored in dam engineering in the past; however, 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 acceleration time histories.

#### **Key Point:**

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

## **C4.2. 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.

### **C4.2.1. 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 & USACE, 2019), 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 dams 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 on 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 on 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 are important for the buttresses and the supporting corbels.

**Key Points:**

- 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 and deformation potential should be considered.
- Structural failure modes considered should address the unique concerns for the type of dam in question.

**C4.2.2. Embankment Dams**

Although many embankment dams have been exposed to earthquake shaking, there have been few instances in which 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 when 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 which, 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 sliding 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. Foundation liquefaction can also lead to upstream or downstream slope instability.

There are many ways in which cracking can occur because of 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.

**Key Points:**

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

### **C4.2.3. Spillways, Gates, Outlet Works, and Other Appurtenances**

For a number of facilities, not unique to any one dam type, 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.

#### **Key Point:**

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

### **C4.2.4. 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).

#### **Key Points:**

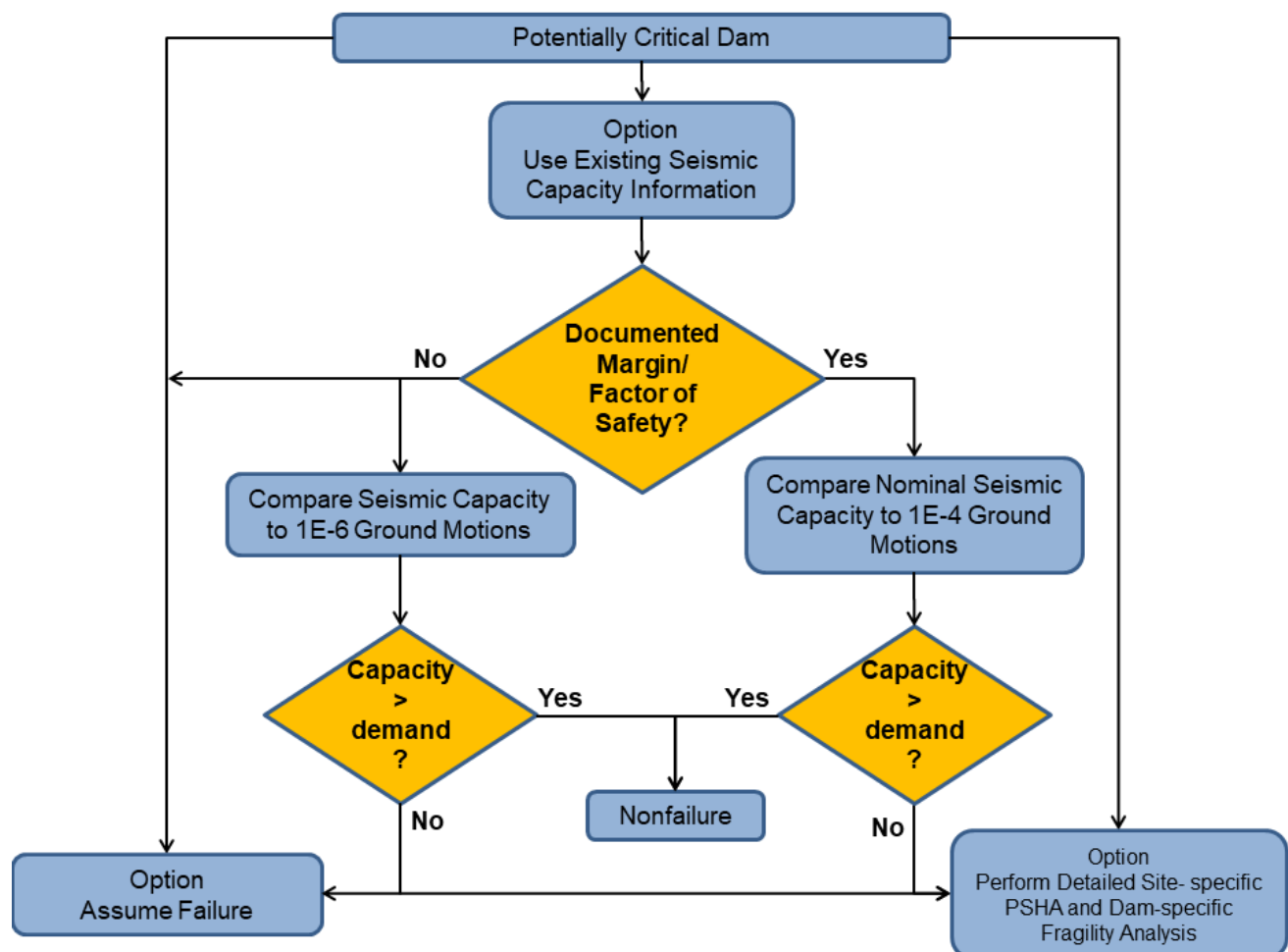
- As with hydrologic levee failures, seismic failure of distant levees is not of concern. Failure of an offsite levee that provides flood protection to the NPP site, where applicable, is of interest (seismic analysis of onsite water control structures including levees, if applicable, are part of the safety analysis report).
- 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.

### **C4.3. Analysis of Seismic Hazards Using Readily Available Tools and Information**

Since there will generally be insufficient time and resources to perform detailed seismic analyses for all dams upstream from the NPP site, the following approach may be applied. It is assumed that the screening approach described in section C3 has already been applied (i.e., inconsequential dams have been removed and noncritical dams have been screened out).

The analysis approach outlined in this section and in section C4.4 is meant to take advantage of existing information for the dam (e.g., seismic design information or seismic qualification studies), along with a corresponding level analysis of the seismic hazard (e.g., existing seismic hazard curves or seismic hazard assessments developed using readily available tools and data). To apply this approach, the seismic capacity of the dam (i.e., based on seismic design or postconstruction seismic capacity studies) should 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 applicant 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 C4.7), as well as more detailed analysis of the seismic capacity of the dam (discussed in section C4.8). Figure 10 outlines the options for performing seismic hazard analysis.



**Figure 10. Seismic Dam Failure Analysis Options**

### C4.3.1. Ground Shaking

Ground shaking is one of the most common seismic loads that should be considered for dams. As discussed in section C1.4.2, it is acceptable to use the  $1 \times 10^{-4}$  annual frequency ground motions, at spectral frequencies important to the dam, for seismic evaluation. When feasible, a sensitivity analysis at  $1 \times 10^{-5}$  annual frequency ground motion is recommended. 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.

#### **Key Points:**

- The seismic hazard at the dam site should be characterized using PSHA for the spectral frequencies of interest for 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 (the 2104 version as of the publishing of this guidance) are suitable for developing bedrock hazard curves and uniform hazard spectra at  $1 \times 10^{-4}$  annual frequency of exceedance (USGS, 2014). However, due diligence should be applied to demonstrate the continued validity of the data used in USGS (2014) for sites in the Western United States.
  - A site amplification analysis should be performed to obtain site amplification functions. Methods developed by the Electric Power Research Institute to perform a site response analysis (EPRI, 1989) as described in NUREG/CR-6728, “Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-Consistent Ground Motion Spectra Guidelines” (USNRC, 2001) are acceptable.
  - Aleatory variability and epistemic uncertainty should be incorporated into analyses to develop the site amplification functions. NRC Research Information Letter (RIL) 2021-15, “Documentation Report for SSHAC Level 2: Site Response,” (USNRC, 2021) provides information on an approach to incorporating epistemic uncertainty into the site amplification function evaluation.
- As an alternative to use of the USGS seismic hazard curves, it is acceptable to perform a site-specific PSHA consistent with the methodologies suitable for use in characterizing seismic hazard at U.S. NPP sites, as described in RG 1.208 (USNRCf).

### C4.3.2. 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 (PFDHAs) 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). An 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).

#### **Key Points:**

- 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 use existing analyses that demonstrate that a dam is not susceptible to fault displacement.

#### **C4.3.3. 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, strength, or both) resulting from repeated rapid straining, regardless of the state of stress before 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). Liquefaction evaluations generally consist of evaluating susceptibility, triggering, and consequences. 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), and
- (5) a seismic history of the site.

Detailed investigations would include surveys, in situ field testing, and laboratory testing, as appropriate to (1) 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 available databases.

RG 1.198, “Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites” (USNRCg), describes screening techniques, as well as procedures for detailed analysis. However, the state of practice has advanced, and an update of RG 1.198 is in process to include additional information. Key supplemental information includes the following:

- Work by Bray and Sancio (2006) and Seed et al. (2003) should be applied when evaluating soil susceptibility to liquefaction. When the plasticity index of a fine-grained soil is three or greater, detailed in situ and laboratory testing are recommended for evaluating cyclic behavior as recommended by Boulanger and Idriss (2006).
- Triggering models referenced in RG 1.198 rely on data from 1985 and earlier. Newer models are available (e.g., Idriss and Boulanger, 2012; Cetin et al., 2018). Differences in some of the newer liquefaction triggering models led to a National Academies study (NASEM, 2021) on the state of the art and practice in assessing earthquake-induced liquefaction. The NASEM study recommends using more than one simplified method when making a liquefaction triggering assessment.
- The Next Generation Liquefaction project with support from the U.S. NRC, has developed a publicly available, living database of liquefaction case histories (NGL, 2022). This database will support development of new triggering and consequence models.
- It should also be noted that semi-empirical liquefaction triggering models are based on case histories where the liquefied soil is at a depth of no greater than 15 meters below the ground surface. Also, the case histories used to develop triggering models are typically from level sites where static shear stresses are negligible.
- Foundation soils below dams may experience much higher effective stresses compared to effective stresses at 15 meters below the ground surface, and dam foundation soils are subject to static shear stresses. Correction factors have been developed to modify the semi-empirical triggering models for these conditions. As noted by Youd et al. (2001), use of these correction factors is beyond routine engineering practice and requires specialized expertise.

**Key Points:**

- The dam site should be evaluated for liquefaction potential.
- RG 1.198, along with the supplemental information noted above, provides guidance on acceptable methods for evaluating the potential for earthquake-induced instability of soils resulting from liquefaction and strength degradation.

**C4.4. Assessment of Seismic Performance Using Existing Studies**

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

**Key Point:**

- Existing studies will be accepted on a case-by-case basis. However, the studies used 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 percent exceedance duration pool level calculated using average daily water levels for the period of record). The average nonflood tailwater level should be used with both headwater conditions above.

**C4.4.1. Ground Shaking**

To use existing studies to demonstrate the seismic capability of a dam, the seismic capacity of the dam (e.g., based on seismic design or postconstruction seismic capacity studies) should be known for spectral frequencies of importance to the dam (e.g., using design response spectrum).

**Key Points:**

- 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 C4.3.1. 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 because of seismic ground shaking. Appropriate margin is usually expressed as a factor of safety, which will depend on 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 RG. If the relevant federal agency guidance proposes a factor of safety of 1.4 under the ground motions considered, then the licensee would show that this factor of safety (1.4) is maintained when the dam is subjected to the  $1 \times 10^{-4}$  ground motion.
- When 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 PGA and the dam has not been reevaluated to define capacity in terms of other intensity measures), the applicant 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 C4.3.1, 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 C7. 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 C4.5. Alternatively, it is acceptable to perform more detailed assessment of the performance of the dam (i.e., perform new assessments) as described in section C4.8.

**C4.4.2. Fault Displacement**

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.

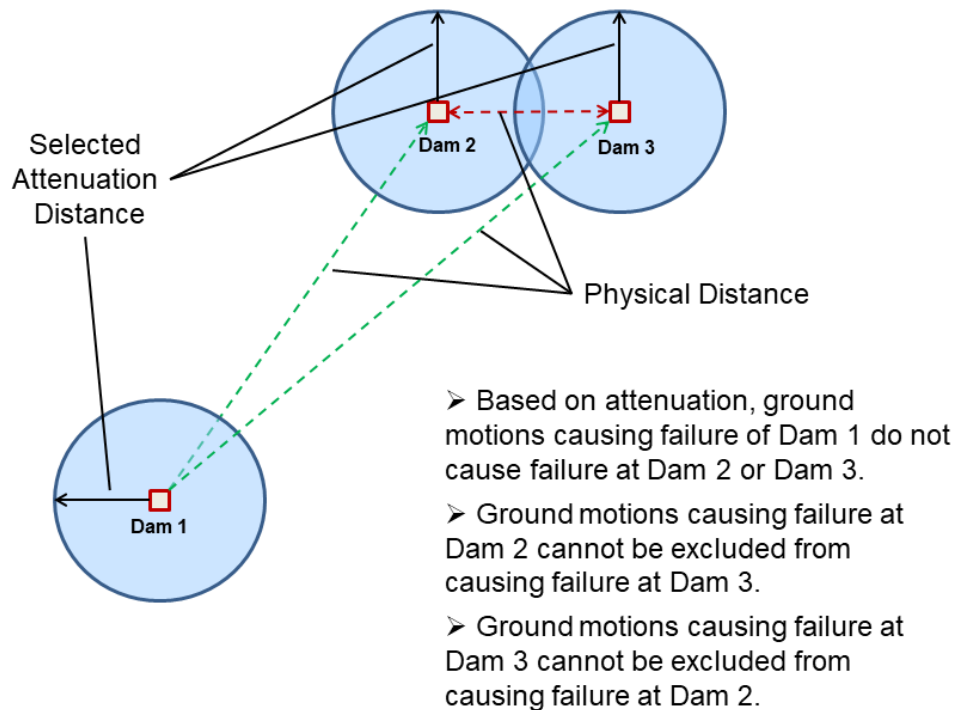
### **C4.4.3. Liquefaction**

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.

### **C4.5. Multiple Dam Failures Due to 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  $1 \times 10^{-4}$  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, applying 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 applicable ground motion models 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 84th 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 a negligible level. Figure 11 illustrates the above concept. A ground motion model 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 the 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 11. Using Ground Motion Attenuation Information to Screen Combined Seismic Failures**

If the attenuation distance approach outlined above does not rule out combined failure, the potential for multiple failures may be further refined through de-aggregation of the seismic hazard at the location of the dams. De-aggregation 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 de-aggregation of the seismic hazard for relevant ground motion measures and at ground motion levels corresponding to the multiple annual frequencies (e.g.,  $1 \times 10^{-2}$ ,  $1 \times 10^{-3}$ ,  $1 \times 10^{-4}$ ). When considering relevant ground motion measures, multiple annual frequencies, and the site characteristics, if the de-aggregation of the hazard indicates that a large portion of the hazard (e.g., greater than 85–90 percent) 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 the 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 de-aggregation (see figure 12).

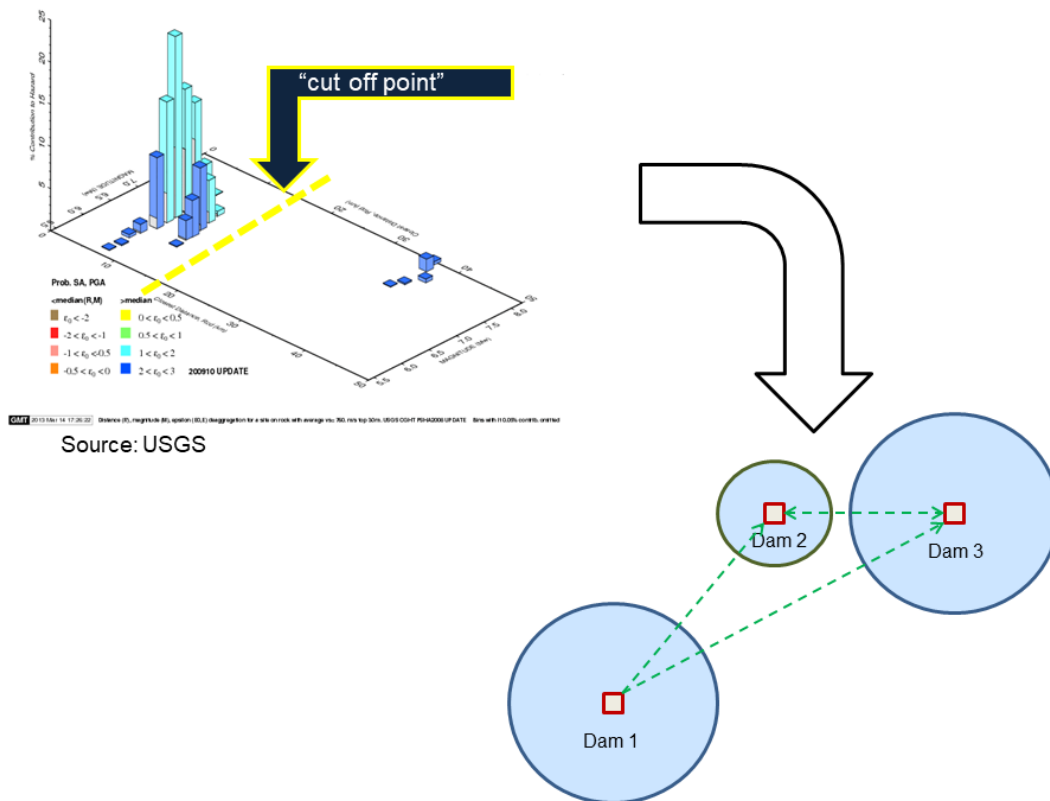


Figure 12. Refinement of Seismic Influence

### Key Points:

- 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.

### C4.6. Modeling Consequences of Seismic Dam Failure

Once a dam has been assumed to fail under a seismic load, the consequences of dam failure will be developed through breach modeling and flood wave routing as discussed in sections C8 and C10, respectively. However, assumptions regarding headwater and tailwater levels, as well as coincident flood flows are discussed here.

### Key Points:

- If the dam failed under the  $1 \times 10^{-4}$  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 less than the  $1 \times 10^{-4}$  annual exceedance probability seismic hazard (ground motion), assume that failure coincides with the peak water level from a 500-year flood (or  $\frac{1}{2}$  PMF, whichever is less) in the watershed above the dam.

- Water level estimates at the site should include effects of a 2-year windspeed 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 in which the operating history 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.

#### **C4.7. Detailed Site-Specific Seismic Hazard Analysis**

When the screening approach described above indicates that a dam cannot be screened out based on 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.

##### **Key Point:**

- Because each dam and its immediate environment form a unique system, it is not feasible to provide detailed guidance that will apply in all cases. Therefore, detailed, site-specific seismic hazard analyses will be reviewed on a case-by-case basis. The components that would normally be part of a detailed seismic hazard evaluation include:
  - estimation of ground shaking
    - seismic source characterization
    - ground motion attenuation modeling
    - site response modeling
    - development of uniform hazard spectra
    - development of time histories
  - estimation of fault displacement

#### **C4.8. Detailed Dam Seismic Capacity Analysis**

Once the earthquake ground motions or displacements have been determined, their impact on the dam and its appurtenances should 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.

**Key Point:**

- Because each dam and its immediate environment form a unique system, it is not feasible to provide detailed guidance that will apply in all cases. Therefore, detailed, site-specific seismic capacity analyses will be reviewed on a case-by-case basis. The components that would normally be part of a detailed evaluation include the following:
  - sliding and overturning stability analysis,
  - deformation analysis, and
  - assessment of liquefaction potential.

**C5. Other (Sunny Day Failures)**

Dam failures not caused by 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 flood gates, valves, spillways, conduits, and other components. The possibility of these failures should be carefully evaluated to ensure that all plausible mechanisms for flooding from dam breaches and failures at and near a site are considered. Plausible causes include but are not limited to the following:

- 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 build up of silt load against dam;
- excessive leakage through foundation;
- embankment slope failure;
- leakage along conduit in embankment;
- channels from tree roots or burrowing; and
- landslide in reservoir.

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

**Key Points:**

- The possibility of sunny day failure causes 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.
- Sunny day failures should not be screened out. If no other plausible failure mechanisms exist, sunny day failures should be the default failure scenario for the purposes of this RG. An exception is that dams failed because of 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.
- A sunny day breach can be used to model piping failures for hydrologic, geologic, structural, seismic, and human-influenced failure modes.

## **C5.1. Overview of Sunny Day Failure by Dam Type**

### **C5.1.1. 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 and 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, which leads to higher uplift and potentially unstable conditions. If conditions appear to be changing, risks are typically estimated 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 should 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.

### **C5.1.2. 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 (non-hydrologic, non-seismic) 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 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 (2019).

### **C5.1.3. Levees**

Failure of levees that provide flood protection to an 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 under-seepage 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 to protect other areas. An intentional breach is usually not initiated without significant planning and notification.

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

**Key Points:**

- 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), and levee condition. 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 design and construction characteristics similar to those of the levee being analyzed.
- If the performance of levees is potentially important to estimating inundation at an 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 because of large uncertainties in the implementation of such plans (e.g., decisions about such actions are often political).
- Assumptions about 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.

**C5.2. 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 are discussed below. Initial water surface elevation used in breach modeling and flood routing is also discussed. Section C8 discusses details of breach modeling, and section C9 addresses details of flood routing.

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. When base flow is considered, the discharge is

typically estimated based on reported base flows through the dam's outlet works or from stream gauge records. Additional inflow (e.g., from a storm event) is not required when analyzing a sunny day breach.

### **C5.2.1. Sunny Day Failure Modes**

An essential element in evaluating the potential for sunny day failure is assessment of credible failure modes. Section C5.1 discusses common sunny day failure modes for various dam types. That discussion is fairly comprehensive, but it 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 requires reviewing all relevant background information on a dam, including geology, design, analysis, construction, operations, dam safety evaluations, and performance monitoring documentation.

### **C5.2.2. Initial Water Surface Elevation**

Section C8 discusses breach modeling in detail but does not discuss assumptions about initial water surface elevations used in the breach modeling.

#### **Key Points:**

- 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, possible instances in which the operating history or rules have been influenced by anomalous conditions such as drought should be considered.

### **C6. Operational Failures and Controlled Releases**

Certain operational failures and even certain controlled releases can lead to flooding at the NPP site. They may occur in a variety of situations and cannot be neatly categorized as a hydrologic, seismic, or sunny day mechanism. They may be a compounding factor in any of these mechanisms.

#### **C6.1. Operational Failures**

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. Examples of these types of failures include the following:

- 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 because of excessive friction or corrosion. This is more common with gates that are not maintained or are 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 powerplant 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.
- Mechanical equipment failure due to changes in operation without a corresponding change in maintenance. 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, leading to premature dam overtopping.
- Overfilling off-stream storage leads to overtopping and failure of the dam. This could happen because of 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. For example, a large earthquake or flood may cut power and phone lines. This may result in an inability to warn in advance of life-threatening downstream flows.

## **C6.2. Controlled Releases**

There may be instances in which controlled releases can lead to inundation at the NPP site. Examples include but are not limited to the following:

- (1) releases performed to prevent dam failure during flood conditions,
- (2) releases performed to rapidly draw down a reservoir to prevent incipient failure after a seismic event, and
- (3) releases performed to rapidly draw down a reservoir to prevent incipient sunny day failure.

Consideration of the potential for controlled releases to generate flooding at the 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.

### **Key Point:**

- The potential for controlled releases that may threaten to inundate the NPP site should be considered.

## **C7. Dam Breach Modeling**

The breach is the opening formed in a dam when it fails, and the aim of breach analysis is estimation of 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 distance from the dam to the locations or populations 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, progressively, or both. 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.

### **C7.1. Breach Modeling for Concrete Dams**

In most cases when 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 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 modeling cases in which the breach width parameter representing the combined lengths of assumed failed monoliths is varied in each case, the resulting reservoir water surface elevations can be used to indicate the extent of reduction of 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 breach geometry 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.

### **C7.2. Breach Modeling of Embankment Dams**

For earthen embankment 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 the 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 dam crest may be of some value in slowing headcutting once the gullies reach the crest but should not be expected to affect initiation.

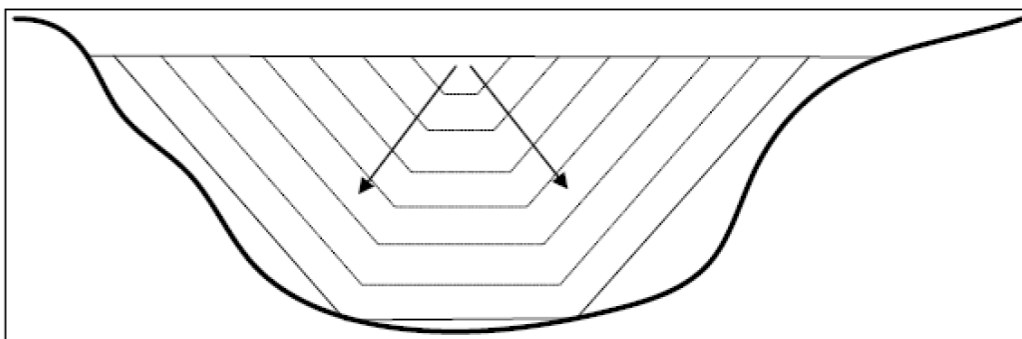
For cohesive soils, the failure mechanism is typically headcut initiation and advance. A small headcut typically forms near the toe of the dam and then advances upstream until the crest of the dam is breached. For cohesionless soils, the failure process typically initiates because of tractive stresses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

- 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, and
- whether a parapet wall is provided and the potential for the wall to fail before or after the dam is overtopped.

For embankment dams, the 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 developing 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 it encounters the bedrock foundation or some other erosion resistant strata. 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 13).
- “Piping” is the 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 13. 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.

Two widely used approaches to breach modeling of embankment dams are 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).

#### **C7.2.1. 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 envelope 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 (about 10–20 dams) and skewed toward 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 6 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 because of piping in the abutment. More recent studies have questioned that explanation.
- The Soil Conservation Service (SCS, 1985) 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 the 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 of dams 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 8 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 compared 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.
- Xu and Zhang (2009) used a database of 182 earth and rockfill dam failure cases. The study used multiparameter nonlinear regression 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).

#### **Key Point:**

- 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 candidate 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.

#### **C7.2.2. 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 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. 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. Many relationships have been published in the last 35 years. USBR, USACE, 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 describes 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 study described the breach shape for earthen dams as varying from triangular to trapezoidal as the breach progressed.
- Singh and Snorrason (1982) studied 20 dam failures and noted that breach width was generally between 2 and 5 times the dam height. They also found that the breach formation time was generally 15 minutes to 1 hour and the maximum overtopping depth before 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 factor 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. They used a multiparameter nonlinear regression 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 failure of dams higher than 15 meters (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 the 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., USACE Hydrologic Engineering Center River Analysis System

(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 (USBR, 2014; Brunner, 2013).

### **Key Points:**

- 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, the use of breach parameters computed using Xu and Zhang (2009) would need to be reviewed on a case-by-case basis.

#### **C7.2.2.1. 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 in the parameters themselves, but also in the overall result of the breach modeling efforts.

Numerous regression equations, summarized in the preceding sections of this RG, 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 dams that failed in cases 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 high. According to Wahl (2010), the best methods of breach width prediction are empirically derived parametric equations (e.g., USBR, 1988; Von Thun and Gillette, 1990; and Froehlich, 1995a). These methods were found to have uncertainties of about  $\pm$  one-third of an order of magnitude.

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 also have a very high uncertainty because of the 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  $\pm 1$  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.

#### **C7.2.2.2. Performing Sensitivity Analysis to Select 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 before 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 because of routing effects (e.g., flood attenuation and floodplain hydraulics).

Significant engineering judgment should be exercised in interpreting breach parameter and 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 is 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.

#### **Key Points:**

- Because of the large uncertainties, inconsistencies, and potential biases discussed above, applicants should not rely on a single method. Instead, applicants should compare the results of several models judged to be appropriate. They should justify their choice of candidate models and final parameter choices. Model and parameter uncertainty, as well as parameter sensitivity in final results, should be addressed.
- Studies have shown that failure time uncertainties can be quite large. Contributions to uncertainty include (1) observations of failure time in case studies generally originate from nonprofessional eyewitnesses and (2) clear and consistent definition of failure time is lacking across (and sometimes within) studies. Therefore, applicants 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.

#### **C7.2.3. 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, sediment transport, or both types of 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; Morris, 2013; Von Damme et al., 2012; Visser et al., 2012; Wu, 2013; Hunt et al., 2021), 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.

#### **Key Point:**

- 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.

#### **C8. Levee Breach Modeling**

For the last several decades, research on the breaching process has focused on the breaching of dams, not levees. As a result, little research has been performed on the breaching of levees. To overcome this issue, simple methods have been used. One such method is the Simplified Breach Analysis Method (Brunner 2016), which estimates the breach width using the mean velocity through the breach. A different method (ERDC, 2022) relies on soil erodibility and the height of the embankment. Although new methods are being developed, little consensus has been reached on this topic and estimated breach parameter values obtained by these methods would need to be reviewed by NRC staff on a case-by-case basis.

Additionally, methods developed for dams have been applied to levees. This can be problematic since the breaching process for levees can be quite different from that for earthen dams. The principal differences include (1) breach sensitivity to upstream and downstream conditions, (2) dimensionality of the outflow, and (3) 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 one dimensional. 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. 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.

### **Key Points:**

- 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), and levee condition. 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 design and construction characteristics similar to those of 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 an NPP site from an onsite or nearby levee will require two-dimensional modeling.

### **C9. 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 flood waters should be considered. However, as discussed in section C2, there may be situations in which 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. Commonly used methods include the following:

- Muskingum,
- Modified Puls (also known as storage routing), and
- 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 should 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 flood wave in a flat channel. The attenuation by hydrologic methods is approximate. A hydraulic model may be necessary to accurately predict the attenuation.

## **C9.1. 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, the analyst should 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.

### **C9.1.1. 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 computations move 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.

### **C9.1.2. Floodplain Storage**

If flood flows exceed the channel carrying capacity, water flows into overbank areas. Depending on the characteristics of the overbanks, that 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. One-dimensional flow models normally accomplish this 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  $K$  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.

### **C9.1.3. 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. They 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;  $u_0$  is the reference mean velocity and  $d_0$  = reference flow depth. (These reference values are average flow conditions of the inflow hydrograph.) They 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.

#### **C9.1.4. 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 the models that can account for backwater should be applied. For full networks, where the flow divides and possibly changes direction during the event, none of the simplified hydrologic models are recommended.

#### **C9.1.5. 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.

#### **C9.1.6. Availability of Calibration Datasets**

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 can be verified only with observed flows. Thus, these empirical models should be avoided if the watershed and channel are ungauged.

#### **Key Points:**

- The use of simplified hydrologic routing should be 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.

## **C9.2. Hydraulic Models**

As stated above, hydraulic routing methods are preferred when routing flood waves from a dam breach. Hydraulic routing provides more accuracy when modeling flood waves from dam breach because it includes terms that other methods neglect.

Many readily available dynamic (unsteady flow) hydraulic models have been used for dam breach outflow hydrograph computation and downstream routing. In general, the hydraulic routing methods are applicable to the dam breach flood wave routing analysis. Recent case studies of dam break flood routing applying widely used 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 two-dimensional analysis in areas of particular interest or sensitivity.

### **Key Points:**

- For estimating inundation at or near the NPP site, two-dimensional models are generally recommended to capture spatial variations. 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. The likely variation in maximum water surface stage at the NPP site should be estimated to account for this uncertainty in the rating curve.

## **C9.3. 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 RG. 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.

### **Key Point:**

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

## **C9.4. Inundation Mapping**

Inundation maps have a variety of uses including emergency action plans (EAPs), 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 RG, inundation maps assist in identifying SSCs important to safety that may require protective or mitigative measures from flooding due to dam breach.

The use of 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. The NRC does not endorse a particular software package.

Several federal agencies have developed guidance documents on producing inundation maps (FEMA, 2013; USACE, 2018).

**Key Points:**

- 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, and
  - velocity data across the NPP site and nearby areas.

## **D. IMPLEMENTATION**

The NRC staff may use this RG as a reference in its regulatory processes, such as licensing, inspection, or enforcement. However, the NRC staff does not intend to use the guidance in this RG to support NRC staff actions in a manner that would constitute backfitting as that term is defined in 10 CFR 50.109, “Backfitting,” and as described in NRC Management Directive 8.4, “Management of Backfitting, Forward Fitting, Issue Finality, and Information Requests” (USNRC, 2019), nor does the NRC staff intend to use the guidance to affect the issue finality of an approval under 10 CFR Part 52, “Licenses, Certifications, and Approvals for Nuclear Power Plants.” The staff also does not intend to use the guidance to support NRC staff actions in a manner that constitutes forward fitting as that term is defined and described in Management Directive 8.4. If a licensee believes that the NRC is using this RG in a manner inconsistent with the discussion in this Implementation section, then the licensee may file a backfitting or forward fitting appeal with the NRC in accordance with the process in Management Directive 8.4.

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<sup>1</sup> Publicly available NRC published documents are available electronically through the NRC Library on the NRC’s public website at <http://www.nrc.gov/reading-rm/doc-collections/> and through the NRC’s Agencywide Documents Access and Management System (ADAMS) at <http://www.nrc.gov/reading-rm/adams.html>. For problems with ADAMS, contact the Public Document Room staff at 301-415-4737 or (800) 397-4209, or email [pdr.resource@nrc.gov](mailto:pdr.resource@nrc.gov). The NRC Public Document Room (PDR), where you may also examine and order copies of publicly available documents, is open by appointment. To make an appointment to visit the PDR, please send an email to [pdr.resource@nrc.gov](mailto:pdr.resource@nrc.gov) or call 1-800-397-4209 or 301-415-4737, between 8 a.m. and 4 p.m. eastern time (ET), Monday through Friday, except Federal holidays.

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## Appendix A: 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, powerplants, tunnels, etc.

**Attenuation.** A decrease in amplitude of the seismic waves with distance due to geometric spreading, energy absorption, and scattering, or a decrease in the amplitude of a flood wave as the result of channel geometry and energy loss.

**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 2,500 feet/second.

**Bedrock motion parameters.** Numerical values representing vibratory ground motion, such as particle acceleration, velocity, and displacement, frequency content, predominant period, spectral intensity, and a duration that define a design earthquake. (These may also be used in a more general sense for ground motion.)

**Body wave.** Waves propagated in the interior of the earth (i.e., the compression (P) and shear (S) waves of an earthquake).

**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 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 later to be filled with impervious material so as 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 storage or control of water.

**Ambursen dam.** A buttress dam in which the upstream part is a relatively thin flat slab usually made of reinforced concrete.

**Arch dam.** A concrete, masonry, or timber dam with the alignment curved upstream so as to transmit a major part of the water load to the abutments.

**Buttress dam.** A dam consisting of a watertight part supported at intervals on the downstream side by a series of buttresses. Buttress dams can take many forms, such as a flat slab or massive head buttress.

**Crib dam.** A gravity dam built up of boxes, crossed timbers or gabions, filled with earth or rock.

**Diversion dam.** A dam built to divert water from a waterway or stream into a different watercourse.

**Double curvature arch dam.** An arch dam that is curved both vertically and horizontally.

**Earthen dam.** An embankment dam in which more than 50 percent of the total volume is formed of compacted earth layers with soil particles generally smaller than 3 inches.

**Embankment dam.** Any dam constructed of excavated natural materials, such as both earthfill and rockfill dams, or of industrial waste materials, such as a tailings dam.

**Gravity dam.** A dam constructed of concrete, masonry, or both, which relies on its weight and internal strength for stability.

**Hollow gravity dam.** A dam constructed of concrete, masonry, or both on the outside but having a hollow interior and relying on its weight for stability.

**Hydraulic fill dam.** An earth dam constructed of materials, often dredged, which are conveyed and placed by suspension in flowing water.

**Masonry dam.** Any dam constructed mainly of stone, brick, or concrete blocks pointed with mortar. A dam having only a masonry facing should not be referred to as a masonry dam.

**Multiple arch dam.** A buttress dam comprising a series of arches for the upstream face.

**Overflow dam.** A dam designed to be overtopped.

**Rockfill dam.** An embankment dam in which more than 50 percent of the total volume is made up of compacted or dumped cobbles, boulders, rock fragments, or quarried rock generally larger than 3 inches.

**Roller compacted concrete dam.** A concrete gravity dam constructed by the use of a dry mix concrete transported by conventional construction equipment and compacted by rolling, usually with vibratory rollers.

**Rubble dam.** A stone masonry dam in which the stones are unshaped or uncoursed.

**Saddle dam.** A subsidiary dam of any type constructed across a saddle or low point on the perimeter of a reservoir.

**Dam failure.** Catastrophic type of failure characterized by the sudden, rapid, and often uncontrolled release of impounded water or the likelihood of such a 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 the nuclear power plant (NPP) site:

**Inconsequential dam.** A dam identified by Federal or State agencies as having minimal or no adverse failure consequences beyond the owner's property; a dam that can be shown to have minimal or no adverse downstream failure consequences.

**Noncritical dams.** A dam or (or set of dams) that can be shown to have low flooding impacts at the NPP site (i.e., flood elevations below systems, structures, and components important to safety) using simplified, conservative methods.

**Potentially critical dam.** A dam or (or set of dams) that cannot be shown to have low flooding impacts at the NPP site (i.e., flood elevations below systems, structures, and components important to safety) using simplified, conservative methods.

**Critical dam.** A dam or (or set of dams) that is shown to have flooding impacts at the NPP site (i.e., flood elevations at or above systems, structures, and components important to safety).

**Damping.** Resistance that reduces vibrations by energy absorption. There are different types 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 runoff. The determination will generally include the results of meteorologic studies that realistically combine wind velocity, duration, direction, and seasonal distribution characteristics.

**Deterministic methodology.** A method in which the chance of occurrence of the variable involved is ignored and the method or model used is considered to follow a definite law of certainty, and not probability.

**Dike.** See **saddle dam**.

**Drain, blanket.** A layer of pervious material placed to facilitate drainage of the foundation 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 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.

**Earthquake, operating basis.** The earthquakes for which the structure is designed to resist and remain operational. It reflects the level of earthquake protection desired for operational or economic reasons and may be determined on a probabilistic basis considering the regional and local geology and seismology.

**Earthquake, safety evaluation.** The earthquake, expressed in terms of magnitude and closest distance from the dam site or in terms of the characteristics of the time history of free-field ground motions, for which the safety of the dam and critical structures associated with the dam are to be evaluated. In many cases, this earthquake will be the maximum credible earthquake to which the dam will be exposed. However, in other cases where the possible sources of ground motion are not readily apparent, it may be a motion with prescribed characteristics selected on the basis of a probabilistic assessment of the ground motions that may occur in the vicinity of the dam. To be considered safe, it should be demonstrated that the dam can withstand this level of earthquake shaking without release of water from the reservoir.

**Earthquake, synthetic.** Earthquake time history records developed from mathematical models that use white noise, filtered white noise, and stationary and nonstationary filtered white noise, or theoretical seismic source models of failure in the fault zone. (White noise is random energy containing all frequency

components in equal proportions. Stationary white noise is random energy with statistical characteristics that do not vary with time.)

**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 vertically 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**.

**Fault.** A fracture or fracture zone in the earth along which there has been displacement of the two sides relative to one another and which is parallel to the fracture.

**Fault, active.** A fault which, because of its present tectonic setting, can undergo movement from time to time in the immediate geologic future.

**Fault, capable.** An active fault that is judged capable of producing macroearthquakes and exhibits one or more of the following characteristics:

Movement at or near the ground surface at least once within the past 35,000 years.

Macroseismicity (3.5 magnitude or greater) instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.

A structural relationship to a capable fault such that movement on one fault could be reasonably expected to cause movement on the other.

Established patterns of microseismicity that define a fault, with historic macroseismicity that can reasonably be associated with the fault.

**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 1-percent-chance flood or expressed as a fraction of the probable maximum flood or other reference flood.

**Flood, safety evaluation.** The largest flood for which the safety of a dam and appurtenant structure is to be evaluated.

**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 a specified stillwater (or other) reservoir surface elevation 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. Specific types of gates include the following:

**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.** 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 a “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 because of 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 structure.

**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 miss-operation of the dam or appurtenances. Impacts may be for a defined area downstream of a dam from flood waters 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 because of 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 1 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.** 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.

**Intensity, seismic.** A numerical index describing the effects of an earthquake on man, manmade structures, or other features of the earth's surface.

**Inundation map.** A map showing areas that would be affected by flooding from releases from a dam's reservoir. The flooding may be from either controlled or uncontrolled releases or as a result of a dam failure. A series of maps for a dam could show the incremental areas flooded by larger flood releases.

**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, powerplant, 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 watertightness, 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, because of the buildup and maintenance of high pore water pressures, which reduce 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 so as 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.

**Magnitude, body wave.** The magnitude of an earthquake measured as the common logarithm of the maximum displacement amplitude (microns) and period (seconds) of the body waves.

**Magnitude, Richter or local.** The magnitude of an earthquake measured as a common logarithm of the displacement amplitude, in microns, of a standard Wood-Anderson seismograph located on firm ground 100 kilometers from the epicenter and having a magnification of 2,800; a natural period of 0.8 second; and a damping coefficient of 80 percent.

**Magnitude, surface wave.** The magnitude of an earthquake measured as the common logarithm of the result of the maximum mutually perpendicular horizontal displacement amplitudes, in microns, of the 20-second period surface waves.

**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 realistically combine wind velocity, duration, direction, fetch, and seasonal distribution characteristics.

**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 for prevention of overtopping caused by wave runup.

**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 to measure water levels or pore water pressures in embankments, foundations, abutments, soil, rock, or concrete.

**Piping.** The progressive development of internal erosion by seepage.

**Predominant period.** The period(s) at which maximum spectral amplitudes are shown on response spectra. Normally, acceleration response spectra are used to determine the predominant period(s) of the earthquake ground motion.

**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 and in which water can be stored.

**Reservoir regulation procedure (rule curve):** The compilation of operating criteria, guidelines, and specifications that govern the storage and release function of a reservoir. The compilation 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, displacement response, or all three 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.

**Scaling.** An adjustment to an earthquake time history or response spectrum where the amplitude of acceleration, velocity, displacement, or all three 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.

**Seismic moment (Mo).** A measure of the earthquake size containing information on the rigidity of the elastic medium in the source region, average dislocation, and area of faulting. It determines the amplitude of the long-period level of the spectrum of ground motion. It is calculated as follows:

$$Mo = \text{Shear Modulus of Faulted Rock (Dynes/cm}^2\text{)} \times \text{Length of Fault Rupture Zone (cm)} \times \text{Width of Fault (cm)} \times \text{Displacement of Fault (cm)}$$

**Seismotectonic province.** A geologic area characterized by similarity of geologic structure and tectonic and seismic history.

**Seismotectonic source area(s).** An area or areas of known or potential seismic activity that may lack a specific identifiable seismotectonic structure.

**Seismotectonic structure.** An identifiable dislocation or distortion within the earth's crust resulting from recent tectonic activity or revealed by seismologic or geologic evidence.

**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.

**Significant wave height.** Average height of the one-third highest individual waves. May be estimated from windspeed, fetch length, and wind duration

**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, displacement, or all three to be used in the analyses of the structure.

**Spectrum intensity.** The integral of the pseudovelocity response spectrum taken over the range of structural vibration periods from 0.1 to 2.5 seconds.

**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.** 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.** A spillway 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 a “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 so as 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 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. Definitions of specific types of storage in reservoirs are as follows:

**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.

**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 because of fluctuations in the outflow from the structures of a dam and because of 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.”

**Top of Dam.** The uppermost surface of a dam proper, not taking into account any crowning for settlement or any structures that are not part of the main water retaining structure.

**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 so as 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 runup.** 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.