From:	Miller, Ed
To:	"RILEY, Jim"
Subject:	Comments on Dam Failure White Paper from Public Meeting
Date:	Friday, October 26, 2012 3:14:00 PM
Attachments:	Dam Failure Rev D.docx.docx.docx

Jim,
  This is the file from the meeting. It has the changes that were entered during the discussion.

Ed Miller
415-2481

POST-FUKUSHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1
Supplemental Guidance for the Evaluation of Dam Failures

Contents

1 Background

In response to the nuclear fuel damage at the Fukushima-Daiichi power plant due to the March 11, 2011 earthquake and subsequent tsunami, the United States Nuclear Regulatory Commission (NRC) is requesting information pursuant to Title 10 of the Code of Federal Regulations, Section 50.54 (f) (10 CFR 50.54(f) or 50.54(f)). As part of this request, licensees will be required to reevaluate flooding hazards, per present-day guidance and methodologies for early site permits and combined license reviews, to assess margin at safety-related structures, systems, components (SSCs) and effectiveness of current licensing basis (CLB) protection and mitigation measures. The request is associated with the NRC's Post-Fukushima Near-Term Task Force (NTTF) Recommendation 2.1 for flooding, approved by the Commission in SECY 11-0137, *Prioritization of Recommended Actions to be Taken in Response to Fukushima Lessons Learned*, dated December 15, 2011.

- **Requests in the March 12, 2012 50.54(f) Letter**

  Requested Action:

  Addressees are requested to perform a reevaluation of all appropriate external flooding sources, including the effects from local intense precipitation on the site, probable maximum flood (PMF) on stream and rivers, storm surges, seiches, tsunami, and dam failures. It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for ESP and Cal reviews including current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The requested information will be gathered in Phase 1 of the NRC staffs two phase process to implement Recommendation 2.1, and will be used to identify potential vulnerabilities.

  For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard with the hazard evaluation.

  Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them. The scope of the integrated assessment report will include full power operations and other plant configurations that could be susceptible due to the status of the flood protection features. The scope also includes those features of the ultimate heat sinks (UHS) that could be adversely affected by the flood conditions and lead to degradation of the flood protection (the loss of UHS from non-flood associated causes are not included). It is also requested that the integrated assessment address the entire duration of the flood conditions.

  Requested Information:

  The NRC staff requests that each addressee provide the following information. Attachment 1 provides additional information regarding present-day methodologies and guidance used by the NRC staff performing ESP and COL reviews. The attachment also provides a stepwise approach for assessing the flood hazard that should be applied to evaluate the potential hazard from flood causing mechanisms at each licensed reactor site.

  1. *Hazard Reevaluation Report*

     Perform a flood hazard reevaluation. Provide a final report documenting results, as well as pertinent site information and detailed analysis. The final report should contain the following:

a. *Site information related to the flood hazard. Relevant SSCs important to safety and the UHS are included in the scope of this reevaluation, and pertinent data concerning these SSCs should be included. Other relevant site data includes the following:*

   i. *detailed site information (both designed and as-built), including present-day site layout, elevation of pertinent SSCs important to safety, site topography, as well as pertinent spatial and temporal data sets*
   ii. *current design basis flood elevations for all flood causing mechanisms*
   iii. *flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance*
   iv. *changes to the watershed and local area since license issuance*
   v. *current licensing basis flood protection and pertinent flood mitigation features at the site*
   vi. *additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results, etc.)*

b. *Evaluation of the flood hazard for each flood causing mechanism, based on present-day methodologies and regulatory guidance. Provide an analysis of each flood causing mechanism that may impact the site including local intense precipitation and site drainage, flooding in streams and rivers, dam breaches and failures, storm surge and seiche, tsunami, channel migration or diversion, and combined effects. Mechanisms that are not applicable at the site may be screened-out; however, a justification should be provided. Provide a basis for inputs and assumptions, methodologies and models used including input and output files, and other pertinent data.*

c. *Comparison of current and reevaluated flood causing mechanisms at the site. Provide an assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood causing mechanism. Include how the findings from Enclosure 4 of this letter (i.e., Recommendation 2.3 flooding walkdowns) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, include how this finding was determined.*

d. *Interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment described below, if necessary.*

e. *Additional actions beyond Requested Information item 1.d taken or planned to address flooding hazards, if any.*

2. *Integrated Assessment Report*

   For the plants where the current design basis floods do not bound the reevaluated hazard for all flood causing mechanisms, provide the following:

   a. *Description of the integrated procedure used to evaluate integrity of the piant for the entire duration of flood conditions at the site.*

b. *Results of the plant evaluations describing the controlling flood mechanisms and its effects, and how the available or planned measures will provide effective protection and mitigation. Discuss whether there is margin beyond the postulated scenarios.*

c. *Description of any additional protection and/or mitigation features that were installed or are planned, including those installed during course of reevaluating the hazard. The description should include the specific features and their functions.*

d. *Identify other actions that have been taken or are planned to address plant-specific vulnerabilities.*

- **Flooding Evaluation**

Prior to the March 2011 Fukushima-Daiichi earthquake/tsunami events, the NRC standard for flood estimation was the 1977 version of Regulatory Guide (RG) 1.59, and its appendices, and Section 2.0 of the Standard Review Plan (NUREG-0800).

In the 50.54(f) letter, the NRC is requesting updated flooding hazard information using ‘present-day regulatory guidance and methodologies to review early site permits (ESPs) and combined license (COL) applications’. Although the update to RG 1.59 is not complete, the NRC is considering NUREG/CR-7046, “Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America”, November 2011, as representing present-day methodologies for flooding evaluations.

NUREG/CR-7046 describes present-day methodologies and technologies that can used to estimate design-basis floods at nuclear power plants for a range of flooding mechanisms, including rivers/streams, dam failures, local intense precipitation (local/site runoff), storm surge, seiche, ice-induced flooding, channel migration/diversion, and combined-effects floods (for dependent or correlated events).

NUREG/CR-6966 (“Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America”) is referenced as a guide for the evaluation of tsunamis.

- **Dam Failures**

Mechanisms that cause dams to fail include overtopping of an unprotected portion of the dam during a significant hydrologic event, piping, liquefaction of foundation from seismic activity, slope/stability issues, uncontrolled seepage, and other deficiencies. The resulting flood waves, including those from domino-type or cascading dam failures, should be evaluated for each site as applicable. Dams of interest to a nuclear site are those located within the upstream watershed of an adjacent stream/river. Water storage and water control structures (such as onsite cooling or auxiliary water reservoirs and onsite levees) that may be located at or above SSCs important to safety should also be evaluated.

Comment [g1]: Consider rewording so clear that all failure modes are considered.

Acceptable models and methods used to evaluate the dam failure and the resulting effects should be appropriate to the type of failure mechanism. References provided herein include acceptable guidance documents to developing dam break hydrographs. Unsteady-flow (e.g. HEC-RAS) or 2D hydraulic models are frequently used to route dam breach hydrographs to the site. Recent analyses completed by entities with appropriate jurisdiction for dams may be incorporated into the analysis. ~~Dam~~ Overtopping dam breach/failure scenarios should include coincidental failure with the peak PMF and domino-type or cascading dam failures unless an engineering justification is provided showing that a failure mode is not credible as part of the refined site specific hazard analysis. Part of the HHA approach may include an assumption that all dams fail, regardless of the cause; timed to produce the worse possible flooding conditions at the site (including compounding flows from cascading failures of dams in series).

Comment [initials2]: FFTF change

2 Definitions

Breach Parameters (adapted from Wahl 1998):

> **Breach Depth/Height** – The vertical extent of the breach, measured from the dam crest to the invert of the breach. Some publications cite the reservoir head on the breach, measured from the reservoir water surface to the breach invert.
>
> **Breach Width** – The ultimate width of the breach, reported either as the average breach width or breach width at the top or bottom of the breach opening.
>
> **Breach Side Slope Factor** – The side slope, typically expressed in terms of 'per unit height' (Z units horizontal to 1 unit vertical or Z:1), of an idealized trapezoidal-shaped breach.
>
> **Breach Initiation Time** – The breach initiation time begins with the first flow over or through a dam that will initiate warning, evacuation, or heightened awareness of the potential for dam failure. The breach initiation time ends at the start of the breach formulation phase.
>
> **Breach Formation Time (or Time of Failure)** – Breach formation time (or time to failure) has been defined in various ways but generally follow the definition used in the NWS DAMBRK model as: *The duration of time between the first breaching of the upstream face of the dam until the breach is fully formed. For overtopping failures, the beginning of breach formulation is after the downstream face of the dam has eroded away and the resulting crevasse has progressed back across the width of the dam crest to reach the upstream face.*

Combined Effect Flood – A combined effect is a plausible combination of dependent flooding mechanisms occurring simultaneously.

Critical and Non-Critical Dams ~~Failure~~ – A 'critical' dam is an upstream dam whose failure is shown, through an engineering evaluation, to have a significant affect at the site. Conversely, a 'non-critical' dam is an upstream dam whose failure is shown to not have a significant affect at the site. The critical (significant affect) – non-critical (insignificant affect) distinction is only used to assist the licensee in focusing refinement efforts on a 'critical' sub-set of dams and differs from the screening process. The final failure scenario would include failure of 'non-critical' dams using conservative breach parameters. Screening, defined further below, is used to eliminate small/remote upstream dams from further consideration.

Dam – A dam is an artificial barrier used to impound water for multiple possible functions, including flood control (attenuation), recreation, water supply, hydroelectric, sediment storage, aquatic habitat, stormwater (quantity/quality) management, or a combination thereof.

Dam Breaches/Failures – A breach/failure, which can be caused by several possible mechanisms including overtopping, seismic activity, slope failures, etc., can produce a floodwave with high flow rates, velocities, and depths. The flood wave attenuates as it moves downstream causing the peak flow rates, velocities, and flood depths to dissipate. Failure of a dam could cause the formation of a floodwave that could threaten lives and property downstream of the barrier. Floodwaves from dam failures of (or other upstream structures) are distinct from wind-generated waves.

Design Basis Flood – A design-basis flood is a plant-specific phenomenon caused by one or an appropriate combination of several hydrometeorological, geoseismic, or structural-failure phenomena, which results in the most severe hazards to structures, systems, and components (SSCs) important to the safety of a nuclear power plant.

Comment [g3]: consider mentioning any downstream dams that may also be critical to site.

Comment [initials4]: FFTF – Added definition

Failure Mode – The means or conditions under which a dam fails. For the purpose of this paper, three failure modes are being considered: hydrologic (induced by an extreme precipitation event), seismic (induced by an earthquake), and 'sunny-day' (no initiating event external to the dam).~~–~~

Flood Warning – Alert systems notifying people and/or facilities along low-lying areas that flooding is possible, likely, and/or imminent. Flood warning time is the time between the alert and arrival of floods and is dependent on the flooding characteristics. Flash floods are typically associated with fast-moving, short-duration, highly-intense storms affecting streams and drainage systems with relatively small watersheds, and generally have short warning times. Warning time for dam failure flooding can be very short and unpredictable, depending on the velocity of the flood wave, the dam's distance from the point of interest, type of dam, and the time taken by the dam owner to notify emergency officials.

Negligible Effects of Dam Failure – Screening upstream dams from consideration in the dam failure scenario development process involves establishing a 'negligible' threshold for increase in stage, discharge, and/or volume at the site. 'Negligible' threshold should be developed on a site-specific basis and may include such considerations as margin of error in the hydraulic analysis.

~~**Non-Critical Dam Failure –**~~

Probable Maximum Flood (PMF) – The PMF is a hypothetical flood (peak discharge, volume, and hydrograph shape) considered to be the most severe reasonably possible, based on comprehensive hydrometeorological application of Probable Maximum Precipitation (PMP) and other hydrologic factors favorable for maximum flood runoff, such as sequential storms, and snowmelt~~, and dam failure~~. ~~Typically, several PMP scenarios are evaluated to establish the bounding (largest possible) PMF, including the all-season (summer) PMP and winter (seasonal) PMP combined with snowmelt.~~

Probable Maximum Precipitation (PMP) – ~~The estimated depth of precipitation for a given duration, drainage area, and time of year for which there is virtually no risk of exceedance. The probable maximum precipitation for a given duration and drainage area approximates the theoretical maximum that is physically possible within the limits of contemporary hydrometeorological knowledge and techniques.~~The greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location and at a particular time of the year, with no allowance made for (future) long term climatic trends.

Riverine Flooding – A watershed's response to a rainfall-runoff event that produces overbank flow at a given location. Riverine flooding adjoining the site, associated with the PMF, is determined by applying the PMP and other hydrologic factors to the watershed draining to the site location.

~~**Safety Margin** – Difference between probable maximum flood hazard conditions and acceptance criteria (e.g. allowable head on a door seal minus probable maximum flood level; time needed to construct temporary cofferdam and minimum flood warning time; etc.~~)

Screening – Screening is the process in which the licensee can eliminate upstream dams from further consideration, in developing dam failure scenarios, because of low differential head, small volume, distance from plant site, and major intervening natural or reservoir detention capacity. Screening dams is different than process for distinguishing 'critical' and 'non-critical' dams. See associated definition above.

Standard Project Flood (SPF) – The US Army Corps of Engineers' (USACE's) definition of the SPF is floods that produce flow rates generally 40% to 60% of the PMF. Historically, the USACE established the SFP based on the flood of record. More recently, risk-based analysis procedures are used to establish the SPF.

Comment [initials5]: FFTF – Added definition

Comment [A6]: FFTF – Added definition

Comment [g7]: Recommend use NRC definition of PMF & PMP
FFTF - OK

Comment [g8]: This definition does not appear to be used. Consider deleting.
FFTF - OK

Also consider addition of a definition for "failure mode"

Comment [initials9]: Later

3 Purpose

This paper is intended to clarify how dam failure should be considered when reevaluating the bounding PMF in response to Enclosure 2 (Recommendation 2.1: Flooding) of the March 12, 2012 50.54(f) letter. This paper provides added guidance to supplement the NUREG/CR-7046, Sections 3.4 and 3.9 and Appendix H.2, related to dam failure considerations. The goal is to achieve a realistic, physics-based, but conservative analysis of flooding. Per NUREG/CR-7046, when dams are present upslope of the site or within the watershed of an adjacent stream/river, three failure modes should be evaluated independently: ~~to create three corresponding flooding fail scenarios~~modes.

1. **Hydrologic Failure:** Dam failure induced by an extreme precipitation/snowmelt event within the dam's upstream watershed; typically associated with overtopping of an unprotected portion of the dam.
2. **Seismic-Induced Failure:** Dam failure induced by an earthquake that causes weakening of the dam's structural components, embankment, foundation, and/or abutments. ~~Section 9.2.1.2 in ANSI/ANS 2.8-1992 states~~:

   ~~*Although the principal cause of a dam failure might be from an earthquake, it is possible that the peak of a flood could coincide with the few minutes' duration of the earthquake. The higher of the following two alternative combinations is an adequate design base for seismic dam failure floods.*~~

   ~~*Alternative I*~~
   1. ~~*25-yr flood.*~~
   2. ~~*Dam failure caused by the safe shutdown earthquake (SSE) coincident with the peak of flood.*~~
   3. ~~*2-yr wind speed applied in the critical direction.*~~

   ~~*Alternative II*~~
   1. ~~*One-half PMF or 500-yr flood, whichever is less.*~~
   2. ~~*Dam failure caused by the operating basis earthquake (OBE) coincident with the peak of flood.*~~
   3. ~~*3.*~~ *1.* ~~*2-yr wind speed applied in the critical direction*~~.
3. **Sunny-Day Failure:** A 'sunny-day' dam failure is not associated or concurrent with an initiating event (such as an extreme flood or earthquake) and may result from a structural, geotechnical, or operational deficiency. Sunny-day failures are typically associated with short warning times. Assumptions for initial water levels and failure modes should be provided.

The resulting scenario for each failure mode is considered independently because each may produce bounding parameters at the site. For example, the 'hydrologic' failure mode may produce the highest volume, peak flow rate, and peak flood level. The 'seismically-induced' failure mode may produce high flows from simultaneous failures and rainfall events, and short warning times. The 'sunny-day' failure mode may produce the shortest warning time and highest dynamic loading condition. Other items worth noting:

- **Loss of Ultimate Heat Sink due** to **Flooding-Induced Downstream Dam Failure:** The NRC is requesting that the Recommendation 2.1: Flood Hazard Reevaluations include an evaluation of the effects of flooding on downstream dams that are used to impound the ultimate heat sink (UHS).

Comment [g10]: Consider rewording
FFTF - OK

Comment [g11]: Recommend deleting
FFTF - OK, but without this information what combinations of seismic and precipitation must be considered?

Comment [initials12]: **FFTF - OK**

- **Security Threats:** ~~It is assumed that f~~Failures from modes other than natural hazards (e.g. terrorism) ~~do not need to be considered~~are not within the scope of Recommendation 2.1, Flooding Reevaluations.

Comment [initials13]: FFTF - OK

4 Approach

4.1 Screening Upstream Dams with Negligible effect of Failure at the Site

Section 5.5 of ANS 2.8 states *"All dams above the plant site shall be considered for potential failure, but some may be eliminated from further consideration because of low differential head, small volume, distance from plant site, and major intervening natural or reservoir detention capacity"*. The purpose of this section is to provide additional guidance for assessing which dams can be screened as having negligible effect of failure at the site and eliminated from further consideration. All other dams should be considered potentially critical dams and subjected to further evaluation.

National and state dam inventories and classification systems can be used to identify dams within the watershed of an adjacent stream/river and obtain critical characteristics for each dam (location, height, and volume). Most states use a system to classify the size and hazard potential of each dam that can assist in the screening process as well. In most cases, dams immediately upslope from the site (not in line with an adjacent stream/river) and very large dams within the watershed should not be screened.

A justification for screening upstream dams should be developed on a site-specific basis and included in the Flood Hazard Reevaluation report. Several optional methods discussed below, and in more detail in Appendix A, provide a quantitative basis for screening upstream dams. The methods are presented in a HHA-type gradation of conservatism and applicable to the hydrologic and seismically-induced failure modes. The process for evaluating sunny-day failure does not require screening since it only involves identifying the worst-case individual or cascading failure scenario. Note that other methods can be used and will be reviewed on a case-by-case basis. The screening process involves establishing a 'negligible increase' threshold at the site. See Section 2 for definition of 'negligible effects of dam failure'.

~~Common to all methods is establishing a 'negligible' increase (threshold) in stage, discharge, and/or volume at the site. The threshold for 'negligible' effect should be developed on a site specific basis. Criteria for establishing a threshold could include no change in impact to plant SSCs, no change in impact on protection or mitigation measures, within the margin of error in hydraulic model (stage-discharge function), etc.~~

Comment [g14]: Mention that other methods can be used and will be reviewed on a case-by-case basis
FFTF – OK, see previous paragraph

1. Volume Method: Estimate and sum the storage volume for <u>all</u> upstream dams in the watershed, assuming pool levels are at the top of each dam. Develop a stage-storage function for the river/floodplain system at the site assuming floodwaters have already reached plant grade. That is, do not credit volume in the channel and/or floodplain below plant grade. With available LiDAR or USGS digital elevation models (DEM), GIS tools can be used to develop the stage-storage function at the site. Developing the stage-discharge function should exclude remote floodplain storage areas that could not be accessed by overbank floodwaters. Compute the difference in elevation, starting at plant grade, by applying the total storage volume for all upstream dams to the stage-storage function. This calculation is representative of having the total upstream storage volume instantaneously and simultaneously transferred to the site with zero flow in the stream/river. If the resulting elevation difference exceeds a negligible threshold, iteratively repeat the process to segregate potentially critical dams from dams with negligible incremental effect of failure at the site.

Comment [g15]: Recommend adding a definition of screening, negligible, critical, and non-critical
FFTF – OK

Comment [g16]: Consider adding a title to each of these methods
FFTF - OK

Comment [g17]: Clarify that this is looking for incremental change
FFTF - OK

2. Peak Outflow without Attenuation Method: Estimate and sum the peak failure outflows for all upstream dams. Assume failure of all dams reach the site instantaneously and simultaneously, ignoring attenuation. Compare the peak outflow sum to the established discharge increase threshold. Or, using an available stage-discharge function (from available hydraulic models or USGS streamflow rating curves to identify a conservative determination of incremental effects for screening purposes), estimate the increase in flood stage, above plant grade, corresponding to the peak failure outflow sum and compare this stage increase to the established threshold value. If the resulting discharge or stage difference exceeds the threshold value, iteratively repeat the process to segregate potentially critical dams from dams with negligible effect of failure at the site.

Comment [g18]: How do you account for uncertainty
FFTF – The screening process is a conservative comparison to determine what dams can be excluded. It evaluates incremental effects; it does not develop an absolute value for flooding levels. Since this is intended to be a conservative high level comparison, uncertainties are not important to consider.

See the proposed change.

3. Peak Outflow with Attenuation Method: Using the established threshold value for increase in peak discharge at the site, develop a relationship between the size of dam (e.g. height and/or volume) and distance to site based on applicable regression equations for peak flow and attenuation, assuming failure occurs at each dam at full pool. (Section 3.7 and Appendix D.) The resulting curve can be used to judge upstream dams having negligible effect of failure at the site. (See example illustration in Figure 1.) Regression equations for attenuation (e.g. USBR (1982) or NWS (1991)) should be tested against available models and/or studies to justify their applicability to the adjacent river/floodplain system.

   If more than one dam exists upstream of the site, the dams can be grouped into zones or clusters with comparable size and proximity to the site. The discharge increase tolerance should be divided by the number of dams in each zone to establish threshold value per dam.

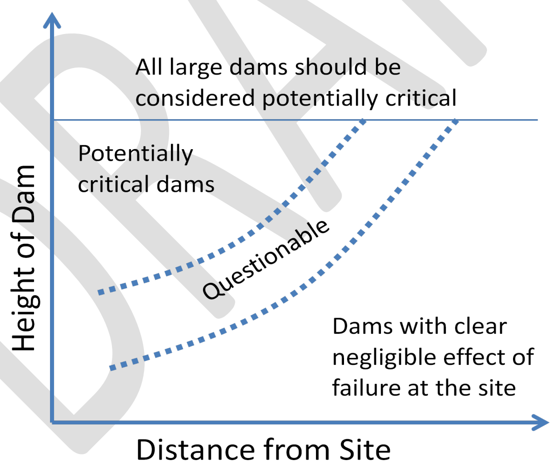


Figure 1 – Example Illustration of Dam Failure Evaluation Screening Approach (Method 3)

4. Rainflow Runoff Method: Use an available rainfall-runoff-routing model (e.g. HEC-HMS) to develop multiple failure scenarios and combinations for hypothetical dams, representative of the number, size, and proximity of the actual upstream dams in the watershed. (Setting up hypothetical dams in a rainfall-runoff-routing model involves much less effort than coding in actual dams.) The hypothetical scenarios should include representative situations of dams in series and cascading

failures. Iteratively remove hypothetical dams, larger to smaller, to the point where the incremental difference in discharge at the site is less than the established threshold value. Size and distance plots, differentiating between dams removed and remaining in the model, could provide a basis for screening dams having a negligible effect at the site. The advantage to this approach is it better represents the affects of multiple upstream dam failures and attenuation to the site. See example illustration in Figure 2.

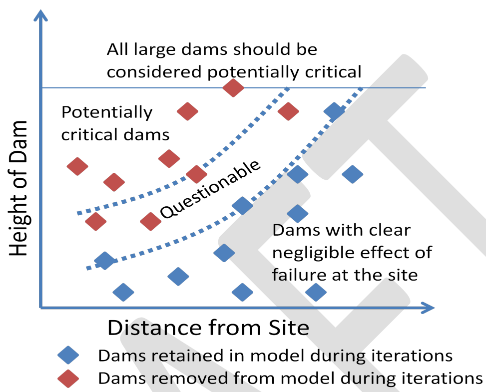


Figure 2 – Example Illustration of Dam Failure Evaluation Screening Approach (Method 4)

Discuss the use of the DSS-Wise tool developed by NCCHE for screening.

Comment [initials19]: FFTF – OK, later

4.2 Individual and Cascading Failure Scenarios

Section 3.4 of NUREG/CR-7046 states that *"dam failure scenarios, particularly those related to cascading dam failures, should be carefully analyzed and documented to establish that the most severe of the possible combinations has been accounted for"*. Typically, two scenarios of upstream dam failure should be considered:

1. **Failure of Individual Dams:** One or more dams may be located upstream of 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. Reasons for failing individual dams depends on the failure mode:

   a. **Hydrologic Failure:** It is likely that a large flood on one tributary would coincide with similar large floods in adjoining tributaries.

   b. **Seismically-Induced Failure:** It is possible that simultaneous failure of individual dams could occur during an earthquake. As discussed further below, individual seismic failure scenarios should consider the location and attenuation of the earthquake.

   c. **Sunny-Day Failure:** Failure of multiple individual dams on separate tributaries is not applicable to the sunny-day failure mode since it is unreasonable to assume that individual dams on separate tributaries would simultaneously fail without an initiating external natural hazard event.

Comment [g20]: Consider clarifying this statement
FFTF - OK

2. **Cascading or Domino-Like Failures of Dams:** Failure of an upstream dam may generate a flood that would become an inflow into the reservoir impounded by a downstream dam and may result in 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 cascades 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 this appendix. The most severe flood at the site resulting from these cascades should be used to determine the governing flood.

Appendix D, Part D.1, of NUREG/CR-7046 provides additional guidance and examples for developing reasonable individual and cascading failure scenarios.

4.3 Overview of HHA Approach for Dam Failure

NUREG/CR-7046 describes the Hierarchical Hazard Assessment (HHA) approach as:

> *"a progressively refined, stepwise estimation of site-specific hazards that evaluates the safety of SSCs with the most conservative plausible assumptions consistent with available data. The HHA process starts with the most conservative simplifying assumptions that maximize the hazards from the probable maximum event for each natural flood-causing phenomenon expected to occur in the vicinity of a proposed site. The focus of this report is on flood hazards. If the site is not inundated by floods from any of the phenomena to an elevation critical for safe operation of the SSCs, a conclusion that the SSCs are not susceptible to flooding would be valid, and no further flood-hazard assessment would be needed."*

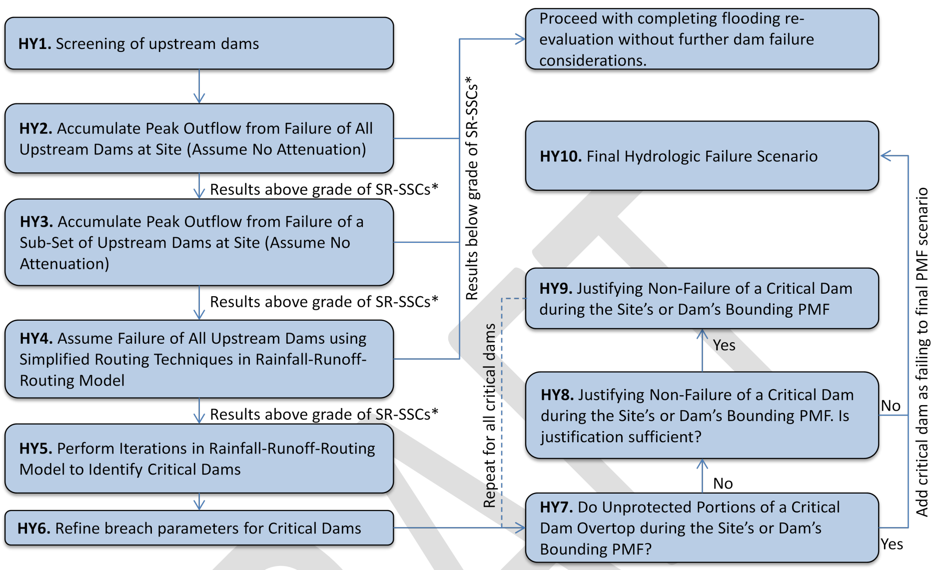
The HHA process allows licensees the option to conduct simplified flooding evaluations, based on varying degrees of conservativeness, to assess susceptibility to flooding. The evaluation is refined using site-specific parameters to achieve a realistic, physics based, but conservative analysis of flooding, particularly when resulting hazard levels exceed acceptance criteria for safety-related SSCs. NUREG/CR-7046 describes the key steps in the process as follows:

1. Identify flood-causing phenomena or mechanisms by reviewing historical data and assessing the geohydrological, geoseismic, and structural failure phenomena in the vicinity of the site and region.
2. For each flood-causing phenomenon, develop a conservative estimate of the flood from the corresponding probable maximum event using conservative simplifying assumptions.
3. If any safety-related SSC is adversely affected by flood hazards, use site-specific data to provide more realistic conditions in the flood analyses. Repeat Step 2; if all safety-related SSCs are unaffected by the estimated flood, or if all site-specific data have been used, specify design bases for each using the most severe hazards from the set of floods corresponding to the flood-causing phenomena.

According to Section 3.4.1 of NUREG/CR-7046, 'the simplest and most conservative dam-breach induced flood may be expected to occur under the assumption that (1) all dams upstream of the site are assumed to fail during the PMF event regardless of their design capacity to safely pass a PMF and (2) the peak discharge from individual dam failures reach the site at the same time.' This general approach was applied to all three failure modes (hydrologic, seismic, and sunny-day).

4.4 Hydrologic-Induced Failure

Figure 3 - Approach to Hydrologic Failure Evaluation



* SR-SSCs: Safety-Related Systems, Structures, and Components.

Figure 3 and the discussion below describe the approach to conducting an evaluation of upstream dam failures induced by a hydrologic (precipitation/snowmelt) event. The licensee and its vendor have the option to bypass selected steps in the HHA approach or go directly to Step HY10 and assume all potentially-critical dams fail).

Step HY1 – Screening of Upstream Dams

Refer to Section 3.1.

Step HY2 – Accumulate Peak Outflow from Failure of All Upstream Dams at Site (Assume No Attenuation)

Assume all potentially critical ~~(unscreened)~~ dams fail during the PMF and all reach the site coincidental to the peak. Add wind-waves from 2-year wind speed. Use applicable regression equation(s), or other appropriate methods, to calculate peak outflow. Assume pool levels are at the top of dam. If results are below grade of safety-related SSCs, proceed with completing flooding reevaluation without further dam failure considerations. If results exceed grade of safety-related SSCs, proceed to next step.

Step HY3 – Accumulate Peak Outflow from Failure of a Sub-Set of Upstream Dams at Site (Assume No Attenuation)

FFTF - Subset is intended to only apply to Step HY3. HY4 was updated in the chart and write-up to clarify.
Clarify which dams are to be considered in each step
FFTF – OK, see above
Adjust wording to clarify the purpose of this process
FFTF – OK, see above
Clarify potential uses of the outputs of this section
FFTF – Output is used to determine if you can proceed without further consideration of dam failure. Clarification will be added within the individual steps.
More guidance on how to refine parameters
FFTF – added reference to section 4.7 under step HY6.

Comment [initials22]: FFTF Change

Assume all potentially critical ~~(unscreened)~~ dams fail during the PMF but only a sub-set reach the site at the same time at the PMF peak. Add wind-waves from 2-year wind speed. Use applicable regression equation(s), or other appropriate methods, to calculate peak outflow. Assume pool levels are at the top of dam. If results are below grade of safety-related SSCs, proceed with completing flooding reevaluation without further dam failure considerations. If results exceed grade of safety-related SSCs, proceed to next step.

Comment [initials23]: FFTF change

Step HY4 – Assume Failure of All Dams using Simplified Routing Techniques in Rainfall-Runoff-Routing Model

Fail all potentially critical ~~(unscreened)~~ dams in rainfall-runoff-routing model (e.g. HEC-HMS) during the PMF, with the trigger being the peak water level, and route hydrographs to site using simplified techniques in model. Add wind-waves from 2-year wind speed. Use conservative breach parameters. Dam failure scenarios should include combinations of individual and/or cascading failures per Section 3.2 and Appendix D (Section D.1) of NUREG/CR-7046. If results are below grade of safety-related SSCs, proceed with completing flooding reevaluation without further dam failure considerations. If results exceed grade of safety-related SSCs, proceed to next step.

Comment [initials24]: FFTF Change

Step HY5 – Perform Iterations in Rainfall-Runoff-Routing Model to Identify Critical Dams

Perform iterations in rainfall-runoff-routing model (e.g. HEC-HMS) to identify critical dam(s) whose failures have a significant impact at site; assume all non-critical ~~(unscreened)~~ dams fail. Proceed to next step.

Comment [initials25]: FFTF Change

Step HY6 – Refine Breach Parameters for Critical Dam

Refine breach parameters for each critical dam (see section 4.7). Breach parameters should be specific to the type of dam (earthen, rock fill, concrete/arch, etc.) and type of failure (overtopping or piping) using realistic but conservative physics -based assumptions.

THE SUBSEQUENT STEPS ARE REPEATED FOR EACH CRITICAL DAM. The objective is to provide the licensee with the option to, with proper justification, credit a particular critical dam as not failing in the final hydrologic failure scenario.

Step HY7 – Do Unprotected Portions of a Critical Dam Overtop during the Site's or Dam's Bounding PMF?

Section 5.5.1 of ANS 2.8, under 'Hydrologic Dam Failures', states that "*critical dams should be subjected analytically to the probable maximum flood from their contributing watershed. If a dam can sustain this flood, no further hydrologic analysis shall be required.*" Therefore, answering this question requires the establishment of two hydrologic scenarios: 1) the bounding PMF scenario for the entire watershed at the site and 2) the bounding PMF for the specific watershed of the critical dam in question. In lieu of developing a dam-specific bounding PMP, documentation from the dam owner can be used to demonstrate that a critical dam can safely pass the dam's bounding PMF; as long as the documentation was developed or approved by a state or federal government agency using criteria/methodologies developed or bounded by USBR, USACE, or FERC. In situations where a critical dam does not overtop during the site's bounding PMF

Comment [g26]: Clarify this discussion on how to use
FFTF – this clarification will depend upon how the ICODS meetings proceed and under what circumstances we can take credit for the work of other entities. Will incorporate a final change later.

but does overtop during the dam's PMF, the licensee has the option to develop an alternative hydrologic scenario for the site that includes the bounding PMP for an individual, critical dam and failure of this dam. It is unreasonable to assume that multiple, individual, critical dams would be subjected to dam-specific bounding PMFs simultaneously. Cascading failures of dams in series should be considered in this alternative hydrologic scenario per Section 3.2 and Appendix D (Section D.1) of NUREG/CR-7046.

Per ANS 2.8, Section 5.5.4, *"if no overtopping is demonstrated, the evaluation may be terminated and the embankment may be declared safe from hydrologic failure"*. Overtopping may be investigated for these two conditions:

- Probable maximum flood surcharge level plus maximum (1%) average height resulting from sustained 2-year wind speed applied in the critical direction; or
- Normal operating level plus maximum (1%) wave height based on the probable maximum gradient wind.

For the purpose of this paper, 'overtopping' is defined as the point at which an unprotected portion of the dam, or portion of the dam structure not designed to convey floodwater, is subject to flow during a postulated flood. Even without overtopping, additional information, discussed in the next step, may be required to demonstrate safety under PMF loading conditions.

Comment [g27]: How does this affect spillways
FFTF – not sure of the meaning of this comment. Spillways are designed to convey floodwater and would seem to be addressed by the text as written.

Step HY8 – Justifying Non-Failure of a Critical Dam during the Site's or Dam's Bounding PMF

For critical dams, where non-failure justification is sought, develop information in Section 5.5.4 of ANS 2.8, demonstrating safety from failure due to instability, erosion, sliding, or overturning during site's or dam's bounding PMF. ~~Stability~~ A valid stability analyses of dams that meets the standards established by the dam's regulator should be used requiring ~~requires~~ documentation of structural dimensions and composition from design plans; construction records; records from installed instrumentation; field surveys, on-site inspections; and special strength testing, coring, and instrumentation. Information from the dam owner, developed or approved by a state or federal agency, can be used to justify non-failure. In situations where a critical dam does not overtop during the site's bounding PMF but does overtop during the dam's bounding PMF, the licensee has the option to develop an alternative hydrologic scenario for the site that includes the bounding PMP for an individual, critical dam and failure of this dam. If justification is sufficient, go the next step. If not, this dam should be included as failing in the final hydrologic failure scenario.

Comment [g28]: Ensure this is using current methods
FFTF – Should be able to use existing analyses as long as they meet the standards of the dam's regulator

Step HY9 – Credit Critical Dam as Not Failing in the Final Hydrologic Failure Scenario

The critical dam can be credited as not failing during the site's bounding PMF in the final hydrologic dam failure scenario. Repeat HY7 through HY9 for the next critical dam.

Step HY10 – Final Hydrologic Failure Scenario

The final hydrologic failure scenario includes:

- Site's bounding PMF;
- Failure of non-critical dams;
- Failure of critical dams with insufficient non-failure justification;

- Wind-waves from 2-year wind speed; and
- Enhanced modeling techniques (e.g. 1D unsteady flow and/or 2D/3D hydrodynamic models) to refine flood level at site (optional).

Trigger failures in the site's bounding PMF model at the peak water surface elevation for individual failures. For dams in series, failure should be triggered to maximize the affect of compounding flows from cascading failures. See Section 3.2.

Additional Considerations (from ANS 2.8, Section 5.5.4.2)

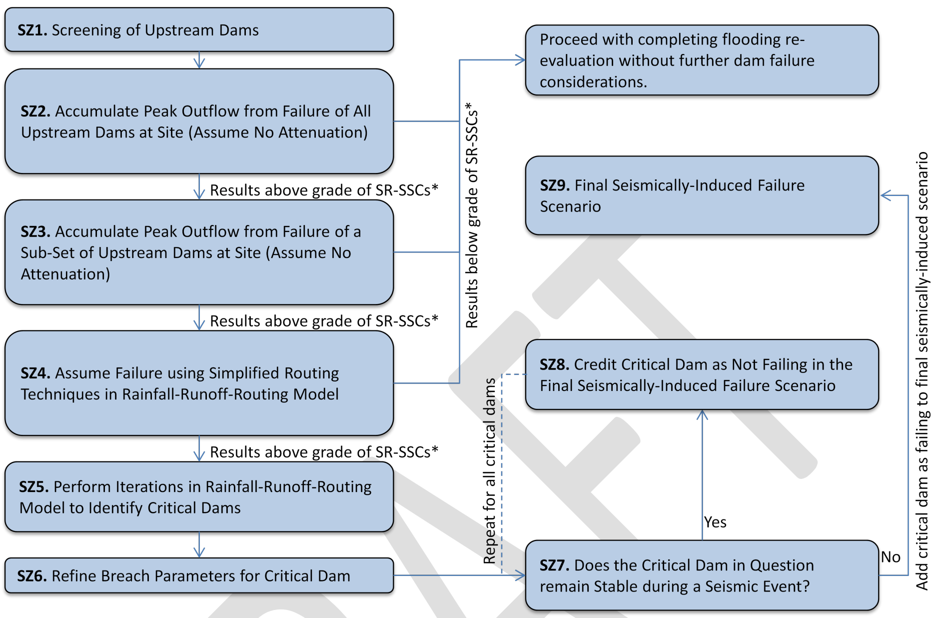
- **Concrete Sections:** Concrete gravity dams should be analyzed against overturning and sliding. With some blocks judged likely to fail and others not, the mode and degree of probable failure can be judged as well as the likely position and amount of downstream debris. From this analysis, the water path and the likely elevation-discharge relationship applying to the failed section can be estimated with reasonable accuracy. Rise of tailwater should be considered in the stability analysis.
- **Arch Dams:** Arch dams can usually sustain considerable overtopping with failure most likely from foundation and abutment failure. However, unless structural safety can be documented, failure should be postulated. Failure of an arch dam might approach instantaneous disappearance with minimum residual downstream debris.
- **Earth and Rockfill:** Earth and rock embankments ~~shall~~ should be evaluated for breaching from overtopping unless justification can be provided to demonstrate that sufficient free board capacity, spillway capacity, operating and maintenance procedures exist that will assure successful passing of the PMP or an upstream dam failure. If there are two or more independent embankments, it may be necessary to fail only one if it produces the most critical flood wave.

Comment [initials29]: FFTF change.

Comment [g30]: Consider moving earlier in the section
FFTF – OK, where?

4.5 Seismically-Induced Failure

Figure 4 - Approach to Seismically-Induced Failure Evaluation



* SR-SSCs: Safety-Related Systems, Structures, and Components.

Figure 4 and the discussion below describe the approach to conducting an evaluation of upstream dam failures induced by a seismic event. The licensee and its vendor have the option to bypass selected steps in the HHA approach or go directly to Step SZ9 and assume all potentially-critical dams fail.

<u>Step SZ1 – Screening of Upstream Dams</u>

Refer to Section 3.1.

<u>Step SZ2 – Accumulate Peak Outflow from Failure of All Upstream Dams at Site (Assume No Attenuation)</u>

Assume all potentially critical ~~(unscreened)~~ dams fail during the ½ PMP or 500-year precipitation (whichever is less) and <u>all</u> reach the site coincidental to the peak. Add wind-waves from 2-year wind speed. Use applicable regression equation(s), or other appropriate methods, to calculate peak outflow. Assume pool levels are at the top of dam. If results are below grade of safety-related SSCs, proceed with completing flooding reevaluation without further dam failure considerations. If results exceed grade of safety-related SSCs, proceed to next step.

<u>Step SZ3 – Accumulate Peak Outflow from Failure of a Sub-Set of Upstream Dams at Site (Assume No Attenuation)</u>

Assume all potentially critical ~~(unscreened)~~ dams fail during the ½ PMP or 500-year precipitation (whichever is less) but only a sub-set reach the site coincidental to the peak; add wind-waves from 2-year wind speed. Use applicable regression equation(s), or other appropriate methods, to calculate peak outflow. Assume pool levels are at the top of dam. If results are below grade of safety-related SSCs, proceed with completing flooding reevaluation without further dam failure considerations. If results exceed grade of safety-related SSCs, proceed to next step.

Step SZ4 – Assume Failure using Simplified Routing Techniques in Rainfall-Runoff-Routing Model

Fail all potentially critical ~~(unscreened)~~ dams in rainfall-runoff model (e.g. HEC-HMS) during the ½ PMP or 500-year precipitation (whichever is less), with the trigger being the critical time of the earthquake, and route hydrographs to site using simplified techniques in model. Add wind-waves from 2-year wind speed. Use conservative breach parameters and assume pool levels are at the top of dam. Dam failure scenarios should include combinations of individual and/or cascading failures per Section 3.2 and Appendix D (Section D.1) of NUREG/CR-7046. If results are below grade of safety-related SSCs, proceed with completing flooding reevaluation without further dam failure considerations. If results exceed grade of safety-related SSCs, proceed to next step.

Step SZ5 – Perform Iterations in Rainfall-Runoff-Routing Model to Identify Critical Dams

Perform iterations in rainfall-runoff-routing model (e.g. HEC-HMS) to identify critical dam(s) whose failures have a significant impact at site; assume all non-critical ~~(unscreened)~~ dams fail. Proceed to next step.

Step SZ6 – Refine Breach Parameters for Critical Dam

Refine breach parameters for each critical dam. Breach parameters should be specific to the type of dam (earthen, rock fill, concrete/arch, etc.) and type of failure (overtopping or piping) using realistic but conservative physics -based assumptions.

THE SUBSEQUENT STEPS ARE REPEATED FOR EACH CRITICAL DAM. The objective is to provide the licensee with the option to, with proper justification, credit a particular critical dam as not failing in the final seismically-induced failure scenario.

Step SZ7 – Does the Critical Dam in Question remain Stable during a Seismic Event?

Information should be developed to assess a dam's ability to withstand a design earthquake. Regulation 10 CFR 100.23 (d)(3) states "the size of seismically induced floods and water waves that could affect a site from either locally or distantly generated seismic activity must be determined". Based on existing guidance in RG 1.59 and ANS 2.8, the earthquake centering shall be evaluated in a location(s) that produce the worst flooding from a seismically induced dam failure at the nuclear power plant site. In regions where two or more dams are located close together, a single seismic event shall be evaluated to determine if multiple dam failures could occur.

The evaluation of the dam's structural stability shall include the concrete and earth sections. The methods for evaluation should be those described by ~~USACE, Bureau of Reclamation (USBR), or Federal Energy~~

~~Regulatory Commission (FERC)~~the dam's regulator. The existing evaluations completed by the dam owner may be used if the review determines that the current standards as prescribed by ~~USACE, USBR, or FERC~~dam's regulator are used and the required factors of safety per those standards are satisfied. In addition, the combined annual exceedance probability for design earthquake loading, seismic failure, and the hydrologic event, shall be 1 x 10^{-6} or less.

Comment [g32]: This assumption may require additional interaction.
FFTF – Discuss at meeting

This includes hazard and fragility

Design Earthquake Loading:

- Ground Motion Hazard Curves – The Recommendation 2.1 Seismic Hazard Reevaluations are ongoing and will be based, in part, on the Central and Eastern United States (CEUS) Source Characterization and new attenuation model; expected to be completed in February 2013. The Recommendation 2.1 Flood Hazard Reevaluations at some sites are scheduled for completion before the CEUS source characterization is available. Therefore, licensees with Flood Hazard Reevaluation Reports due by March 2013 are provided with two options for developing the ground motion hazard curves.

    1. Use USGS (2008) to determine the mean seismic hazard curves for 1 Hz, 5 Hz, 10 Hz, and PGA. Apply one of five EPRI mean amplification functions to the mean rock seismic hazard curves based on the known geologic conditions at the site. EPRI mean amplification functions can be found in EPRI (1993).

    Comment [initials33]: **FFTF – If this stands until the next update is required, it is viable.**

    2. Submit the Flood Hazard Reevaluation Reports assuming all critical (and non-critical) dams fail during a seismic event, combined with the lesser of the ½ PMP and 500-year precipitation (Step SZ4). ~~The seismic hazard at the critical dams can be re-assessed, once the CEUS source characterization data and attenuation model are available, as a supplemental seismic submittal to the Flood Hazard Reevaluation Report.~~

    Comment [initials34]: **FFTF – see comment above concerning combination of events**

    ~~2.~~3. Use the CEUS seismic source term and associated attenuation model. If this results in not being able to submit the reevaluation in accordance with the committed schedule, submit all elements of the flooding reevaluation that are completed on the scheduled date. Establish a new completion date at the time of this submittal for completion of the upstream dam failure and overall conclusions.

    Comment [initials35]: **FFTF – one of the options we have been discussing**

- From the site-adjusted mean hazard curves, develop the 10^{-4} Uniform Hazard Response Spectrum (UHRS) and hazard curves for 1 Hz, 5 Hz, 10 Hz, and PGA. ~~(Note: the simplified analysis described below may only require hazard curves for PGA).~~

Comment [g36]: Reconcile the 10E-4 here with 10E-6 mentioned previously

The probability of seismic failure of a dam can be estimated using simplified procedures as described in the following steps:

- Develop failure criteria for each seismic failure mode. The criteria should be based on dam type (concrete sections, arch dams, earthfill, and rockfill, ~~etc~~), construction details (slope protection, filters and drains, core width, past performance, etc), and overall construction quality. Examples of failure criteria could be maximum crest settlement, factor of safety against sliding, ~~-~~and fault offset at the foundation elevation. It is noted that not all potential seismic failure modes will need to be addressed at each site. For instance, potential failure due to surface fault rupture can be screened out for sites where no known faulting is present.

- If existing evaluations have been completed by the dam owner using current standards prescribed by the dam's regulator, summarize analyses results including ground motion parameters used, factors of safety for each failure mode, performance results (i.e., settlement or crest deformation).

- If the existing analyses include High Consequence of a Low Probability of Failure (HCLPF) results, and the results are enveloped by the ground motions from Options 1 or 3 above, the dam can be considered to have a probability of failure of less than one percent. If the HCLPF capacity is greater than the ground motions in Options 1 or 3 above, use the results of the HCLPF analyses to estimate the probability of failure for the ground motions in Option 1 or 3.
- If the existing analyses are deterministic and do not include fragility evaluations, the deterministic evaluations should be updated to estimate the median ground parameter (A_m) for each failure criteria.~~For each potential seismic failure mode (fault offset, permanent seismic deformation including potential for liquefaction), review available reports and estimate current seismic capacity. Ideally, the existing analyses could be scaled or modified to estimate the ground motion level at which the dam fails according to each of the failure modes and failure criteria. This could be done by revising the existing analyses, and increasing the seismic load until the failure criteria is reached.~~
- Use the median ground motion parameter corresponding to failure and an assumed uncertainty values (β_R and β_U) ~~(σ_{ln})~~ to develop lognormal fragility curves for each failure mode.
- Estimate the probability of failure at the ground motion level from Option 1 or 3 .~~for a full range of ground motion values for each failure mode and sum the probabilities to estimate the aggregate probability of seismically induced failure.~~

Data needed for the seismic evaluation include:

- Design or as-built drawings;
- Existing seismic stability evaluation reports containing:
  - Description of dam materials (zones, filters, surface protection);
  - Description of geologic setting;
  - Description of foundation conditions;
  - Description of cut-off trenches or foundation grouting; and
  - Description of previous analyses (ground motion inputs, methods, results).
- Instrumentation Data;
- Summary of past performance;
- Shear and compression wave velocity data within foundation; and
- Description of spillway and low-level outlet facilities.

If justification is sufficient, go the next step. If not, this dam should be included as failing in the final seismically-induced failure scenario.

Step SZ8 – Credit Critical Dam as Not Failing in the Final Seismically-Induced Failure Scenario

The critical dam can be credited as not failing in the final seismically-induced dam failure scenario. Repeat SZ7 for the next critical dam.

Step SZ9 – Final Seismically-Induced Failure Scenario

The final seismically-induced failure scenario includes:

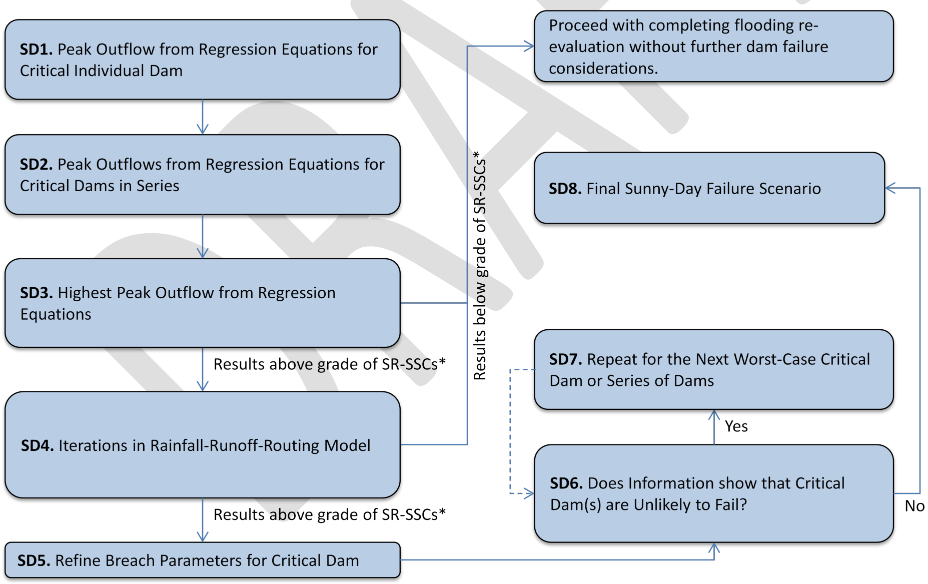
- ½ PMP or 500-year precipitation (whichever is less);
- Failure of non-critical dams;
- Failure of critical dams with insufficient non-failure justification;
- Wind-waves from 2-year wind speed; and
- Enhanced modeling techniques (e.g. 1D unsteady flow and/or 2D/3D hydrodynamic models) to refine flood level at site (optional).

Comment [initials38]: FFTF – need to define event combinations

Trigger individual failures in the final model at the same time, determined by optimizing the effects of the earthquake. For dams in series, failure should be triggered to maximize the affect of compounding flows from cascading failures. See Section 3.2.

4.6 Sunny-Day Failure

Figure 5 - Approach to Sunny-Day Failure Evaluation



* SR-SSCs: Safety-Related Systems, Structures, and Components.

A sunny-day failure is a failure that is not induced by a precipitation event. (For the purposes of this paper, a seismically-induced failure is being considered separately.) Sunny-day failures are typically attributed to structural weakness or deficiency in the dam embankment, foundation, and/or abutments. Potential causes of failure (from Section 6.3.2 of ANS 2.8) include:

- Deterioration of concrete due to cracking, weathering, or chemical growth;
- Deterioration of embankment protection such as riprap or grass cover;
- Excessive saturation of downstream face or toe of embankment;
- Excessive embankment settlement;
- Cracking of embankment due to uneven settlement;
- Erosion or cavitation in waterways and channels, including spillways;
- Excessive pore pressure in structure, foundation, or abutment;
- Failure of spillway gates to operate during flood because of mechanical or electrical breakdown or clogging with debris;
- Buildup of silt load against dam;
- Excessive leakage through foundation;
- Leakage along conduit in embankment;
- Channels from tree roots or burrowing;
- Excessive reservoir rim leakage; and/or
- Landslide in reservoir.

While generally expected not to produce flood discharges and water levels that exceed the hydrologic or seismically-induced failure scenarios, discussed above, it can be associated with the shortest warning times. Some licensees may consider applying sunny-day failure warning times to the seismically-induced failure scenarios; in which case, sunny-day failure may not need to be a consideration at the site. The following describes the steps in a sunny-day failure evaluation:

<u>Step SD1 – Peak Outflow from Regression Equations for Critical Individual Dam</u>

Use applicable regression equation(s) and/or other appropriate methods to calculate the peak outflow at individual upstream dams, largest and closest to the site. Iterations may be required to identify the critical individual dam. Assume pool levels are at the top of dam.

<u>Step SD2 – Peak Outflows from Regression Equations for Critical Dams in Series</u>

Use applicable regression equation(s) and/or other appropriate methods to calculate and add the peak outflows for upstream dams in series (if relevant), largest and closest to the site. Iterations may be required to identify the critical series of dams. Assume pool levels are at the top of dam.

<u>Step SD3 – Highest Peak Outflow from Regression Equations</u>

Use the highest peak outflow from individual failure (SD1) or highest cascading peak outflow from dams in series (SD2), whichever is greater, and transpose directly to site (no attenuation). Add wind-waves from 2-year wind speed. If results are below grade of safety-related SSCs, proceed with completing flooding reevaluation without further dam failure considerations. If results exceed grade of safety-related SSCs, proceed to next step.

Step SD4 – Iterations in Rainfall-Runoff-Routing Model

Perform iterations in rainfall-runoff model (e.g. HEC-HMS) to identify critical dam(s) whose individual or cascading sunny-day failures have an effect at the site. (Other than cascading failures for dams in series, simultaneous individual failures are not being considered.) Add wind-waves from 2-year wind speed. Use conservative breach parameters. If results are below grade of safety-related SSCs, proceed with completing flooding reevaluation without further dam failure considerations. If results exceed grade of safety-related SSCs, proceed to next step.

Step SD5 – Refine Breach Parameters for Critical Dam

Refine breach parameters for the critical dam. Breach parameters should be specific to the type of dam (earthen, rock fill, concrete/arch, etc.) and type of failure (overtopping or piping) using realistic but conservative physics -based assumptions.

THE SUBSEQUENT STEPS ARE REPEATED TO IDENITFY THE WORST-CASE CRITICAL DAM. The objective is to identify the worst-case critical dam or provide the licensee with the option to, with proper justification, credit all critical dams as not failing in the sunny-day failure scenario.

Step SD6 – Does Information show that Critical Dam(s) are Unlikely to Fail?

Develop information, discussed below, to appraise likelihood of failure for the worst-case critical dam or series of dams. Information from the dam owner, developed or approved by a state or federal agency, can be used to justify non-failure. If justification adequately shows that the worst-case critical dam is unlikely to fail, proceed to Step SD7. If not, this represents the worst-case critical dam or series of dams for the sunny-day failure scenario (Step SD8).

For each critical dam the licensee intends to credit as unlikely to experience sunny-day failure, the information below may be required to demonstrate safety under 'sunny-day' conditions:

Comment [g39]: Additional interactions will be necessary on this point.

- Structural dimensions;
- Construction records;
- Records from installed monitoring instrumentation and/or piezometer wells;
- Field surveys
- On-site inspection reports;
- Maintenance records;
- Risk tolerance of operating agency; and

- Durable operation, inspection, monitoring, maintenance, and corrective action procedures and agreement.

Information from the dam owner, developed or approved by a state or federal agency, can be used to demonstrate that sunny-day failure is unlikely. If non-failure justification is adequate (such as concrete dam with rock abutments to eliminate the possibility of a piping failure), the next worst-case critical sunny-day dam failure (if applicable) should be evaluated.

Comment [initials40]: FFTF change

Step SD7 – Repeat for the Next Worst-Case Critical Dam or Series of Dams

Sunny-day dam failure does not need to be considered for this dam or series of dams. Repeat for next worse-case critical dam or series of dams (cascading failures) until all critical dams or series of dams have been considered.

Step SD8 – Final Sunny-Day Failure Scenario

The final sunny-day failure scenario includes:

- Failure of worse-case critical dam or series of dams (cascading failures);
- No precipitation;
- Wind-waves from 2-year wind speed ; and
- Enhanced modeling techniques (e.g. 1D unsteady flow and/or 2D/3D hydrodynamic models) to refine flood level at site (optional)

Assume failure occurs at ~~[normal high-water~~full normal pool level] ~~[water level at the top-of-dam]~~. Given the nature of a sunny-day failure, it would be unreasonable to assume simultaneous individual failures.

Comment [g41]: Normal high would be a reasonable assumption

4.7 Breach Parameters and Development

4.7.1 Empirically-Based (Regression) Peak Outflow Estimation

These methods include relatively simple regression equations to estimate the peak outflow and attenuation resulting from a dam failure. Wahl (1998) identified regression equations that estimate the peak outflow as a function of dam and/or reservoir properties based on real dam failure data. Five peak outflow discharge estimation methods are listed below and presented in more detail in Appendix D. Note, original technical papers or documentation should be reviewed prior to using these equations to understand their limitations. Wahl (2004) indicates that the Froehlich (1995a) method has the lowest uncertainty of the dam breach peak discharges equations available at the time. Furthermore, Pierce (2010) indicated that the USBR (1982) and Froehlich (1995a) equations 'remain valid for conservative peak-outflow predictions' for embankment dams. As part of the HHA process, attenuation of the peak discharge can be ignored to conservatively account for the effect of the breach at the site. However, the USBR (1982) provides a simplified, conservative method for estimating the peak flow reduction as a function of distance to the site (miles). (See Figure 2 in USBR (1982).)

- USBR (1982) Peak Outflow
- Froehlich (1995a) Peak Outflow

- National Weather Service (NWS) Simplified Dam Break Model (Whetmore, 1991; Reed, 2011)
- Natural Resources Conservation Service (NRCS) Technical Release (TR) 60 (2005) (formerly the Soil Conservation Service (SCS))
- Walder and O'Connor 1997

Provide summary for Pierce/Thornton (2010)

Comment [initials42]: FFTF - later

4.7.2 Empirically-Based (Regression) Breach Parameter Estimation

The Bureau of Reclamation (Wahl, 1998) provides a relatively comprehensive review of methods for predicting breach parameters. Since estimates of breach parameters vary significantly, Wahl suggested using several methods to establish a range of breach parameters, giving due consideration to the dam's design characteristics. Other notable and more recent reviews of breach parameter prediction methods (and peak-flow prediction equations and related dam-failure modeling guidance) include Washington State (2007) and Colorado Department of Natural Resources (2010).

The USACE (Gee, 2008) provided a review of three (3) regression models for breach parameter development:

- Froehlich (1995b) (updated in 2008) – Based on 63 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rockfill dams to establish methods to estimate average breach width, side slopes, and failure time.
- MacDonald and Langridge-Monopolis (MacDonald, 1984) – Based on 42 predominately earthfill, earthfill with a clay core, and rockfill dams to establish a 'Breach Formulation Factor' (product of the volume of water released from the dam and the height of the water above the dam).
- Von Thun and Gillette (1990) – Based on 57 dams from both Froehich (1987) and MacDonald and Langridge-Monopolis (1984) papers to estimate side slopes and breach development time.

Gee (2008) indicated that the above parameter estimation methods were applied to five (5) breach situations for comparison and provided the results of these comparisons to two (2) of the five (5) in the 2008 paper. The comparison for the Oros Dam, which failed by an overtopping event in March 1960 in Brazil, is provided in Figure 5. Gee (2008) concluded that "the methods predict a wide range of breach parameters and therefore, a large difference in outflow hydrographs. The MacDonald method routinely produced the largest peak outflows". Gee (2008) also discusses physically-based breach formulation models that use sediment transport functions; this is addressed in the next section. As noted previously, original technical papers or documentation should be reviewed prior to using these equations to understand their limitations. Justification should be developed for the selected method(s). More than one method should be used provide higher confidence in the results.

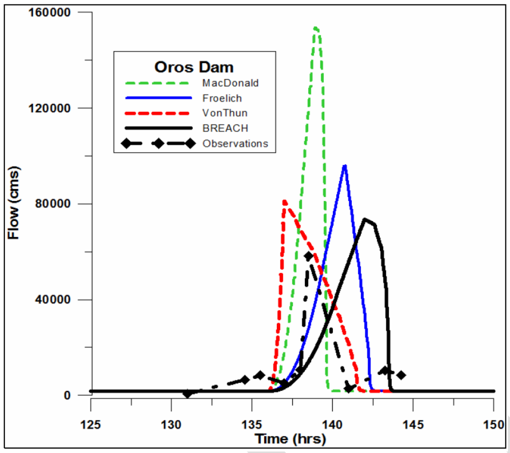


Figure 6 - Breach Hydrographs for Oros Dam (Gee, 2008)

Following a recommendation by Wahl (2008), Xu and Zhang (2009) developed new equations to estimate breach parameters for earth and rockfill dams. The new equations are based on an analysis that includes case study data from China and more recent failures not previously analyzed by the earlier investigators. From a database of 182 failures, they were able to utilize 75 for development of the new equations. A key difference from earlier works was the incorporation of soil erodibility into the method, which proved to be the most influential of all those examined. Xu and Zhang subdivided breaching parameters into two groups, geometric and hydrographic, and included:

- Geometric
  - Breach Depth (H_b)
  - Breach Top Width (B_t)
  - Average Breach Width (B_{ave})
  - Breach Bottom Width (B_b)
  - Breach Side Slope Factor (Z)
- Hydrographic
  - Peak Outflow (Q_p) (see also previous section)
  - Failure Time (T_f)

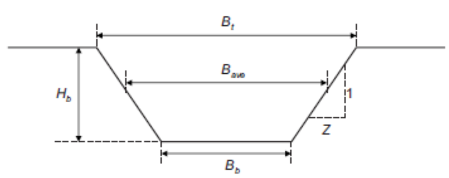


Figure 7 - Geometric Parameters of an Idealized Dam Breach (Xu and Zhang, 2009)

It is important to note the failure times predicted by Xu and Zhang are longer than those predicted by most other methods. Many other well-known case studies show longer failure times than those reported by other investigators. For the Teton Dam failure, as an example, Xu and Zhang (2009) used a 4-hour failure time in developing their regression equation, whereas Froehlich (1995a and 2008) used a 1.25-hour failure time.

Additional Consideration for Concrete Dams

In general, the current approach to concrete dams is instantaneous failure. The analysis does not necessarily need to include failure of the entire dam. For example, for a dam with large gates on the top, it may be reasonable to analyze a failure mode where only the gates fail, but that the concrete portion of the dam beneath and adjacent to the gates remains intact. For dams with distinct structural segments (e.g., buttress dams), limiting failure to one or more segments that are most prone to a deficiency may be justifiable.

4.7.3 Physically-Based Methods

In 2004, the Centre for Energy Advancement through Technological Innovation (CEATI) formed a Dam Erosion and Breaching working group within their Dam Safety Interest Group (DSIG). The objective of this group was to collaborate on the development of improved methods for simulating embankment erosion and breach processes. The focus of the group's work was on physically-based computer models. The DSIG group comprised members from the USACE, USBR, USDA-ARS, Hydro Quebec, BC Hydro, HR Wallingford, Elforsk (Scandinavian Utility), and EDF (French Electrical Utility). Tasks undertaken by the CEATI-DSIG group included:

- International review of currently available breach models and new models under development;
- Selection of 3 most promising models for closer evaluation;
- Assembly of high quality case study data and large-scale laboratory test data (for model validation purposes);
- Evaluation of model performance against seven selected data sets;
    - Two large-scale lab tests conducted by USDA-ARS (Stillwater, OK);
    - Three large-scale tests conducted in Norway in connection with the European IMPACT project; and
    - Two actual dam failures (Oros, Banqiao).

The DSIG Project concluded (Wahl 2009; Morris et al. 2012) that the HR-BREACH and SIMBA/WinDAM models were both very capable, with many similarities and some differences. The two models continue to be separately under development at this time. The most significant differences between the models at this time are:

- HR-BREACH analyzes overtopping and piping failure modes and allows definition of zoned embankment geometry;
- WinDAM analyzes only homogeneous embankments and the overtopping failure mode;
- HR-BREACH includes an energy-based headcut erosion model and several alternative surficial erosion models (sediment transport equations);
- WinDAM offers both stress-based and energy-based headcut erosion models, but no surficial erosion of the body of the embankment;
- HR-BREACH is not publicly available, but is available via consultation with its developer, HR Wallingford, and is also being incorporated into commercial dam-break flood routing models used in Europe.
- WinDAM is publicly available from the USDA-NRCS. The technology contained in WinDAM was first developed in the SIMBA model (a research tool never made available to the public) by the Agricultural Research Service (USDA-ARS). WinDAM also contains earthen spillway headcut erosion analysis capabilities that are similar to the SITES model, also distributed by USDA-NRCS. A version

of WinDAM that will analyze the piping failure mode is under development and may be available in late 2013.

- Both models allow simulation of the erosion and failure of grass or riprap armoring on the exterior of an embankment, although they use different algorithms.

CEATI-DSIG project (Morris et al. 2012)~~Table 4~~ provides a subjective comparison of the properties and capabilities of the models evaluated in the ~~CEATI-DSIG project (Morris et al. 2012)~~.

~~**Table 4 - Comparison of Physically-Based/Erosion Process Models**~~

	HR-BREACH	SIMBA / WinDAM	FIREBIRD	NWS-BREACH
Erosion Process Models	Good	Good	**Fair**	**Limited**
Surface protection	Vegetation (CIRIA) and riprap	Vegetation, riprap in WinDAM	**Limited**	Yes
Headcut erosion	Good	Best	**No**	**No**
Stress-based	—	Yes	—	—
Energy-based	Yes	Yes (in WinDAM)	—	—
Surface erosion	Yes	**No**	Yes	Yes
Mass-wasting / soil-wasting	Stress-based bank failures and arch failure	Bank failures implicit	**Some**	**Some**
Effects of Submergence	Yes	Yes (in WinDAM)	**No**	Yes
Piping progression	Yes	In development	**Some**	Yes
Data Input Guidance	Good	Good	**Limited**	**Limited**
Ease of Use	Good	Good	**Difficult**	**Difficult**
Computational Efficiency	Good	Good	**Fair**	Good
Documentation	Excellent	Excellent	**Limited**	Good
Organizational Support for Continued Development	Good	Good	**Weak**	**None**
Embankment Geometry Options	Simple Zoning	Homogeneous, (Zoned in future)	Simple Zoning	Primitive Zoning

The use of a physically-based breach model requires significantly greater effort by the analyst, since dam and reservoir details must be specified, alternatives for erosion calculations must be selected, and soil erodibility properties must be estimated or measured. Sensitivity analyses must also be carried out to investigate the effects of variation of input parameters. The use of physically-based models may be justified when more accurate results are needed and soil erodibility can be reasonably estimated.

Expand this section to possibly discuss:

- Other physically-based models that have appeared in recent years: Macchione (2008) in Journal of Hydraulic Engineering (2 papers), and Wang and Bowles (2005; 3 papers in Advances in Water Resources --- 2007, one paper in Water Resources Research), Weiming Wu at USDA-ARS (Oxford, Mississippi)
- Jimmy O'Brien integrating the NWS-BREACH methodology into the commercially available FLO-2D model and correcting problems with NWS-BREACH.

4.7.4 Uncertainty

In general, uncertainty in formulating a dam failure should be evaluated by applying multiple methods, applicable to the dam in question, and evaluating sensitivity to reasonable variations in input parameters. Xu and Zhang (2009) developed a comparison in empirical prediction equations using the case studies in

Comment [initials43]: FFTF – removed reference to software preferences their research, which will produce bias towards the Xu and Zhang results. (See Table 5.) Nevertheless, the Xu and Zhang method appears to offer the least variability and seems to accommodate a wider range of situations.

Add contributions from Wahl (2004) and Froehlich (2008)

Comment [initials44]: FFTF - later

Table 1 - Comparison of Different Parameter Prediction Equations based on Case Studies in Xu and Zhang (2009)

	Best prediction models in this paper			Best-simplified prediction models in this paper			Bureau of Reclamation (1982, 1988)			Froehlich (1995a,b)		
Parameter	Number of cases[a]	Mean	SD	Number of cases[a]	Mean	SD	Number of cases[a]	Mean	SD	Number of cases[a]	Mean	SD
H_b	62	1.01	0.08	66	1.01	0.08	—	—	—	—	—	—
B_t	52	1.03	0.32	59	1.03	0.32	—	—	—	—	—	—
B_{ave}	43	1.02	0.32	52	1.03	0.33	51	1.31	0.94	51	1.07	0.43
Q_p	33	1.01	0.34	38	1.03	0.39	36	0.95	0.80	36	1.30	0.84
T_f	27	1.02	0.43	28	1.02	0.44	27	3.08	4.00	27	2.30	2.05

[a]The number of cases after using the objective outlier-exclusion algorithm (Rousseeuw 1998) at the 95% probability level.

4.7.5 Modeling

4.7.5.1 Modeling Dam Failure in Rainfall-Runoff Models

Riverine systems with upstream dams will, ordinarily, require the development of a rainfall-runoff-routing model (e.g. HEC-HMS, TR-20, etc.) to estimate a watershed's response to the Probable Maximum Precipitation (PMP). Potentially critical upstream dams would normally be included in the model. The final steps of the HHA approach include using this rainfall-runoff-routing model to simulate dam failure and perform hydrologic routing to the site.

While using HEC-HMS for river reach hydrograph routing has advantages, namely numerical stability and minimal data requirements, its ability to accurately route breach hydrographs is limited. It uses a simplified hydrologic (kinematic wave) routing method, compared to hydraulic (dynamic wave) routing method (such as that used in the HEC-RAS unsteady flow model), to estimate the effect of channel/floodplain storage on hydrograph attenuation and peak flow rates. See Section 3.2.4 for additional discussion on flood hydrograph routing.

HEC-HMS has the ability to, not only perform river reach routing, but also generate breach hydrograph at the dam given specific breach parameters. Similar to HEC-RAS, HEC-HMS uses forms of the weir and orifice equations to compute breach discharge values for overtopping and piping failure modes, respectively, at each time step to generate the breach hydrograph. As shown in Figure 2, the dam breach parameters in HEC-HMS include:

- Final Bottom Width (B_b)
- Final Bottom Elevation
- Left/Right Side Slope (Z)
- Breach Weir Coefficient (for Overtopping Breaches)
- Breach Formation Time
- Piping/Orifice Coefficient (for Piping Breaches)
- Initial Piping Elevation
- Failure Trigger

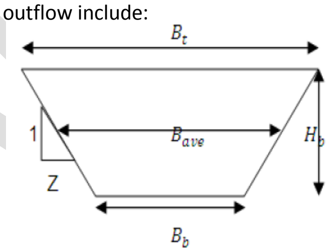
Additional information on developing breach parameters is provided in Section 3.2. Alternatively, the dam breach hydrograph can be developed outside the rainfall-runoff-routing model and entered as a user-defined hydrograph.

Regardless of the methodology, the HHA approach warrants the use of conservative breach parameters and peak outflow and attenuation estimates. As discussed further in Section 3.2.1, the HHA approach should also consider combinations of individual and cascading failures and make conservative assumptions regarding the trigger-settings for these combinations.

4.7.5.2 Modeling Dam Failure in 1D Unsteady Flow Models

Frequently, a refined site-specific analysis is desired to predict dam failure hazard conditions at a nuclear site, accounting for time-progression of the breach and flood attenuation storage along the riverine/floodplain system between the dam and nuclear site. The computer modeling tool frequently used for this analysis is the USACE HEC-RAS Unsteady-Flow model.

HEC-RAS generates a breach hydrograph by calculating discharge values in discrete time-steps as the breach progresses. At each time-step, HEC-RAS calculates a discharge (with a known head) using the weir equation (for an overtopping breach) or orifice equation (for a piping breach). The average discharge is used to estimate the volume released, corresponding drop in pool elevation, and discharge for the subsequent time-step to construct the breach hydrograph. The breach parameters needed for the USACE HEC-RAS Unsteady-Flow model will be the focus of this section. Figure 4 shows the HEC-RAS window view that receives the dam breach parameters. The parameters affecting outflow include:



- Final Bottom Width (B_b)
- Final Bottom Elevation
- Left/Right Side Slope (Z)
- Breach Weir Coefficient (for Overtopping Breaches)
- Full Formulation Time
- Piping/Orifice Coefficient (for Piping Breaches)
- Initial Piping Elevation
- Failure Trigger
  (Water surface elevation, water surface elevation + duration, or user-defined time)
- Starting Water Surface Elevation

4.7.5.3 Modeling Dam Failure in 2D Models

xxxx

5 Pertinent References

Comment [A45]: Divide up references, similar to ANS 2.8 –
FFTF - later

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Comment [initials46]: FFTF – deleted duplicate reference

Appendix A

Screening Upstream Dams with Negligible effect of Failure at the Site

Example Screening Evaluation

As an example, use the USBR (1982) equations for attenuation and peak outflow estimates, respectively, as follows:

Equation 1

Equation 2

Converting the USBR (1982) Q_p equation to English units (where 1 cms = 35.315 cfs and 1 meter = 3.281 feet),

Simplifying,

Equation 3

Substituting Equation 3 into Equation 1,

Equation 4

With a given allowable Q_r (the attenuated peak flow at the site; say a certain % of the PMF), the threshold between 'site-specific dam failure warranted' and 'dams having inconsequential impact at site' can be established. Re-arranging Equation 4 to solve for downstream distance from dam (in miles), X,

Equation 5

Equation 5 is plotted on Figure 2 for a range of allowable dam breach peak flow rates at the site, from 1,000 to 200,000 cfs. Figure 2 can be used to further screen consequences of dam failure. With height and location of dams known, from state dam inventories, and the assistance of GIS tools, information can be plotted on Figure 2 to assess the need for site-specific evaluation. For example, it has been established that a nuclear site can accommodate 5,000 cfs from upstream dam failure, in addition to the PMF peak flow rate. According to Figure 2, dams with combinations of distances and dam heights to the right of the 5,000-cfs curve (e.g. 200 miles, 50 feet; 250 miles, 100 feet) can be assumed to have inconsequential affect from dam failure and eliminated from further consideration. Having multiple dams within the same distance range should factor into the allowable peak discharge per dam.

Comment [g47]: Possibly move to an appendix.
FFTF – this appendix is being rewritten. Comments will be addressed with the revision
Describe limits of scope of applicability (downstream distance, dam height, etc...) and reflect limits in figure.

Comment [g48]: What is the definition of Qp

Comment [g49]: What is the definition of Qr

Comment [g50]: Needs further development to address this issue; define limits in example application of USBR equations.

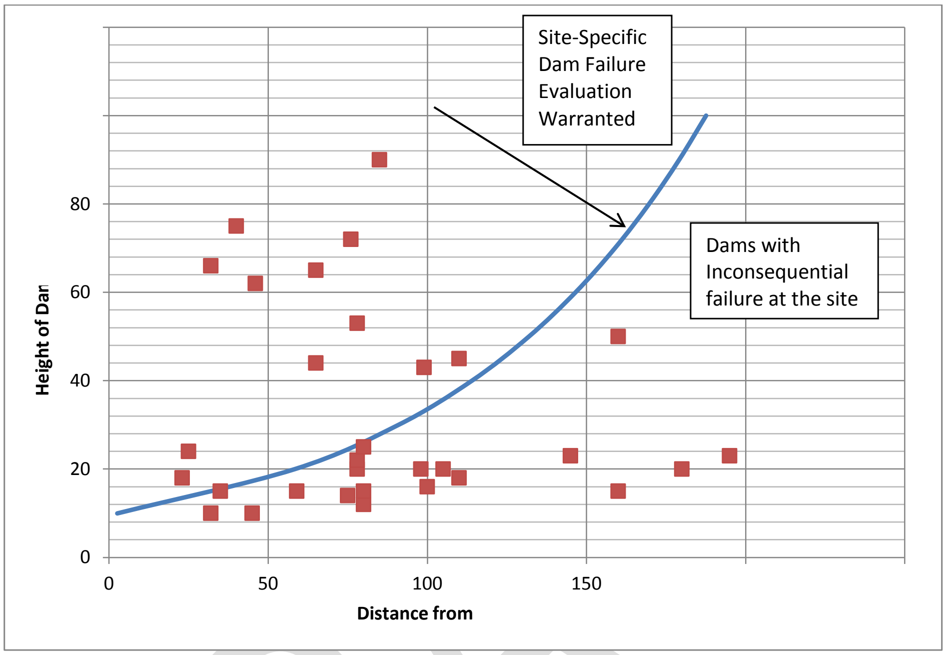

Figure 8 – Distance/Height of Dam Plot

Add example for multiple dams and zones.

Comment [g51]: Possibly move to an appendix.

Describe limits of scope of applicability (downstream distance, dam height, etc...) and reflect limits in figure.

Appendix B

Failure Mode Examples

Appendix C

Seismic

Example Seismic Evaluation

Take the case of a 100ft tall earthfill dam. This dam is a well constructed zoned earthfill dam with a wide crest, compacted clay core and well-designed filters and drains. The freeboard at the normal operating level is 15 feet. Because of the overall construction, the failure criteria for crest settlement was established to be 10% of the dam height, or about 10 feet.

Previous analysis was done for a PGA of 0.15g, based on the median deterministic ground motions estimated at the time of the previous work. The results of the previous analysis showed that the expected seismically induced permanent crest settlement was about 4 feet. The analysis would then be revised by simply increasing the PGA (or scaling the input time history) until the estimated seismic crest settlement is 10 feet. This PGA value would then be considered the median PGA causing failure, and a lognormal fragility curve could be constructed about this median value using an assumed uncertainty (□$_{\text{ln-PGA}}$) of about 0.55.

Comment [A52]: More to it than crest settlement; liquefaction and finite element analysis

The probabilities of failure at each discreet ground motion level are then multiplied by the annual probabilities of exceedance for that ground motion level, as determined from the simplified PSHA in Step 1, to estimate the annual probability of failure due to seismically induced crest settlement. If fault offset were considered a potential failure mode, the above process would be repeated using the existing analyses for fault offset and the annual probability of failure from fault offset would be added to the annual probability of failure due to crest settlement.

As for the white paper, the primary issue I have with Appendix C is that the example assumes a lot of work has already been done, and that all a dam owner has to do is change the parameters to assess the margin and exceedance probability. For many/most of the dams, I suspect that the type of analysis that is referenced in Appendix C has never been done so we don't know the baseline results much less the margins. Only a minority of TVA dams have been analyzed for seismic deformations. There is a very substantial effort required just to be in a position to do the Appendix C margin analysis. The scope of the effort needs to be recognized by all interests so we can have a realistic set of expectations about what can be done and on what schedule.

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Appendix D

Additional Details on Breach Parameters

Peak Flow Regression Equations

- USBR (1982) Peak Outflow (Case Study for 21 dam failures)

Where:

Q_p = Peak breach discharge (cm/sec)

H_w = Height of water in the reservoir at the time of failure above the final bottom elevation of the breach (meters)

- Froehlich (1995b) Peak Outflow (Case Study for 22 dam failures)

Where:

Q_p = Peak breach discharge (cm/sec)

H_w = Height of water in the reservoir at the time of failure above the final bottom elevation of the breach (meters)

V_w = Reservoir volume at the time of failure (m^3)

- National Weather Service (NWS) Simplified Dam Break Model (for dam heights between 12 and 285 feet)

Where:

Q_b = Breach flow + non-breach flow (cfs)

Q_o = Non-breach flow (cfs)

B_r = Final average breach width (feet), approximately 1H to 5H or

C = 23.4 x A_s/B_r

A_s = Reservoir surface area at maximum pool level (acres)

H = Selected failure depth above final breach elevation (feet)

T_f = Time to failure (hours), approximately H/120 or minimum of 10 minutes or

K_o = 0.7 for piping and 1.0 for overtopping failure

V_s = Storage volume (acre-feet)

- Natural Resources Conservation Service (NRCS); formerly the Soil Conservation Service (SCS)

  For $H_w \geq 103$ feet, $Q_{max}=(65)H_w^{1.85}$

  For $H_w < 103$ feet, $Q_{max}=(1100)H_w^{1.35}$

  - But not less than $Q_{max}=(3.2)H_w^{2.5}$
  - Nor greater than $Q_{max}=(65)H_w^{1.85}$

  When width of valley (L) at water level (H_w) is less than ______, replace equation $Q_{max}=(65)H_w^{1.85}$ with $Q_{max}=(0.416)LH_w^{1.5}$

  Where:

  Q_{max} = Peak breach discharge (cfs)

  B_r = Breach factor (acre)

  V_s = Reservoir storage at the time of failure (acre-feet)

  H_w = Depth of water at the dam at the time of failure; if dam is overtopped, depth is set equal to the height of the dam (feet)

  A = Cross-sectional area of embankment at the assumed location of breach (square feet)

  T = Theoretical breach width at the water surface elevation corresponding to the depth, H_w, for the equation $Q_{max}=(65)H_w^{1.85}$

  L = Width of the valley at the water surface elevation corresponding to the depth, H_w (feet)

- Walder and O'Connor (1997)

  Add more details on this method.

Peak Flow Routing Regression Equations

The USBR (1982) provides a simplified method for estimating the peak flow reduction as a function of distance to the site (miles). This dam breach peak flow rate at the site can be added to the PMF peak to estimate the combined flooding impact at the site.

Where:

X = Distance downstream of the dam measured along the floodplain (miles)

Q_r = Peak discharge corresponding to distance X (cfs)

Q_p = Peak dam break discharge at the dam (cfs)

Breach Parameter Regression Equations

- Xu and Zhang (2009)

Xu and Zhang (2009) expressed the five key breaching parameters (H_b, B_t, B_{ave}, Q_p, and T_f) in dimensionless forms and five controlling parameters as follows.

Breaching parameters	
Breach depth	$Y_1 = H_b / H_d$
Breach top width	$Y_2 = B_t / H_b$
Average breach width	$Y_3 = B_{ave} / H_b$
Peak outflow rate	$Y_4 = Q_p / \sqrt{g V_w^{5/3}}$
Failure time	$Y_5 = T_f / T_r$

Control variables			
Dam height	$X_1 = H_d / H_r$		
Reservoir shape coefficient	$X_2 = V_w^{1/3} / H_w$		
Dam type	X_{31}	X_{32}	X_{33}
with corewalls	1[a](e[b])	0(1)	0(1)
concrete faced	0(1)	1(e)	0(1)
homogeneous/zoned-fill	0(1)	0(1)	1(e)
Failure mode	X_{41}	X_{42}	
overtopping	1(e)	0(1)	
seepage erosion/piping	0(1)	1(e)	
Dam erodibility	X_{51}	X_{52}	X_{53}
high erodibility	1(e)	0(1)	0(1)
medium erodibility	0(1)	1(e)	0(1)
low erodibility	0(1)	0(1)	1(e)

Note: [a]Values for additive regression analysis;
[b]Values for multiplicative regression analysis.
H_r = 15 m; T_r = 1 hour.

Figure 9- Summary of the Five Breaching and Control Parameters (Xu and Zhang, 2009)

A multi-variable regression analysis was conducted to generate the following equations for breach parameters:

Where:

- Variables use metric units (meters for length/width/height and cubic meters for volume).
- Time variables are in units of hours.
- H_r = 15 meters.
- T_r = 1 hour.
- $B_2 = b_3 + b_4 + b_5$ for Equation 14.
- $B_3 = b_3 + b_4 + b_5$ for Equation 15.
- $B_5 = b_3 + b_4 + b_5$ for Equation 17.

- b_3 represents the type of dam, b_4 represents the failure mechanism of breach, and b_5 and C_{11} represents erodibility for the respective equations.
- See Table 1.

Table 2 - Constants for Use in Breach Parameter Equations (Xu and Zhang, 2009)

Variable	B_2 (Eq. 14)	B_3 (Eq. 15)	B_5 (Eq. 17)
Dam Description (b_3)			
Corewalls	0.061	-0.41	-0.327
Concrete Face	0.088	0.026	-0.674
Homogeneous/Core Fill	-0.089	-0.226	-0.189
Failure Mode (b_4)			
Overtopping	0.299	0.149	-0.579
Seepage/Piping	-0.239	-0.389	-0.611
Erodibility (b_5)			
High	0.411	0.291	-1.205
Medium	-0.062	-0.14	-0.564
Low	-0.289	-0.391	0.579
Erodibility (C_{11} in Eq. 13)			
High	1.04		
Medium	0.947		
Low	0.804		

Xu and Zhang (2009) also provide the results of two case studies, the Banqiao and Teton dam failures. Refer to Table 2 and Table 3.

- NRCS TR-66 (USDA 1985) provides a methodology for computing outflow hydrographs for overtopping breaches of earthen dams.

Where:

$Q_{t=tn}$ = Peak discharge at time t_n of breach hydrograph (cfs) (see previous section)

Q_p = Peak breach discharge (cfs)

V = Initial storage volume (cubic feet)

t = Time after peak (seconds)

- Others

Appendix E

Uncertainty

Appendix F

Sample of Information Requested from Dam Owners

Overall

1. Original design memorandums for each of the main stem dams.
2. As-built plans and O&M manuals for each main stem dam.
3. Operating rules of gates and releases for each main stem dam.
4. Emergency operation procedures for the main stem dams.
5. Spillway design hydrographs for each main stem dam.
6. Spillway and gate rating curves for each main stem dam.
7. Most recent reservoir elevation-capacity data for each main stem reservoir.
8. Original HEC-2 and or HEC-RAS models.
9. Recent extreme Precipitation Meteorological Studies.
10. Available documentation and electronic models developed flood-frequency studies.
11. All available documentation and electronic models for upstream dam break studies.
12. HEC-HMS models watershed of adjacent waterway.
13. LiDAR data.
14. 2011 Flooding high-water data.
15. Historic hydrology information or flooding reports.
16. Annual inspection reports for critical upstream dams.
17. Historic aerial/topography/navigation mapping.
18. Any additional information (e.g. in-process, planned, proposed) that may be relevant to the hazard reevaluation efforts.

Seismic

1. Location of Dam.
2. Design and/or as-build drawings.
3. Type of soil (material) used to construct the dam.
4. Characteristics of the foundation soils (or rock).
5. Is the dam a rock fill dam or zoned?
6. What are the slopes of the outer embankment and slopes of any zones within the dam?
7. Are there any filter drains in the dam construction?

8. Type of wave protection provided upstream and is the dam grassed or riprapped on the downstream side?
9. The degree of compaction was used for the earth construction. How thick were the lifts when constructed?
10. What are the design water levels (both upstream and downstream)?
11. Is there a concrete or other spillway through the dam?
12. Is there an overflow (emergency spillway and at what elevation)?
13. Height and length of dam.
14. How is the dam integrated into the abutments?
15. Is there a key trench for seepage control?
16. Is there a slurry wall or other seepage cutoff through the dam (most likely in the center)?
17. Was any slope stability performed and for what conditions?
18. Soil properties of the material(s) used to construct the dam.
19. Specifications for the construction of the dam.

Appendix G

Overview of Flood Hydrograph Routing

1-Dimensional

Flood hydrograph routing in a 1-dimensional model is a procedure to determine the time and magnitude of flow passing though a hydrologic system, such as reservoirs, ponds, channels, floodplains, etc. Flood routing accounts for changes in the time distribution of flood flows caused by storage and attenuation. The effect of storage is to re-distribute the hydrograph by shifting the centroid of the inflow hydrograph by the time of re-distribution to form the outflow hydrograph. The time of re-distribution occurs for level pool or reservoir routing situations. For very long channels, the entire flood wave travels a considerable distance and the centroid of its hydrograph may then be shifted by a time period longer than the time of re-distribution; called time of translation. The total shift in centroid can be called the time of flood movement, equal to the combined effect of the time of re-distribution and time of translation. See Figure 8.

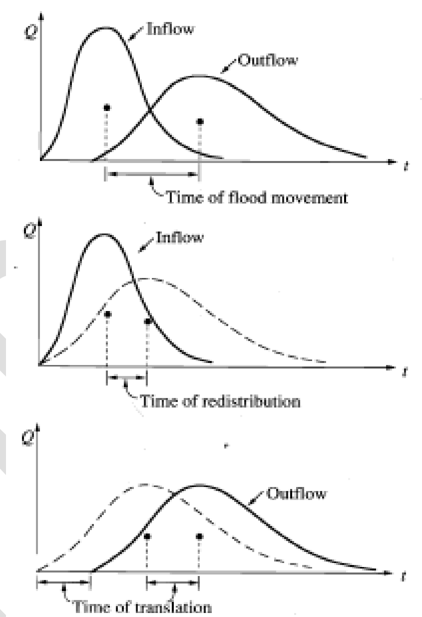

Figure 10 – Hydrograph Attenuation and Redistribution

The process for reservoir (level pool) routing can be expressed using the Continuity Equation (below). The inflow hydrograph, *I(t)*, is typically known. The outflow hydrograph, *Q(t),* can be solved with another relationship, called a storage function, to relate *S*, *I*, and *Q*.

Equation 6 - Continuity

Other routing computations, including channel/floodplain routing, can vary in complexity; this paper will focus on the two typically used for dam breach routing. Both are based on the St. Venant equation, derived from the combination of the continuity and momentum equations, as illustrated below. As indicated in Equation 7, the St. Venant equation can be applied in 1-dimensional models for:

- **Kinematic (Simplified) Wave Routing** – The kinematic wave routing is based on a finite difference estimation of the continuity equation and simplification of the momentum equation (assume $S_f = S_o$). As indicated in Equation 7, the solution assumes steady-state and uniform flow conditions. The kinematic wave routing method is used in the USACE HEC-HMS model.
- **Dynamic (Time-Dependent or Unsteady) Wave Routing** – The dynamic wave method is a more accurate routing procedure that solves the entire St. Venant equation (Equation 7) and considers changes in flow rates with respect to time, a factor that can be significant with a dam breach wave. The dynamic wave routing method is used in the USACE HEC-RAS (unsteady-flow) model, MIKE 21, the NWS FLDWAV model, and others. Developing a model using dynamic wave routing techniques involves much greater effort than the kinematic wave solution but produces more accurate results. After the initial setup, a dynamic wave model frequently requires refinements to cross-section spacing and computational time increments to reach and maintain model stability.

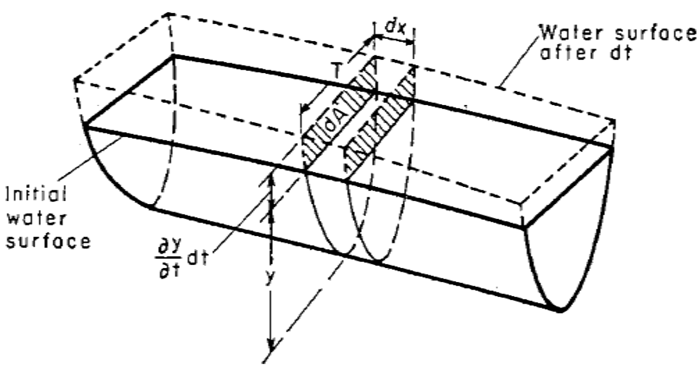

Figure 11 – Definition Sketch for St. Venant Equation

Equation 7 - St. Venant Equation

Unsteady Varied
Steady Varied
Steady Uniform
Kinematic Wave Solution
Dynamic Wave Solution

2-Dimensional

In some cases, flow pattern complexities, unusual dam failure configurations, and/or a desire for increased accuracy warrants the use of Two Dimensional (2D) (finite-element or finite-difference) hydrodynamic modeling to simulate the affects of dam failure. 2D models have the added advantage of producing velocity vectors (direction and magnitude) at the site to better assess hydrodynamic and debris loading conditions at the site due to dam failure. Some 2D models use finite-element solutions of continuity and momentum functions based on a triangular mesh, representing the surface terrain, developed from a series of points/nodes with X, Y, Z attributes. Other 2D models use finite-difference solution methods based on a surface terrain represented by grid elements. Some 2D models can be used to generate and route breach hydrographs; others can only perform the hydrodynamic routing of a user defined breach hydrograph. Example models include:

- HEC-RAS 4.2 (currently being beta-tested but is expected to include a 2D component)
- RiverFLO-2D
- FLO-2D
- River-2D
- MIKE-21
- SRH-2-D Model (The Bureau of Reclamations)