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

Comment [g1]: Recommend add section numbers

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.

- **~~Summary of~~ 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.

  - ~~Evaluate all relevant flooding mechanisms using present-day regulations, methodologies, engineering practices, and modeling software (Phase 1). Actions associated with Phase 2 (above) are not being requested at this time, pending completion of the Phase 1 evaluations.~~
  - ~~Where the reevaluated flood exceeds the design basis, submit an interim action plan that documents actions planned or taken to address safety issues (if any) at the new hazard levels.~~
  - ~~Perform an integrated assessment of the plant for the entire duration of the flood conditions to identify vulnerabilities and corrective actions under full power operations and other plant configurations. The scope also includes those features of the ultimate heat sinks that could be adversely affected by flood conditions and lead to degradation of the flood protection. (The loss of ultimate heat sink from non-flood causes is not included.)~~

  Requested Information:

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Comment [g3]: Include whole requested information list from 10 CFR 50.54(f) letter

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.*

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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 Guidance**

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:

~~A. Probable Maximum and Seismically Induced Floods on Streams and Coastal Areas (which references American National Standards Institute (ANSI) Standard N170-1976, superseded by ANSI/ANS (American Nuclear Society) 2.8, "Determining Design Basis Flooding at Power Reactor Sites", July 28, 1992)~~

~~B. Alternative Methods of Estimating Probable Maximum Floods~~

~~C. Simplified Methods of Estimating Probable Maximum Surges~~

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; ~~superseding Appendix A of RG 1.59 (ANSI/ANS 2.8).~~

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).

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Comment [g4]: Reference Standard Review Plan in effect at time of issuance of 10 CFR 50.54(f) letter

~~Note that NUREG/CR-7046 does not address tsunamis;~~ 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.

- **Deterministic versus Probabilistic Approaches**

  NUREG/CR-7046 provides only an introduction to the application of probabilistic methods in flood estimation at nuclear power plants, acknowledging that detailed methodology and guidance are currently not available. For flooding hazard reevaluations, deterministic methods should be used. Probabilistic methods may be included to establish a relationship between flood magnitude and exceedance probability in developing the Integrated Assessment, required if the reevaluated flood hazard is not bounded by the current licensing basis.

Comment [g5]: There is limited experience on doing probabilistic floods. Significant NRC review would be expected.

- **Hierarchical Hazard Assessment (HHA) Approach**

  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 ~~while ensuring that these conditions are consistent with those used by Federal agencies in similar design considerations~~. 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.

- **Dam Breaches and 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. 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. 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 ~~State and Federal Agencies~~entities with appropriate jurisdiction for dams may be incorporated into the analysis~~used~~. 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).

Should we define a dam? The paper does say: "State dam inventories and classification systems can be used to identify dams within the watershed of an adjacent river." Is that enough? It implicitly eliminates any impounded reservoirs on the site. Do we need to be clearer or is better to leave it as it is?

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2 Scope and 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 detailed 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. The following is a summary of ~~guidance provided in NUREG/CR-7046~~definitions:

Comment [g6]: Define each failure mechanism

- **Hydrologic Upstream Dam Failure:** ~~PMF hydrographs, generated from PMP scenarios discussed previously, should be routed through upstream dams using the USACE HEC-HMS (or equivalent) model. If the model indicates that one or more dams are unable to safely pass the PMF (i.e. the PMF hydrograph overtops an unprotected portion of the dam(s)), the dams should be breached to coincide with the peak of the PMF. Dams in series should be breached as cascading failures~~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.
- **Seismic Upstream Dam Failure:** ~~NUREG/CR-7046, Appendix D states that "dam breach usually refers to a structural failure of the embankment that may be caused by a hydrologic event (e.g., overtopping of a dam during a flood, leading to erosion of the dam face or piping and resulting in erosion of the embankment) or a seismic event". Seismic events are not expected to occur coincidently with a large hydrologic event. It is also expected that large hydrologic events (i.e., the PMF) bound the seismic events since release of stored water impounded by the dam during the PMF would be greater than during the seismic event; although, the seismic event may produce the bounding warning time. The methods for evaluating a seismic failure are per Appendix H.2 of NUREG/CR-7046. The following seismic/precipitation combinations should be considered:~~

  ~~1. Safe shut-down earthquake (SSE) and 25-year precipitation.~~

~~2.~~• ~~Operational basis earthquake (OBE) and ½ PMP or 500-year precipitation, whichever is less.~~Dam failure induced by an earthquake that causes weakening of the dam's structural components, embankment, foundation, and/or abutments. NUREG/CR-7046 associates a precipitation event with an earthquake, based on a $1x10^{-6}$ annual exceedance probability, to create seismically-induced failure scenarios.

- **Sunny~~-~~Day Upstream Dam Failure:** A 'sunny~~-~~day' dam failure is ~~typically~~ not associated or concurrent with an initiating external event (such as an extreme flood or earthquake) and may ~~occur due to failures of embankment material or foundation, such as those due to piping through the embankment~~result from a structural or operational deficiency. ~~'Sunny day' failures would normally not exceed flood magnitudes resulting from the hydrologic and seismic failure scenarios discussed above. However, it is recommended that the affects of a 'sunny day' failure be considered particularly when mitigation measures protect safety-related SSCs from such a failure, given the more limited warning time generally associated with a 'sunny day' failure.~~
- **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).
- **Security Threats:** It is assumed that failures from modes other than natural hazards (e.g. terrorism) do not need to be considered in the Recommendation 2.1, Flooding Reevaluations.

3 Approach

3.1 Hierarchical Hazard Assessment (HHA) Approach for Upstream Dam Failure

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.' Per Figure 1, the HHA approach to dam failure evaluations includes two key steps:

1. Identify non-critical ~~Eliminate~~ dams ~~judged to have~~having inconsequential affect of failure at site
2. Simplified modeling assuming all potentially critical dams fail during PMF
3. ~~Refined/Site-Specific Dam Failure Evaluation~~

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Comment [g7]: Recommend quoting the ANSI guide instead. ANSI/ANS 2.8-1992, Section 9.2.1.2

Comment [g8]: However a justification for this would need to be provided

Warning time? Assumptions for initial water levels and failure modes?

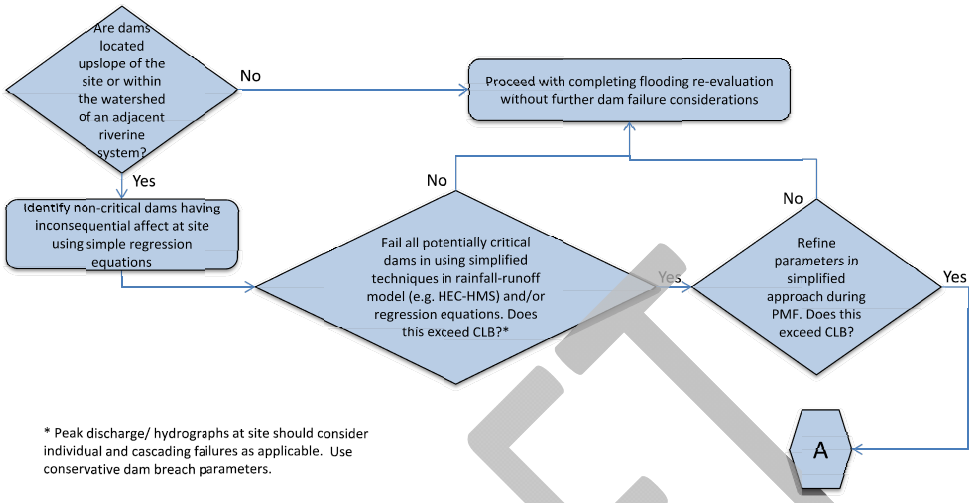

Figure 1- HHA for Dam Failure

3.1.1 Eliminate Dams Judged to have Inconsequential Affect of Failure at 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 judging which dams can be screened from further consideration.

State dam inventories and classification systems can be used to identify dams within the watershed of an adjacent river. Most states use a system to classify the size and hazard potential of each dam that can be used to identify dams that can be eliminated from further consideration (e.g. small, low-hazard dams). The only exception are dams immediately upslope from the site; failure from even small, upslope dams can have adverse consequences at the site.

When in question, a relationship can be developed between the size of dam (e.g. height) and distance to site to further screen out dams from further consideration. Peak flow and attenuation estimates can be used to develop a relationship between dam size and distance and establish thresholds for dams with inconsequential failure. Key to this step is understanding the allowable tolerance for increase in flow rate at the site, above the PMF peak flow rate, resulting from dam failure. If more than one dam is being evaluated for screening, the dams should be grouped in zones to cluster dams with similar distance from the site. The allowable tolerance for increased flow at the site should be divided by the number of dams in each zone to establish an allowable peak flow per dam. See the example in Appendix .

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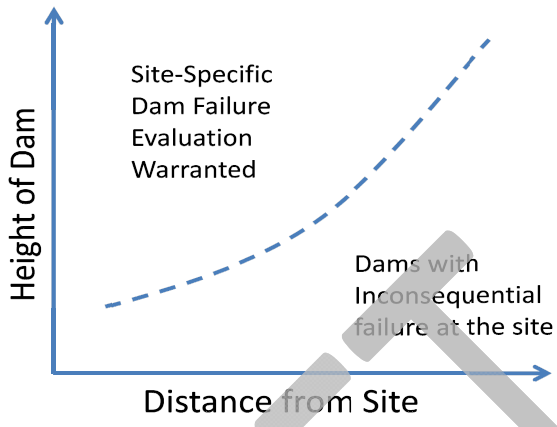

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3.1.2 Assume Failure during PMF using Simplified Techniques

3.1.2.1 Peak Flow Rate Regression Equations

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 discharge as a function of dam and/or reservoir properties based on real dam failure data. Four peak outflow discharge estimation methods are ~~presented~~ listed below and presented in more detail in Appendix ___. Note, original technical papers or documentation should be reviewed prior to using these equations to understand their limitations. Wahl (2004) indicates that the Froehlich (1995b) 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 (1995) equations 'remain valid for conservative peak-outflow predictions' for embankment dams.

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- USBR (1982) Peak Outflow (Case Study for 21 dam failures)
- Froehlich (1995b) Peak Outflow (Case Study for 22 dam failures)
- National Weather Service (NWS) Simplified Dam Break Model (for dam heights between 12 and 285 feet)
- Natural Resources Conservation Service (NRCS); formerly the Soil Conservation Service (SCS)

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As part of the HHA process, attenuation of the peak discharge can be ignored to conservatively account for the affect of the breach at the site. However, 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)

3.1.2.2 Hydrologic (HEC-HMS) Model

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). Upstream dams, whose failures are judged to affect the site, 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. For the purpose of this paper, the HEC-HMS model will provide the basis for this stage in the HHA process.

While using HEC-HMS for river reach hydrograph routing has advantages, namely numerical stability and minimal data requirements, its ability to accurately routing 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 affects of channel/floodplain storage on hydrograph attenuation and peak flow rates. See Section 3.2.5 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 certain 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)
- Full Formulation Time
- Piping/Orifice Coefficient (for Piping Breaches)
- Initial Piping Elevation
- Failure Trigger

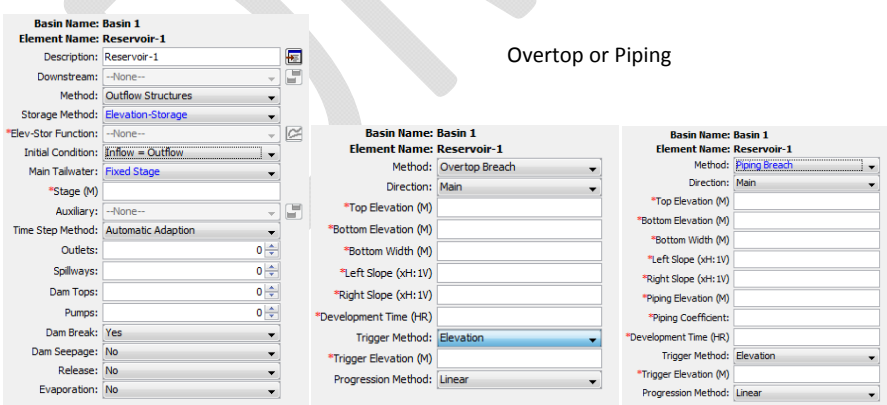

Figure 2- Dam Breach Menu Options in HEC-HMS

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

$$Q_{t=t_n} = Q_p e^{(-t_n Q_p/V)}$$

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 fee)

t = Time after peak (seconds)

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.

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3.2 Refined Upstream Dam Failure Evaluation

3.2.1 ~~Breach Scenarios~~Individual and Cascading Failures

~~*3.2.1.1 Individual and Cascading Failures*~~

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; and
2. Cascading or domino-like failures of dams."

Appendix D, Part D.1, of NUREG/CR-7046 provides additional guidance and examples for developing reasonable individual and cascading failure scenarios. These scenarios should be considered under each of the following failure modes. Per NUREG/CR-7046, three types of failure modes should be evaluated:

1. Hydrologic Failure (Failure Induced by PMF);
2. Seismically-Induced Failure; and
3. Sunny-Day Failure.

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3.2.2 Failure Modes

~~*3.2.1.2*~~*3.2.2.1* ~~*Overtopping*~~ *Hydrologic Failure*

Comment [g9]: Begin by discussing individual dam failure, then develop into successive dam failures. Discuss storm centering.

Comment [g10]: Recommend discussion of how to use analyses from other entities (e.g., PMP for an individual dam). Refer back to ANSI 2.8. This discussion may be better placed in another section.

Step 1A

If failure of all potentially critical dams is assumed, formulate breaches, per Section 3.2.3, and trigger failures in the site's bounding PMF hydrologic (HEC-HMS) 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. More accurate breach hydrograph development and routing techniques (e.g. HEC-RAS unsteady-flow, 2D models, etc.) can be used to further refine the affects of more critical dam failures at the site. No addition consideration is needed for dam failure.

Step 1B

If the licensee intends to credit some or all of the potentially critical dams as unlikely to experience hydrologically-induced failure, iteratively run the site's bounding PMF hydrologic (HEC-HMS) model to isolate critical dams whose failures have a significant impact at the site. Per Appendix D, Part D.1, of NUREG/CR-7046, this step should consider reasonable combinations of individual and/or cascading failures. Non-critical dams should be assumed to fail in subsequent modeling steps. Proceed to Step 2 for additional evaluations of critical dams.

Step 2

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.*" The primary criterion for assessing hydrologic failure is overtopping. 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.

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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.

These conditions can be applied to the hydrologic (HEC-HMS) model to determine if unprotected portions of critical dams (portions not designed to convey flow) are overtopped by:

1. The site's bounding PMF; or
2. The dam's bounding PMF.

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.

Critical dams subject to overtopping by either of the above two PMF scenarios should be included as failing in the site's bounding PMF model. In situations where a critical dam does not overtop during the site's bounding PMF but does overtop during the dam's PMF, the licensee has the option to proceed to Step 3 to justify non-failure during the site's bounding PMF and, assuming justification is sufficient, consider non-failure of the dam in the site's bounding PMF model. However, the licensee should, in such cases, 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.

As indicated previously, ANS 2.8, Section 5.5.4, specifies that, *"if no overtopping is demonstrated, the evaluation may be terminated and the embankment may be declared safe from hydrologic failure"*. Nevertheless, additional information, discussed in Step 3, may be required to demonstrate safety under PMF loading conditions, even without overtopping.

Step 3

For critical dams, where non-failure justification is sought, the information below (from Section 5.5.4 of ANS 2.8) may be required to further demonstrate safety from failure due to instability, erosion, sliding, or overturning. Detailed stability analysis of dams requires documentation of structural dimensions and condition 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.

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.

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- **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 be evaluated for breaching from overtopping. If there are two or more independent embankments, it may be necessary to fail only one if it produces the most critical flood wave.

~~The overtopping hydrologic dam failure shall be based on the PMP/snowmelt scenario that produces the bounding PMF at the site. (See Section H.1 of NUREG/CR-7046.) The hydrologic models (i.e. HEC-HMS) used to develop the PMF hydrographs at the site should include routing computations for dams having an attenuation and/or potential dam failure affect at the site. As part of the HHA approach in NUREG/CR-7046, dams can be assumed to fail at the peak reservoir levels, with due consideration to successive or domino dam failures, during the bounding PMP/snowmelt scenario. Conservative breach parameters and simplified routing procedures in HEC-HMS, as discussed in Section __, can be used to assess the impact of this 'total' breach scenario at the site or to evaluate which dam failure(s) is having the greatest affect at the site.~~

~~If further refinements are warranted, routing computations in HEC-HMS (or equivalent) can be used to identify dams having 'unprotected' portions overtopping during the bounding PMP/snowmelt scenario. Dams are typically equipped with emergency or auxiliary spillways designed to safely pass extreme flows. 'Unprotected' portions are those that were not designed to pass flow, such as the top of an unarmored earthen dam. Design or as-built drawings of the dam(s) in question should be obtained to understand which portions are protected and designed to pass flow. The refined 'overtopping' failure scenario would included failure of dams shown to have unprotected portions overtopping combined with the bounding PMP/snowmelt scenario; again, with due consideration to successive or domino dam failures.~~

~~3.2.1.3~~3.2.2.2 *Seismically-Induced Failure*

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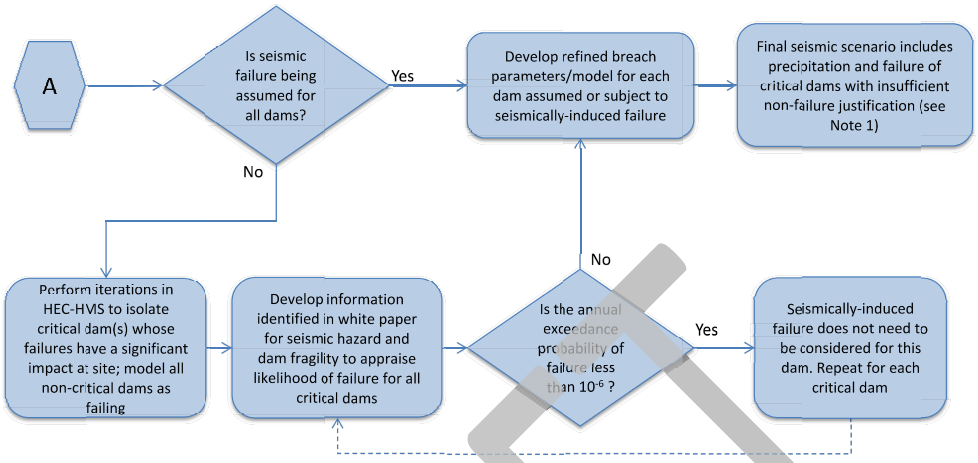
Comment [g11]: Clarify this thought

Comment [g12]: Possibly develop a logic diagram depicting this thought process?



1. Dam failure scenarios should include combinations of individual and/or cascading failures as applicable. Trigger failure for each dam at optimal time of seismic event.
2. Breach parameters/formulations should be specific to the type of dam (earthen, rock fill, concrete/arch, etc.) and mode of failure (overtopping or piping) using realistic but conservative physics -based assumptions.

As stated previously, seismic events are not expected to occur coincidently with a large hydrologic event. It is also expected that large hydrologic events (i.e., the PMF) bound the seismic events since release of stored water impounded by the dam during the PMF would be greater than during the seismic event; although, the seismic event may produce the bounding warning time. The methods for evaluating a seismic failure are per Appendix H.2 of NUREG/CR-7046. The following seismic/precipitation combinations, thought to have an annual probability of exceedance of less than $1x10^{-6}$ (ANS, 1992), should be considered:

1. Safe shut-down earthquake (SSE) and 25-year precipitation.
2. Operational basis earthquake (OBE) and lesser of the ½ PMP or 500-year precipitation~~, whichever is less~~.

The combinations described in NUREG/CR-7046 are directly from ANS 2.8 (1992), specifically Sections 6.2 and 9.2.1.2, and Regulatory Guide 1.59. As part of the HHA approach in NUREG/CR-7046, a failure of all upstream dams under any seismic event and the lesser of the ½ ~~1/2 PMF~~PMP and 500-year precipitation, with due consideration to successive or domino dam failures, would produce a bounding scenario. Any postulated breach should be timed to coincide with the maximum reservoir level or optimal time for multiple dams, even if some of the dams have not yet reached maximum levels. Given that the initiating event is an earthquake, it would be unreasonable to vary failure times to force peak flow rates to reach the site at the same time. ~~To further refine the process, the details described in ANS 2.8 (1992) and RG 1.59 would be required.~~

If dams are not assumed to fail from seismic activity, 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

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Comment [g13]: See previous comment on this wording for water levels during the seismic event

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.

A dam's structural stability shall be demonstrated to survive a local equivalent of the Safe Shutdown Earthquake (SSE) and Operating Basis Earthquake (OBE). Given the lack of a probabilistic SSE, as described in 10 CFR 100.23 (d)(1), the deterministic SSEs and OBEs, as defined by the current licensing basis, should be used in this evaluation. These earthquakes may be used for dams that are within the same general tectonic region. For dams that are large distances from the nuclear site, the dam's maximum credible or design earthquake may be used to evaluate for the combined events by using the annual exceedance probability. Per ANS 2.8 (1992), the average annual exceedance for the combined events of 1×10^{-6} is an acceptable goal for selection of flood design bases for the nuclear power reactor plants. Therefore, a cumulative annual exceedance may be determined for the combined flood and earthquake event and compared to the acceptable goal. If the design earthquake and flood cumulative annual exceedance probability is not comparable, additional dam analyses may be required.

Comment [g14]: Additional consideration needs to be given to ongoing review of seismic hazards and the interrelationship with this analysis

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 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 are used and the required factors of safety per those standards are satisfied. In addition, the annual exceedance probability for maximum credible or design earthquake loading, combined with the hydrologic event annual exceedance probability, shall be 1×10^{-6} or less.

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

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- Estimate ground motion hazard curves;
- Develop failure criteria for each potential seismic failure mode;
- Use existing analyses to estimate the seismic capacity for each potential failure mode;
- Assumed fragility curve shapes; and
- Combine the ground motion hazard curves with the fragility curves to estimate annual probability of failure. Sum the probabilities of failure for each failure mode to estimate the aggregate annual probability of seismic failure.

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The following describe each of the key parameters:

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- Ground Motion Hazard Curves – 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). 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].
- Develop failure criteria for each seismic failure mode. The criteria should be based on dam type (concrete, earthfill, rockfill, etc), construction details (slope protection, filters and drains, core width, etc), and overall construction quality. Examples of failure criteria could be maximum crest settlement and fault offset at the foundation elevation.

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- 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 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 ground motion parameter corresponding to failure and an assumed uncertainty value (σ_{ln}) to develop lognormal fragility curves for each failure mode.
- Estimate the probability of failure 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.

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~~3.2.1.4~~3.2.2.3 'Sunny-Day' Failure

A sunny-day failure is, as the name implies, 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;

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- 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.

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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. Per Section 6.3 of ANS 2.8, "*dam failures from other onsite causes might result from gradual changes in, under, and adjacent to the dam. With proper inspection and monitoring, gradual changes threatening dam safety might be detected and adequate corrective measures can be taken*". The following describes the steps in a sunny-day failure evaluation:

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Step 1

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Iteratively run the site's bounding PMF hydrologic (HEC-HMS) model to isolate critical dams whose individual or cascading sunny-day failures have a significant adverse impact at the site. This step should consider induced failures of downstream dams in series. Given the nature of a sunny-day failure, it would be unreasonable to assume simultaneous individual failures. Identify the worst-case individual or cascading failure among critical dams.

Step 2A

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If failure of the worst-case critical dam is assumed, formulate breaches, per Section 3.2.3, and trigger failure assuming the water level is at [normal high-water] [the top of the dam]. For dams in series, failure should be triggered to maximize the affect of compounding flows from cascading failures. More accurate breach hydrograph development and routing techniques (e.g. HEC-RAS unsteady-flow, 2D models, etc.) can be used to further refine the affects of more critical dam failures at the site.

Comment [A15]: Need to discuss what's reasonable here. Sunny-day failure with water at the top of the dam doesn't fit the 'surprise' condition. It assumes a significant preceding event that would result in heightened alert.

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Step 2B

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

- Structural dimensions;
- Construction records;

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- Records from installed monitoring instrumentation and/or piezometer wells;
- Field surveys
- On-site inspection reports; 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; in which case, the next worst-case critical sunny-day dam failure (if applicable) should be evaluated.

~~A sunny-day failure is, as the name implies, 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. 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. Nevertheless, if a sunny-day failure of an upstream dam is thought to affect the site, information, such as ongoing monitoring (e.g. piezometer wells to monitor seepage), maintenance, and operational procedures, can be provided to demonstrate that a sunny-day failure is not credible.~~

Comment [g16]: Describe the dam-specific attributes of the maintenance, dam age, risk tolerance of the operating agency. See Information Notice 2012 regarding use of historical failure databases.

Durable maintenance agreement?

See also ANSI/ANS-2.8-1992 Section 6.3

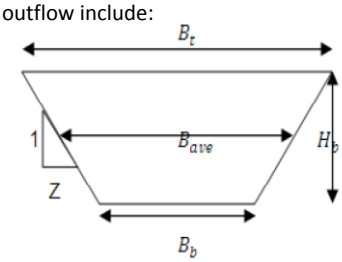
~~3.2.2~~3.2.3 Breach Formulation

~~3.2.2.1~~3.2.3.1 *Empirically-Based Methods*

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 3 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

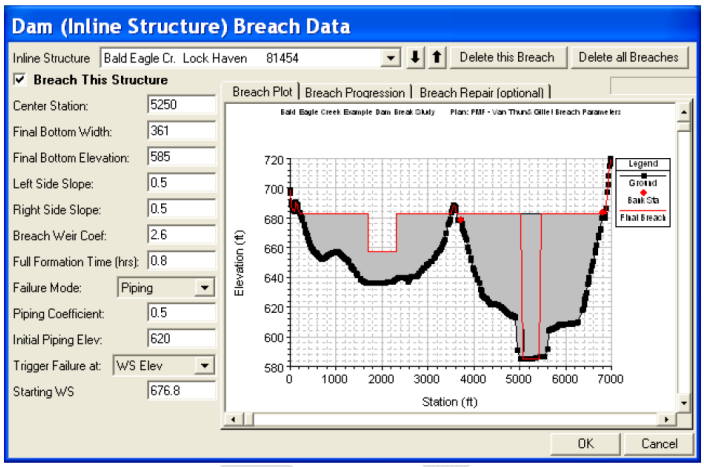


Figure 3 - HEC-RAS Dam Breach Editor

The Bureau of Reclamation (Wahl, 1988) provides additional literature review of breach parameters. Wahl (1998) compiles a list of methods to predict 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.

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

- Froehlich (1987, 1995a, 1995b) – 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 4. 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.

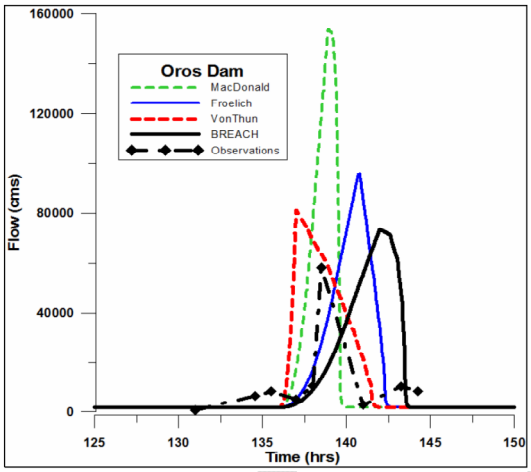


Figure 4 - Breach Hydrographs for Oros Dam (Gee, 2008)

Following a recommendation by Wahl (2008), Xu and Zhang (2009) developed equations to compute breach parameters for earth and rockfill dams. The new equations are based on widely accepted methods developed by Froehlich (1987 and 1995) and empirical data to close the gap between idealized parameters and an analysis of 182 earth and rockfill dam breach events. Of the 182 cases, Xu and Zhang (2009) used the 75 failure cases that had sufficient information to develop regression equations. 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)
  - Failure Time (T_f)

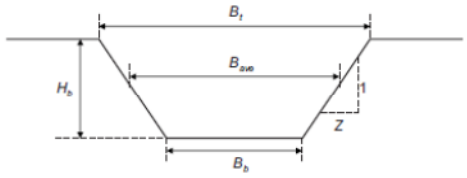


Figure 5 - Geometric Parameters of an Idealized Dam Breach (Xu and Zhang, 2009)

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.

~~3.2.2.2~~3.2.3.2 Physically-Based Methods

In 2004, the Centre for Energy Advancement through Technological Innovation (CEATI) formed a Dam Safety Interest Group (DSIG) to investigate the available physically-based numerical models to simulate embankment erosion and breach development. The DSIG group comprised members from the USACE,

USBR, USDA, BC Hydro, Elforsk (Scandinavian Utility), and EDF (French Utility). The objective was to compare the available modeling tools and recommend models for further development and use in prediction of embankment breach formulation. The review and validation by the CEATI DSIG Project included:

- International review of breach models;
- Selection of 3 most promising for closer evaluation;
- Review and collation of field and laboratory data;
- Evaluation of model performance against seven selected data sets;
    - Two from USDA Stillwater;
    - Three from the European IMPACT project; and
    - Two from actual dam failures (Oros, Banquio).

The DSIG Project concluded that:

- The HR BREACH and SIMBA/WinDAM best representative; and
- HR BREACH offers zoned cross-section analysis.

Table 1 provides a more comprehensive comparison of the findings.

Table 1 - Comparison of Physically-Based/Erosion Process Models

	HR-BREACH	SIMBA / WinDAM	FIRE2BRD	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

~~3.2.3~~3.2.4 Uncertainty in Breach Formulation

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

their research, which will produce bias towards the Xu and Zhang results. (See Table 2.) Nevertheless, the Xu and Zhang method appears to offer the least variability and seems to accommodate a wider range of situations.

Table 2 - 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.

~~3.2.4~~3.2.5 Breach Hydrograph Routing

~~3.2.4.1~~3.2.5.1 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 6.

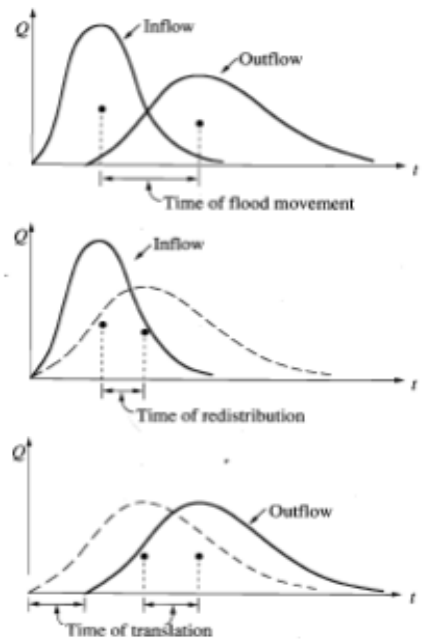


Figure 6 – Hydrograph Attenuation and Redistribution

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 1 - Continuity

$$\frac{dS}{dt} = I(t) - Q(t)$$

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 2, 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 2, 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 2) 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.

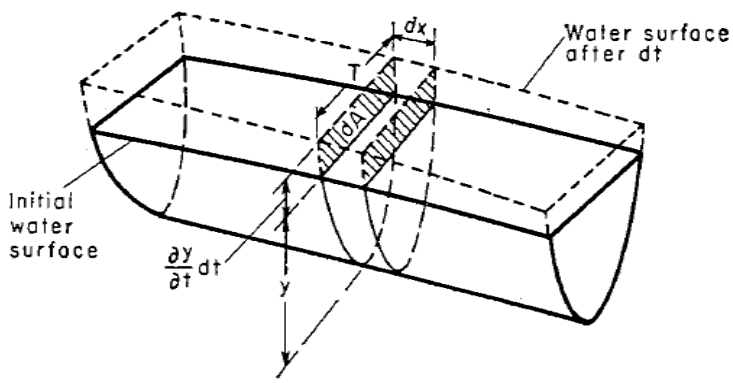


Figure 7 – Definition Sketch for St. Venant Equation

Equation 2 - St. Venant Equation

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\beta \frac{Q^2}{A}\right) + gA\left(\frac{\partial h}{\partial x} - s_f\right) = 0$$

Unsteady	Steady	Steady
Varied	Varied	Uniform

Kinematic Wave Solution

Dynamic Wave Solution

~~3.2.4.2~~3.2.5.2 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)

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APPENDIX A

Regression Equations (Peak Flow and Breach Parameters)

Peak Flow

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

$$Q_p = 19.1 H_w^{1.85}$$

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)

$$Q_p = 0.607 V_w^{0.295} h_w^{1.24}$$

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)

$$Q_b = Q_o + 3.1 B_r \left(\frac{C}{T_f + C/\sqrt{H}} \right)^3$$

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
$B_r = 9.5 K_o (V_s H)^{0.25}$

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
$T_f = \frac{0.59(V_s^{0.47})}{H^{0.91}}$

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 $T = \frac{(65)H_w^{0.35}}{0.416}$, 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) $B_r = \frac{V_s H_w}{A}$

  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)

Breach Parameters

<u>Xu and Zhang (2009)</u>

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		Control variables			
Breach depth	$Y_1 = H_b / H_d$	Dam height	$X_1 = H_d / H_r$		
Breach top width	$Y_2 = B_t / H_b$	Reservoir shape coefficient	$X_2 = V_w^{1/3} / H_w$		
Average breach width	$Y_3 = B_{ave} / H_b$	Dam type	X_{31}	X_{32}	X_{33}
Peak outflow rate	$Y_4 = Q_p / \sqrt{g V_w^{5/3}}$	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 time	$Y_5 = T_f / T_r$	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 8- 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:

$$H_b = \left(C_{11}\left(\frac{H_d}{H_r}\right)^{-0.048}\right) H_d \ (Equation\ 13\ in\ Xu\ and\ Zhang, 2009)$$

$$B_t = \left(1.062\left(\frac{H_d}{H_r}\right)^{0.092}\left(\frac{V_w{}^{1/3}}{H_w}\right)^{0.508} e^{B_2}\right) H_b \ (Equation\ 14\ in\ Xu\ and\ Zhang, 2009)$$

$$B_{ave} = \left(0.787\left(\frac{H_d}{H_r}\right)^{0.133}\left(\frac{V_w{}^{1/3}}{H_w}\right)^{0.652} e^{B_3}\right) H_b \ (Equation\ 15\ in\ Xu\ and\ Zhang, 2009)$$

$$T_f = \left(0.304\left(\frac{H_d}{H_r}\right)^{0.707}\left(\frac{V_w{}^{1/3}}{H_w}\right)^{1.228} e^{B_5}\right) T_r \ (Equation\ 17\ in\ Xu\ and\ Zhang, 2009)$$

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 **Error! Reference source not found.**Table 1.

Table 3 - 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 **Error! Reference source not found.**Table 2 and **Error! Reference source not found.**Table 3.

Others

APPENDIX B

Example Screening Evaluation

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

Comment [g17]: Possibly move to an appendix.

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

Equation 3

$$Q_r = 10^{\log(Q_p)-0.01X}\ (Q_p\ in\ cfs; X\ in\ miles)$$

Equation 4

$$Q_p = 19.1H_w^{1.85}\ (Q_p\ in\ cms;\ H_w\ in\ meters)$$

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

Comment [g18]: What is the definition of Qp

$$Q_p = 35.315(19.1(0.305H_w)^{1.85})$$

Simplifying,

Equation 5

$$Q_p = 74.980H_w^{1.85}\ (Q_p\ in\ cfs;\ H_w\ in\ feet)$$

Substituting **Error! Reference source not found.**Equation 3 into **Error! Reference source not found.**Equation 1,

Equation 6

$$Q_r = 10^{\log(74.980H_w^{1.85})-0.01X}\ (Q_p\ in\ cfs; H_w\ in\ feet; X\ in\ miles)$$

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 **Error! Reference source not found.**Equation 4 to solve for downstream distance from dam (in miles), X,

Comment [g19]: What is the definition of Qr

$$logQ_r = log\left(10^{\log(74.980H_w^{1.85})-0.01X}\right)$$

$$logQ_r = (log(74.980H_w^{1.85}) - 0.01X)(log10)$$

$$0.01X = log(74.980H_w^{1.85}) - logQ_r$$

Equation 7

$$X = 100(log(74.980H_w^{1.85}) - logQ_r)\ (Q_r\ in\ cfs; H_w\ in\ feet; X\ in\ miles)$$

Error! Reference source not found. Equation 5 is plotted on **Error! Reference source not found.**Figure 2 for a range of allowable dam breach peak flow rates at the site, from 1,000 to 200,000 cfs. **Error! Reference source not found.**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 **Error! Reference source not found.**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 **Error! Reference source not found.**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 [g20]: Needs further development to address this issue; define limits in example application of USBR equations.

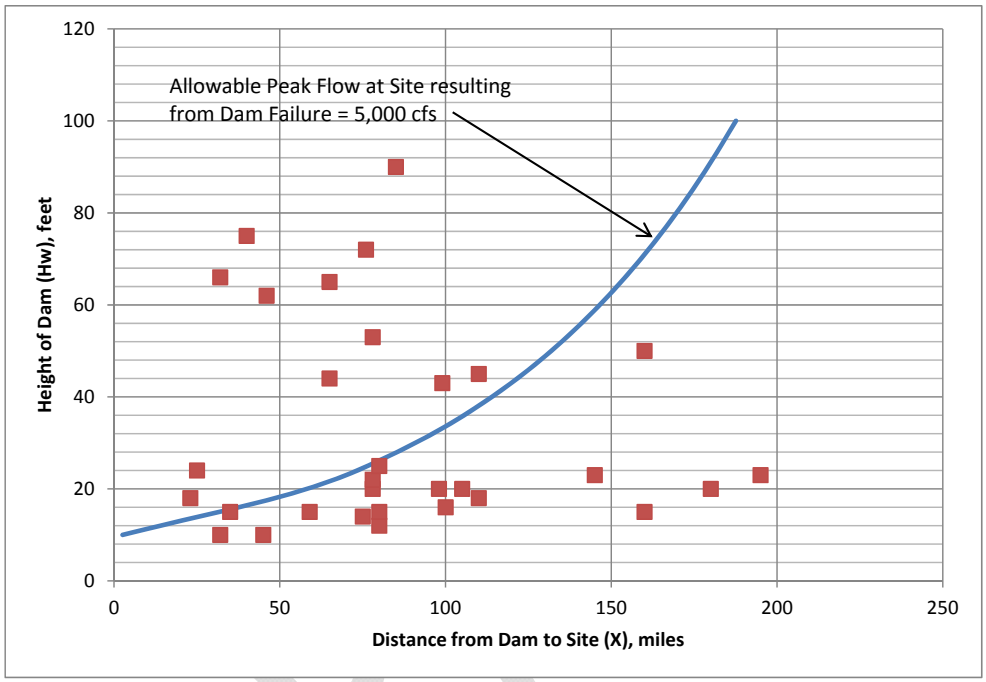

Figure 9 – Distance/Height of Dam Plot

Add example for multiple dams and zones.

Comment [g21]: Possibly move to an appendix.

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

APPENDIX C

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 ($\beta_{ln\text{-}PGA}$) of about 0.55.

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.

APPENDIX D

Sample of Information Requested from Dam Owners (Overall and Seismic)

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.

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