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**Subject:** Dam Failure Rev A2 08-02-12.docx  
**Date:** Thursday, August 02, 2012 4:23:28 PM  
**Attachments:** [Dam Failure Rev A2 08-02-12.docx](#)

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Chris, Ed;

Per my voice mail, please review the version attached to this message in place of the version attached to my previous message. This version addresses some additional task force comments.

Thanks and I apologize for any confusion.



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# POST-FUKUSHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1

## Supplemental Guidance for the Evaluation of Dam Failures

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## A. 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:

- 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:

- Hazard Reevaluation Report – Documents the results of the new evaluations for all relevant flooding mechanisms.
- Integrated Assessment Report – Documents corrective actions (completed and/or planned) for plants where the current design basis floods do not bound the reevaluated hazard for relevant mechanisms and the entire duration of the flood.

- **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 be 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).

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.

- **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 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 with appropriate jurisdiction for dams may be 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).

## B. 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 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:

- **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.
- **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 coincidentally 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.
- **Sunny Day Upstream Dam Failure:** A ‘sunny day’ dam failure is typically not associated or concurrent with a seismic event or an extreme flood and may occur due to failures of embankment material or foundation, such as those due to piping through the embankment. ‘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.

## C. Approach

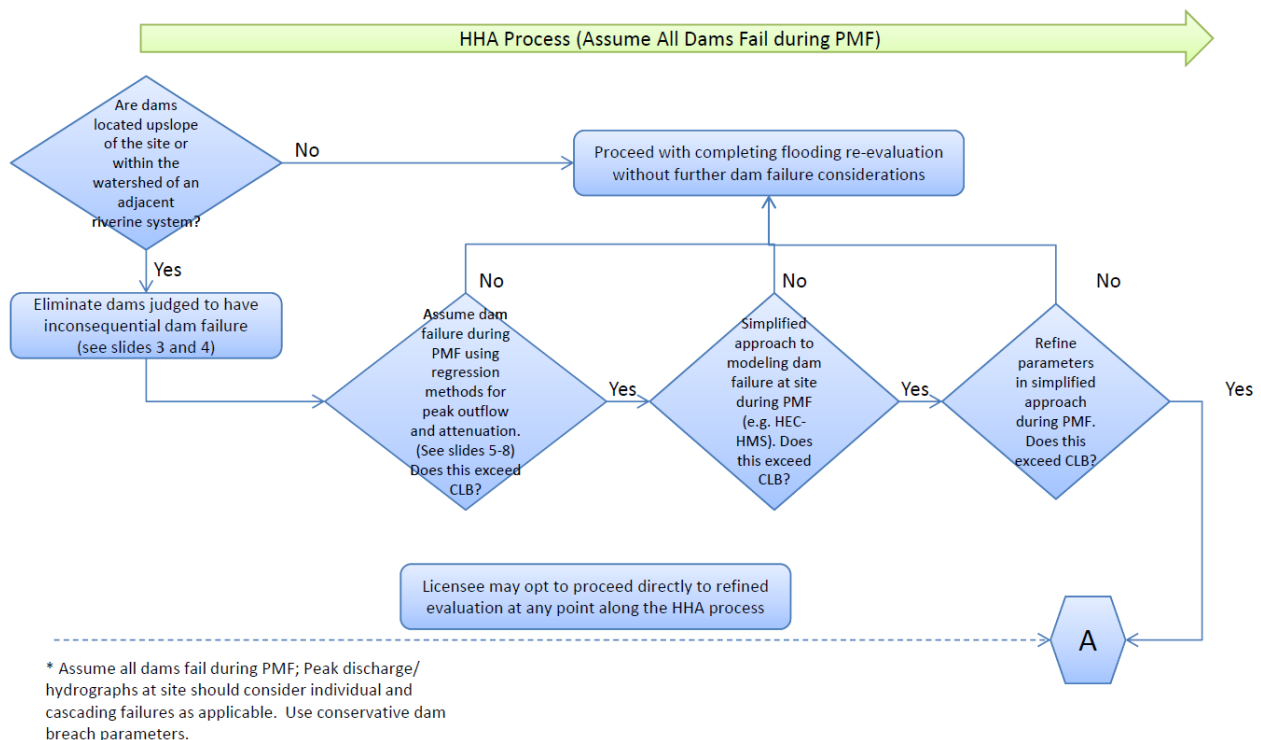
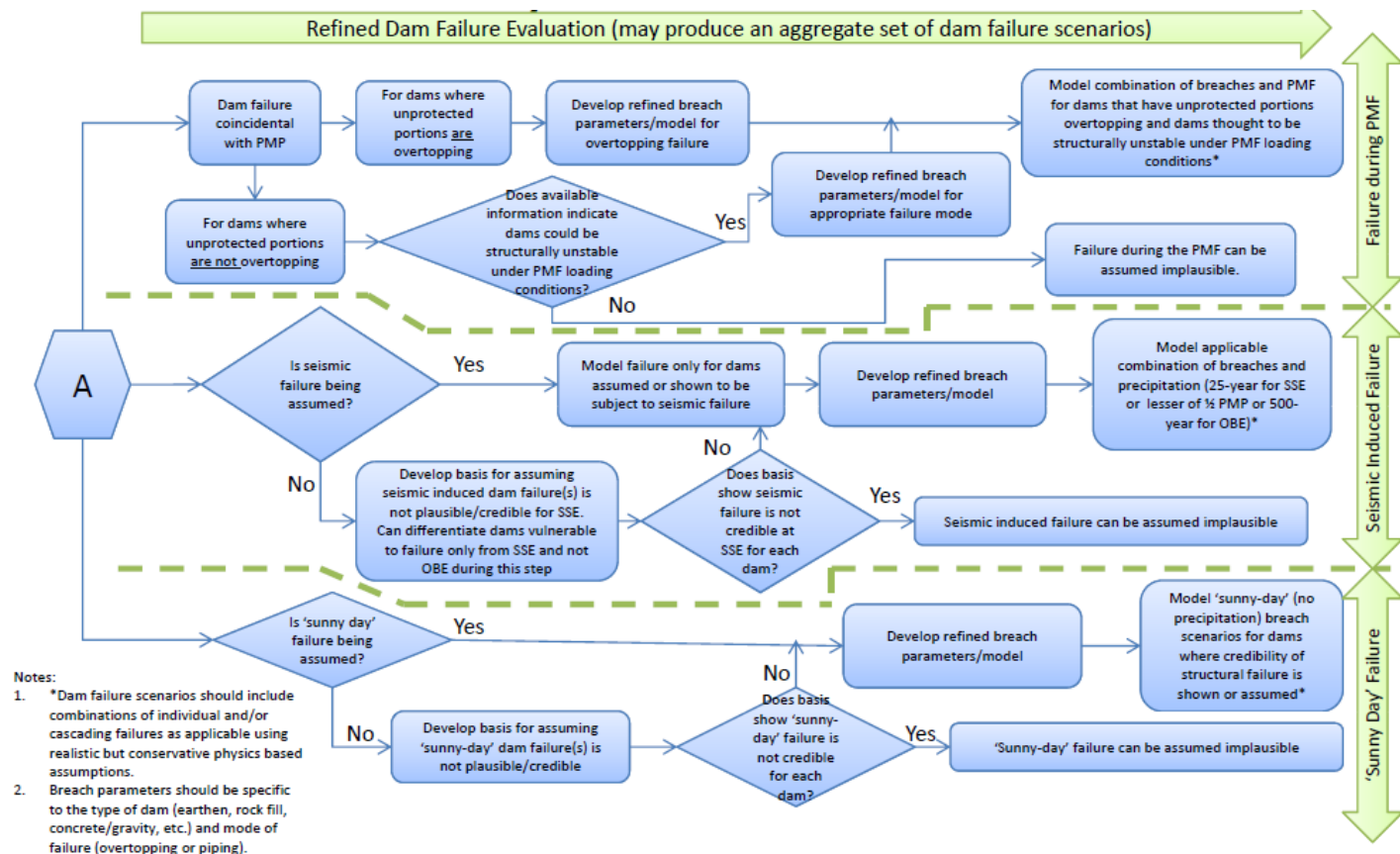


Figure 1 - Dam Failure Process/Decision Flow Chart



## C.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 (Decision/Process Flow Chart), the HHA approach to dam failure evaluations includes the following possible steps:

1. Eliminate dams judged to have inconsequential dam failure
2. Assume Failure during PMF (using Regression Peak Outflow and Attenuation Estimates)
3. Simplified Modeling of Dam Failure during PMF
4. Refined/Site-Specific Dam Failure Evaluation

### Eliminate Dams Judged to have Inconsequential Dam Failure

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, discussed further in the next section, can be used to develop a relationship between dam size and distance and establish thresholds for dams with inconsequential failure. For 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  $Q_r$  (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 1 for a range of allowable dam breach peak flow rates at the site, from 1,000 to 200,000 cfs. Figure 1 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 1 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 1, 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.



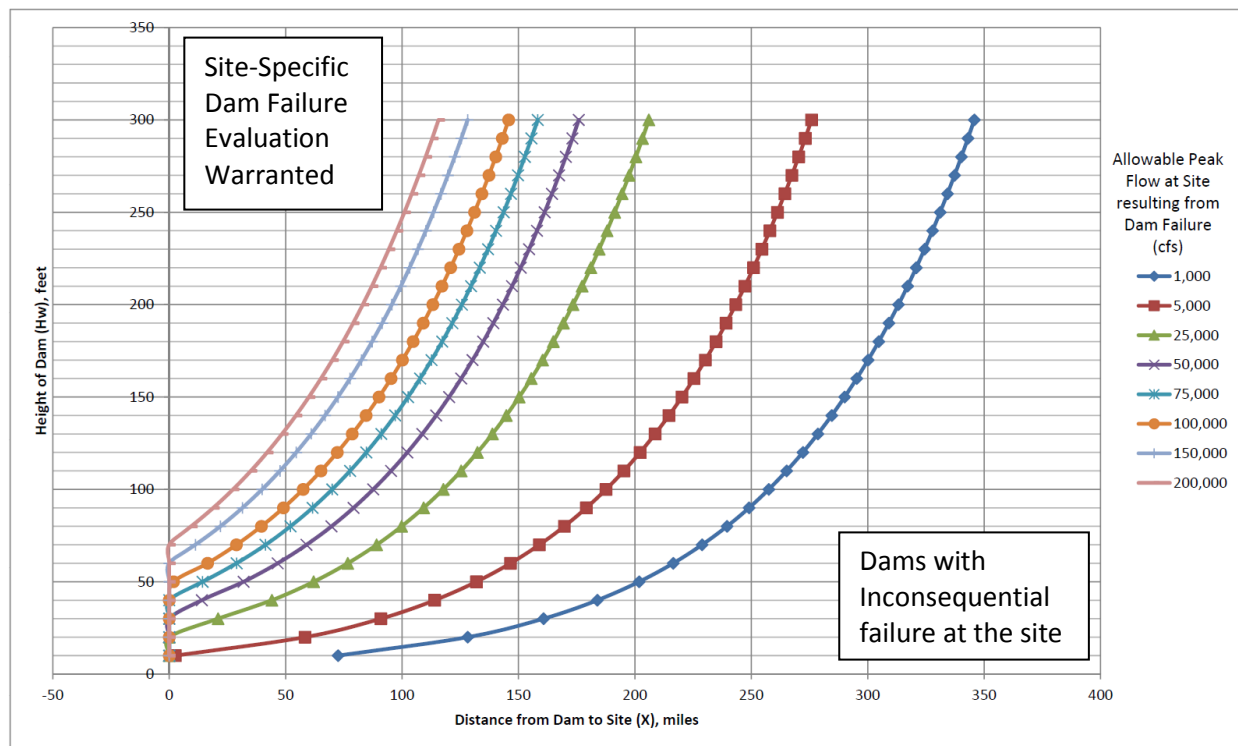


Figure 2 – Distance/Height of Dam Plot

### Assume Failure during PMF using Regression Peak Outflow and Attenuation Estimates

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 below. 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.

- 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

$C = 23.4 \times A_s / B_r$

$A_s$  = 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)

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)

### **Use Hydrologic Model as a Simplified Means of Evaluating Dam Failure/PMF Combination**

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    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 3- Dam Breach Menu Options in HEC-HMS

Additional information on developing breach parameters is provided in Section C.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.

Where:

$Q_{t=t_n}$  = 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)

## C.2 Refined Upstream Dam Failure Evaluation

### Breach Scenarios

#### *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.

#### *Overtopping Failure*

The overtopping 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.

#### *Seismically-Induced Failure*

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 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.

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 1/2 PMF, 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. 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 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  $1 \times 10^{-6}$  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.

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  $1 \times 10^{-6}$  or less.

### *'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. 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.

## Breach Formulation

### 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 1 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

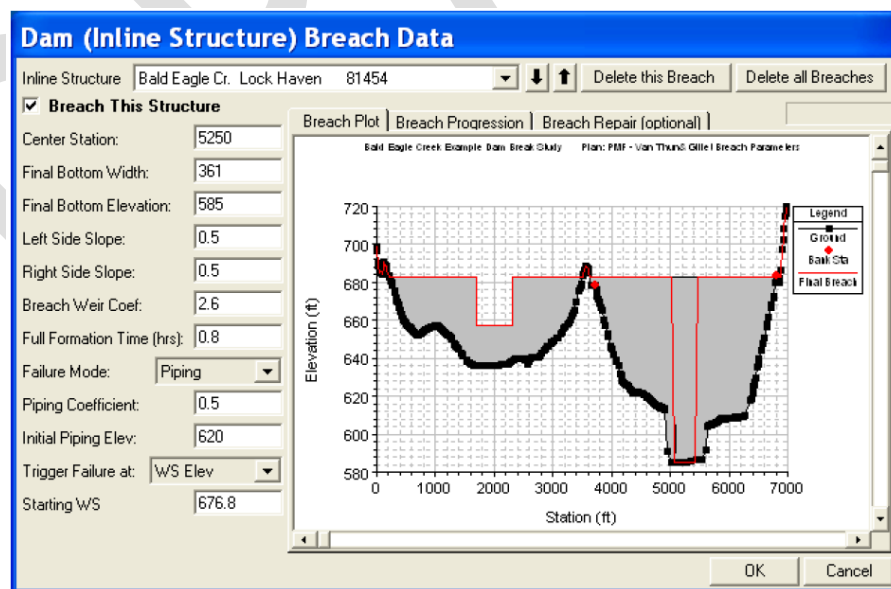
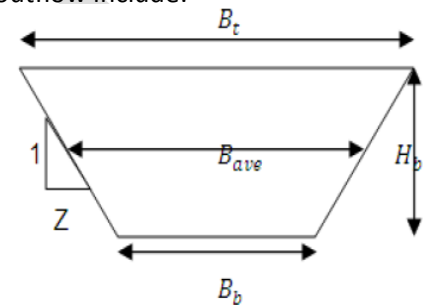


Figure 4 - HEC-RAS Dam Breach Editor



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 Froehlich (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.

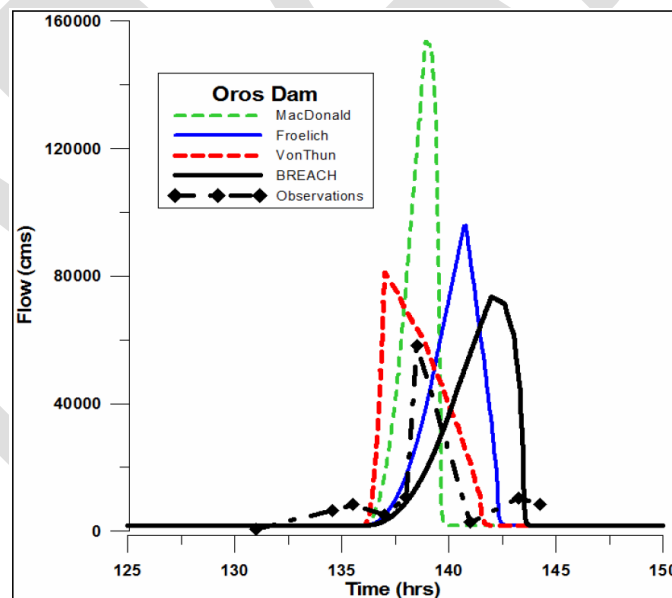


Figure 5 - Breach Hydrographs for Oros Dam (Gee, 2008)

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 ( $T_f$ )

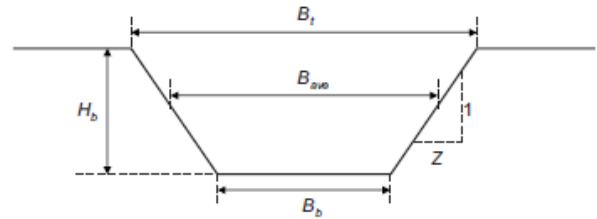


Figure 6 - Geometric Parameters of an Idealized Dam Breach (Xu and Zhang, 2009)

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		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	$X_{31} \quad X_{32} \quad X_{33}$
Peak outflow rate	$Y_4 = Q_p / \sqrt{g V_w^{5/3}}$	with corewalls	1 <sup>a</sup> (e <sup>b</sup> ) 0(1) 0(1)
Failure time	$Y_5 = T_f / T_r$	concrete faced	0(1) 1(e) 0(1)
		homogeneous/zoned-fill	0(1) 0(1) 1(e)
		Failure mode	$X_{41} \quad X_{42}$
		overtopping	1(e) 0(1)
		seepage erosion/piping	0(1) 1(e)
		Dam erodibility	$X_{51} \quad X_{52} \quad X_{53}$
		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: <sup>a</sup>Values for additive regression analysis;  
<sup>b</sup>Values for multiplicative regression analysis.  
 $H_r = 15$  m;  $T_r = 1$  hour.

Figure 7- 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 1 - 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.

**Table 2 - Predictions of Breach Parameters for the Banqiao Dam Failure (Xu and Zhang, 2009)**

Breaching parameter	Observed value	Best prediction models		Best-simplified prediction models	
		Mean prediction	95% prediction interval	Mean prediction	95% prediction interval
Breach depth (m)	29.5	25.2	19.9–30.6	25.3	20.2–30.3
Breach top width (m)	372	325	143–739	301	135–668
Breach average width (m)	291	273	116–643	262	114–601
Peak outflow (m <sup>3</sup> /s)	78,100	55,268	16,514–184,971	52,982	17,566–159,804
Failure time (h)	5.5	3.0	0.9–10.1	3.2	1.1–9.5

**Table 3 - Predictions of Breach Parameters for the Teton Dam Failure (Xu and Zhang, 2009)**

Breaching parameter	Observed value	Best prediction models		Best-simplified prediction models	
		Mean prediction	95% prediction interval	Mean prediction	95% prediction interval
Breach depth (m)	87	78	57–98	77	58–96
Breach top width (m)	238	198	87–451	181	82–402
Breach average width (m)	151	150	64–354	127	55–291
Peak outflow (m <sup>3</sup> /s)	65,120	75,220	22,475–251,745	55,146	18,283–166,333
Failure time (h)	4.0	4.1	1.2–13.6	3.2	1.1–9.5

### *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.

### ***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, Banquo).

The DSIG Project concluded that:

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

Table 4 provides a more comprehensive comparison of the findings.

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

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

## 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 3.) Nevertheless, the Xu and Zhang method appears to offer the least variability and seems to accommodate a wider range of situations.

**Table 5 - Comparison of Different Parameter Prediction Equations based on Case Studies in Xu and Zhang (2009)**

Parameter	Best prediction models in this paper			Best-simplified prediction models in this paper			Bureau of Reclamation (1982, 1988)			Froehlich (1995a,b)		
	Number of cases <sup>a</sup>	Mean	SD	Number of cases <sup>a</sup>	Mean	SD	Number of cases <sup>a</sup>	Mean	SD	Number of cases <sup>a</sup>	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

<sup>a</sup>The number of cases after using the objective outlier-exclusion algorithm (Rousseeuw 1998) at the 95% probability level.

## Breach Hydrograph Routing

### 1-Dimensional

Flood hydrograph routing in a 1-dimensional model is a procedure to determine the time and magnitude of flow passing through 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 5.

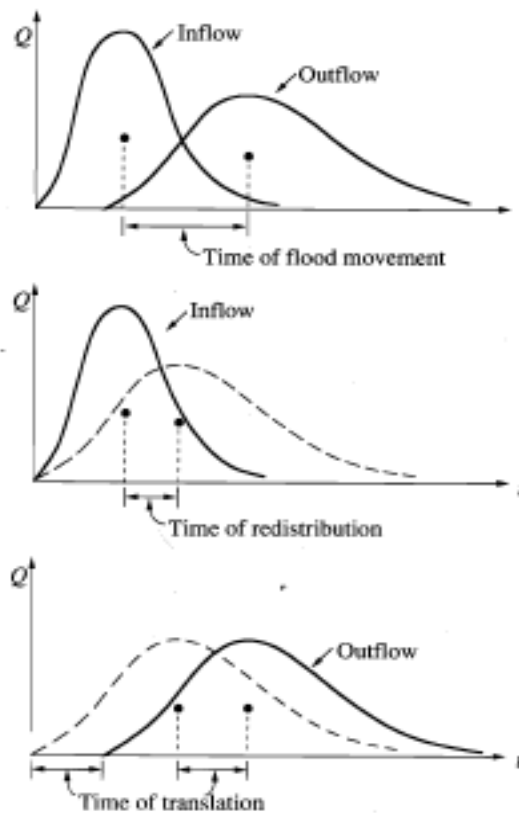


Figure 8 – Hydrograph Attenuation and Redistribution

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 8, 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 8, 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 8) 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.

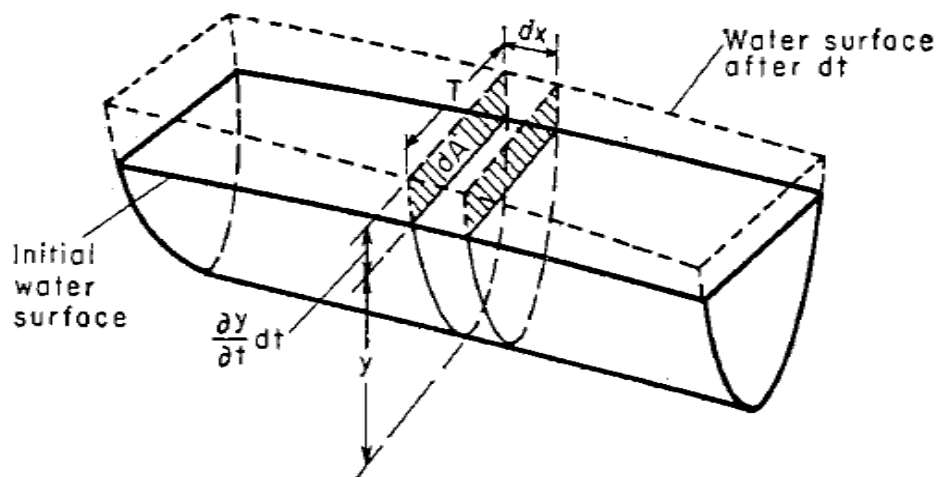
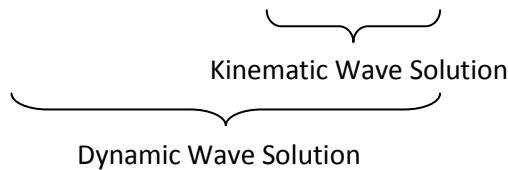


Figure 9 – Definition Sketch for St. Venant Equation

#### Equation 7 - St. Venant Equation

$$\underbrace{\frac{\partial}{\partial t} + \frac{\partial}{\partial x}}_{\text{Unsteady Varied}} \underbrace{\left( \frac{\partial y}{\partial t} + \frac{\partial}{\partial x} \right)}_{\text{Steady Varied}} \underbrace{\left( \frac{\partial}{\partial x} + \frac{\partial}{\partial x} \right)}_{\text{Steady Uniform}} = 0$$

Page 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

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