ENCLOSURE 1

Jocassee and Keowee Dams, Breach Parameter Review

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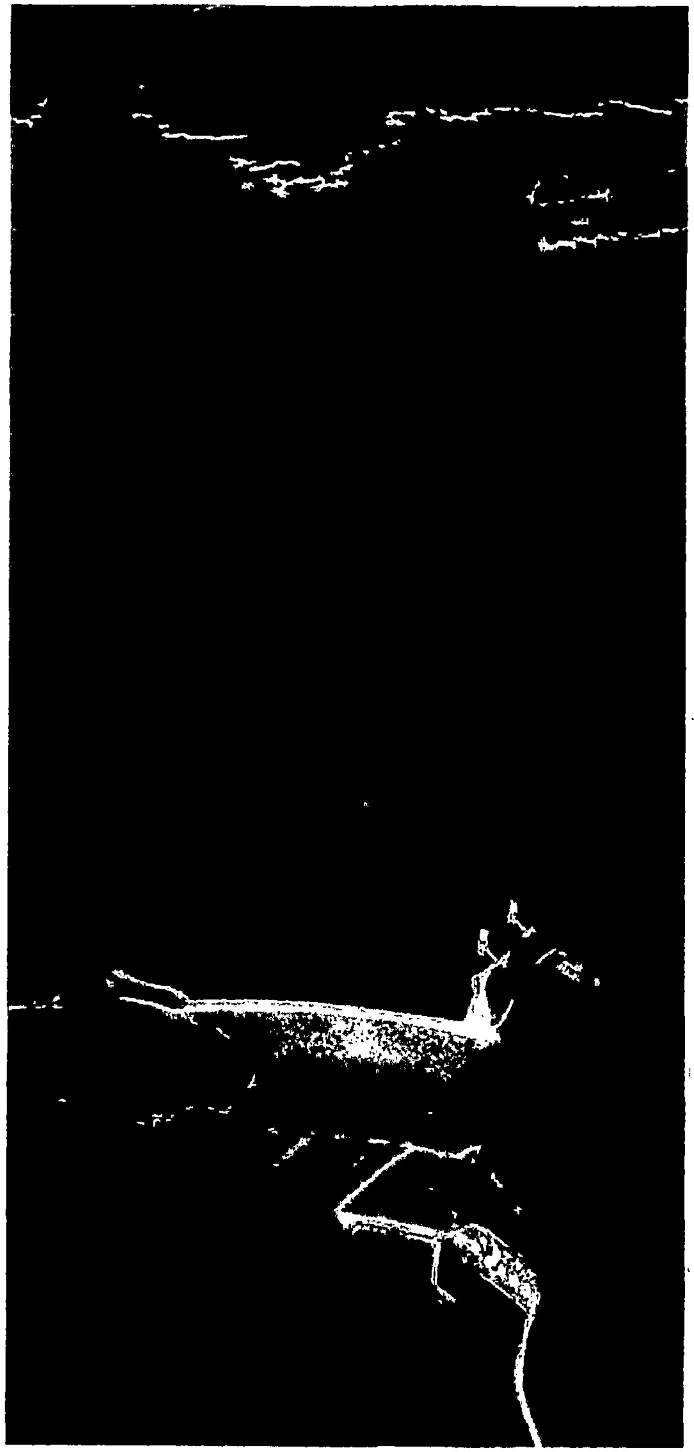
JOCASSEE AND KEOWEE DAMS



Breach Parameter Review

February 2013

By

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Jocassee Dam – Breach Parameter Independent Review

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1.0 Scope of the Independent Review

The scope and deliverable for this review was stated as follows in an email, dated December 11, 2012, from Mr. Dean Hubbard, Duke Energy, to Mr. Joseph Ehasz, URS Civil Construction & Mining.

> ***Scope of the independent review:*** *Provide an independent review of the postulated breach parameter assumptions for the Jocassee dam, and the Keowee reservoir dams and dikes to be used as inputs for routing of flood waters with HEC-RAS and SRH-2D. The independent review would determine if the breach parameters are realistic and appropriately consider the dam/dike designs, construction, materials and failure progression for the assumed failure mode. The Jocassee assumed failure mode is a piping failure and the Keowee reservoir dams and dikes assumed failure mode is overtopping. The breach parameters also need to be considered current state of knowledge and analytical methods (50.54(f) Letter requirement) and therefore defendable in an NRC review as being realistic but conservative.*
>
> ***Deliverable:*** *Independent initial review of the breach parameter assumptions for the Jocassee dam and Keowee dams and dikes with verbal report by conference call by 12/20/12. Independent review written summary with any recommendations for breach parameter adjustments to be submitted by January 15, 2013. The assessment should include a review of the validity of the model used (Xu Zhang 2009) and any other model that would provide more realistic breach parameters.*

Sections 2.0 – 6.0 focus on Jocassee Dam for an assumed piping failure mode and Section 7.0 contains a brief review of the failure of Keowee Dam as the result of upstream failure of Jocassee Dam.

Regarding the requirement that the breach parameters are to be *defendable in an NRC review as being realistic but conservative*, we cannot predict the outcome of an NRC review but we have based our review on our engineering expertise and specific knowledge of Jocassee Dam.

The following materials were provided by Duke Energy for this review:

1. *Final Breach Data Matrix Table* - containing a comparison of a) the breach parameters and related attributes for the "100 Wilson Case 2" and b) a range of breach parameter estimates and related attributes based on the Xu and Zhang (2009) regression method for Jocassee Dam.

2. *Xu Zhang Breach Data* – containing a range of breach parameter estimates and related attributes based on the Xu and Zhang (2009) regression method for Jocassee Dam.

3. *Jocassee Main Dam Design, Construction and Performance, Report to Design Engineering Department, Duke Power Company, Charlotte, North Carolina, by George F. Sowers, April 1987.*

4. Xu, Y. and Zhang, L.M., 2009, Breaching Parameters for Earth and Rockfill Dams, *Journal of Geotechnical and Geoenvironmental Engineering*, 135(12): 1957-1970.

5. An Excel Workbook *Jocassee BEP LE-ME Hydrographs Final* containing HEC-RAS breach hydrographs for Jocassee Dam.

6. An Excel Workbook *Final_Breach Data_Matrix – 02082013* containing revised final breach parameter estimates based on the Xu and Zhang (2009) regression method and a summary of HEC-RAS inputs for Jocassee Dam.

7. Video clips from HEC-RAS for Jocassee Dam breach with low and medium erodibility.

8. An Excel Workbook *Keowee Dam Breach Parameters* containing Keowee breach parameters.

9. An Excel Workbook *Keowee Breach Hydrograph – HDR BEP LE* containing HEC-RAS breach hydrographs for Keowee Dam.

10. Video clip from HEC-RAS for Keowee Dam breach.

In addition, we obtained a copy of the complete January 2010 Ph.D. Thesis, "Analysis of Dam Failures and Diagnosis of Distresses for Dam Rehabilitation", by Dr. Xu Yao from The Hong Kong University of Science and Technology.

2.0 Introduction

This independent review was performed on the Jocassee Dam postulated breach parameter estimates and underlying assumptions by Joseph L. Ehasz and Dr. David S. Bowles with assistance from Drs. Loren Anderson and Sanjay S. Chauhan. The review focused on the potential piping breach parameters for the Jocassee Dam based on the Xu and Zhang (2009) regression equations. The overtopping breach of Keowee Dam as the result of the failure of the upstream Jocassee Dam is discussed in Section 7.0.

Although review of potential dam breach parameters is the focus of this report, it is important to note that the physical features of the Jocassee Dam design and construction strongly influence the reality of any dam breach potential. Therefore, this review begins in Section 3.0 with a summary of some important physical features of the Jocassee Dam. We then evaluate if piping should initiate, would detection and successful intervention be likely to prevent a failure in Section 4.1, what piping failure modes might apply to Jocassee Dam in Section 4.2, and some general breach parameter considerations in Section 5.0. In Section 6.0 we evaluate, using the "state of knowledge," what are the appropriate breach parameters for Jocassee Dam? In Section 7.0 important features of Keowee Dam are summarized and the proposed breach parameters for Keowee Dam are reviewed. Conclusions are summarized in Section 8.0 and recommendations are listed in Section 9.0.

Appendix A contains descriptions of potential piping failure modes for Jocassee Dam. Appendix B contains breach parameters estimates from our application of the Xu and Zhang (2009) method to Jocassee Dam with a comparison to HDR mean estimates.

3.0 Background on Jocassee Dam

Even before considering the potential Jocassee Dam breaching and establishing the appropriate breach parameters it is instructive to discuss the design and construction of the Jocassee Dam. The following statements refer to the report entitled, "Jocassee Main Dam Design, Construction and Performance", by George F. Sowers, April 1987, and are significant to dam performance.

- The dam was completed in 1967 and has been continuously monitored and has performed very well to the present. Even though the dam was designed and constructed in the mid-1960's it has all the modern and defensive measures of the rockfill dams designed and constructed today. See Figures 1 and 2.

- Beginning at the rock foundation level, the (b)(7)(F) [redacted] (b)(7)(F) [redacted].

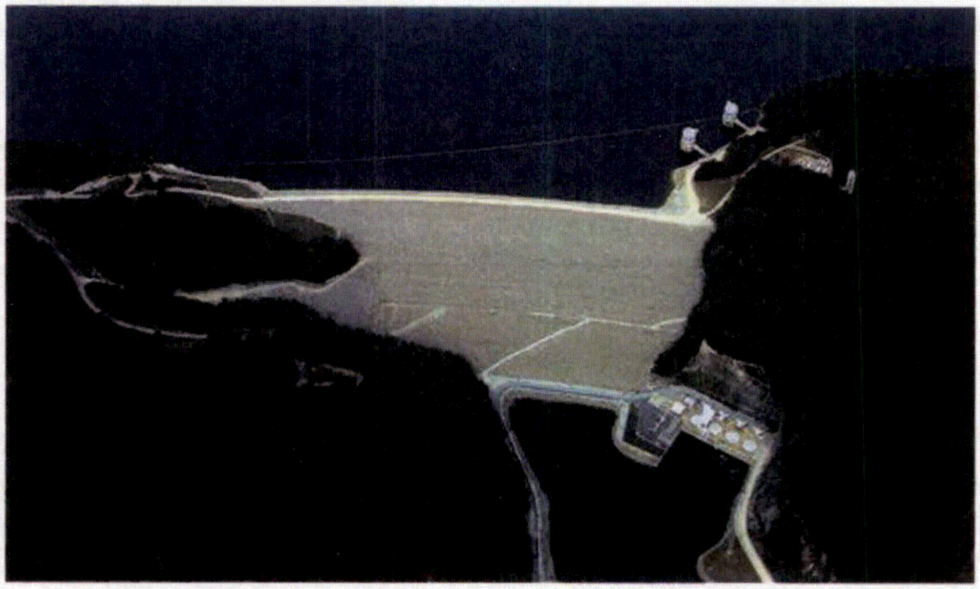


Figure 1. Google aerial view of Jocassee Dam.

(b)(7)(F)

Figure 2. Jocassee Dam cross section (Sowers 1987).

In addition, in order to decrease the hydraulic gradients as low as possible, the core zone was (b)(7)(F)

(b)(7)(F)

offset a reduction in horizontal stresses caused by deflection of the dam (b)(7)(F) [redacted] (b)(7)(F) [redacted] This ensures that the horizontal stresses (b)(7)(F) [redacted] along the foundation contact. See Figure 2.

- In order to protect the core materials from internal erosion and material movement and potential piping of the core (b)(7)(F) [redacted] (b)(7)(F) [redacted] (b)(7)(F) [redacted] See Figure 2.

- In order to establish a dense rockfill, (b)(7)(F) [redacted] (b)(7)(F) [redacted] This process produced a very strong and dense rockfill and a very stable rockfill embankment zone.

- Quality Control: selection of materials and documentation was enforced by the Resident Engineering staff and maintained as an important function during the construction. As indicated above, the selection and placement of the various zones were carefully done so as to maintain the compatibility of adjacent materials to eliminate migration of materials and maintain a stable embankment. Grain-size and density testing to verify materials and compaction were conducted at intervals to verify and maintain control of the placements. Field inspection was maintained during construction with Duke Engineering staff observing and interfacing to ensure the design intent was maintained during construction.

All of the above facts are important when considering the potential for internal erosion and piping at Jocassee Dam. The materials and features employed during design and construction, as described above, were all designed to minimize the possibility of piping and failure of the embankment. Thus, it is even difficult to envision the development of a piping condition at the Jocassee Dam, especially given the defensive design measures incorporated and with the past 45 years of excellent performance. However, a deterministic approach is being used to postulate a sunny day piping failure for Jocassee Dam. In Section 4.0, the potential for a piping failure of Jocassee Dam is discussed before discussing breach parameter considerations in Section 5.0 and reviewing the proposed breach parameter estimates using the Xu and Zhang (2009) method in Section 6.0.

4.0 Dam Failure Considerations

4.1 Piping Failure Mechanism

Given the description of Jocassee Dam design, construction and years of good performance in Section 3.0 it is very difficult to envision a "piping failure" through the main Jocassee Dam embankment. In addition, we are not aware that there has ever been a piping failure of a modern rockfill dam anywhere in the world.

For piping to occur, the following conditions must all exist:

1. there must be a source of water and a flow path,
2. there must be an unprotected exit for the eroded material,
3. there must be erodible material in the flow path, and
4. the material must support a roof for piping to enlarge and propagate.

The piping phenomenon within a dam must originate with internal erosion and material movement somewhere within the dam, perhaps at the foundation-core contact, and exit downstream and progress all along the foundation contact to daylight or to atmospheric conditions along the toe of the dam. The flow of water and materials must have an exit, to which it can carry materials and then progress upstream along some erosion path and move materials from upstream; thus, the formation of the "piping phenomenon". With increasing movement of large amounts of water carrying materials, the flow would eventually form a path within the downstream rockfill portion of the dam. If the process continues and the flows increase, it would eventually engage a larger and larger portion of the rockfill and eventually form a raveling condition within the rockfill large enough to make the downstream shell of the dam unstable. If it becomes unstable enough to move rockfill downstream or to cause pockets through collapses, it could then expose the core, which is the water retaining structure of the dam for the upstream reservoir. If support is removed from the downstream side of the core, it would become unstable and partially collapse. The breach of the dam would progress as it overtops the core and forms an overtopping breach and failure of the dam. This is considered the start of the "breach development phase" as discussed in Section 5.0.

The defensive measures within the Jocassee Dam are designed, constructed and incorporated to essentially eliminate the piping process that is described above. (b)(7)(F) [redacted] (b)(7)(F) [redacted] thus minimizing the erosion potential. To further defend against erosion and material movement, filters and a drain were specifically designed to eliminate material particle movement from one material in the embankment section through another and progressing downstream. Specifically, the filter gradation was designed to eliminate core material particles from moving through the filter and the gradation of the drain materials was designed to eliminate filter materials from moving through the drain. In addition, the construction specifications were detailed such that (b)(7)(F) [redacted] All of these measures essentially eliminate the potential for migration of materials and internal erosion of the dam materials; and hence, prevent a piping condition from forming. Thus, an embankment piping failure of the Jocassee Dam is extremely unlikely, either deterministically or probabilistically.

However, if one hypothesizes that "piping develops along some hypothetical path," then one can ask, "What would be the conditions of a failure?" Given the above defensive measures, piping would be a very slow and detectable process; with very likely enough time for intervention. In the event of piping initiation, emergency action could be taken by (b)(7)(F) [redacted] The condition would be observed and immediately considered unusual and serious enough to be studied, evaluated, and appropriate emergency action taken.

Given the fact that the (b)(7)(F) [redacted] especially along the downstream portion, the stability of the downstream shell will be maintained even with considerable flow through the rockfill. Experience shows that rockfill can support considerable flows and still maintain stability. A study by Leps (1973), entitled "Flow Through Rockfill," has presented several cases of actual high flows through rockfills. At Hell Hole Dam in California over 20,000 cfs passed over and through a rockfill embankment dam without distress. In addition to simply passing the flow, it is essentially impossible to form a "pipe or tunnel" within the rockfill without seeing it, since the structure would collapse and block any formation of a "pipe". The flow may cause a raveling or movement and adjustment of the larger rock materials; however, it would not wash out the rockfill completely and expose the core very rapidly. Thus, it would be a long process and provide time to intervene by lowering the upstream water levels and reducing the reservoir storage. Once again, even in the hypothetical case of piping in a rockfill dam, there would very likely be time to intervene, significantly reduce the chance of dam failure, and reduce the outflow discharge and effects on the downstream flooding conditions.

4.2 Potential Piping Failure Modes for Jocassee Dam

The information and discussion in Section 4.1 relates to a piping failure of the main or general sections of the Jocassee Dam embankment that would involve gross movements and complete failure of the dam. Earlier Potential Failure Modes Analyses (PFMAs) have investigated other sections of the dam and abutments for alternate potential piping conditions. In a PFMA, which was facilitated by RAC Engineers & Economists and included engineers from Duke Hydro and HDR, all 28 initiating mechanisms included in the Piping and Seepage Toolbox (Reclamation-USACE-URS-UNSW 2008) were considered for their applicability to Jocassee Dam. In addition, the PFMA reviewed potential failure modes identified and categorized in a PFMA conducted by Findlay Engineering (2004) for submission to the FERC. The following three piping failure modes were identified and categorized as being credible for the Jocassee main embankment dam in the RAC PFMA:

1. JPFM 5: Piping in the [redacted] area at approximately Elevation 1020 ft. msl:
    a. JPFM 5a1: Piping through the foundation (natural dam) in the [redacted]
    b. JPFM 5a2: Piping through the foundation [redacted] near the [redacted]
    c. JPFM 5b: Piping through the foundation (Natural Dam) at the interface between the weathered rock in the abutment and the core material.
2. JPFM 4: Piping through the rock foundation in the [redacted]
3. JPFM 3: Piping through embankment.

Each of these failure modes is described in the Appendix A.

The most likely ("least unlikely") dam piping failure mode in the [redacted] is piping through the [redacted] (JPFM 5a1). [redacted] piping through the [redacted] of the main dam (JPFM 4) is a much more unlikely piping failure mode and main dam embankment piping is extremely unlikely for the reasons described in the Sections 3.0 and 4.1 of this report.

The normal maximum operating level for the Jocassee Reservoir is Elevation 1,110 ft. msl. The reservoir operating rule is designed to keep the reservoir at or below this level for all but relatively rare floods. Historically the reservoir has never exceeded Elevation 1,110 ft. msl, although the annual peak reservoir level has been within a foot of that elevation in many years during its 45-year life. It is estimated that for the Jocassee reservoir pool to exceed Elevation 1,110 ft. msl. would require a major flood of the order of the 1 in 200 AEP or rarer.

5.0 Breach Parameter Considerations

When considering the piping failure of a dam it should be recognized that the seepage/erosion/piping process, which is the first phase (breach initiation) of the breach process (Wahl 1998), takes considerable time, as discussed in Section 4.1, especially if the embankment is well protected as described for Jocassee Dam in Section 3.0. However, the breach failure times that are the focus of this review are the second phase (breach development) or the time that the embankment would take to washout after the internal erosion process (piping) advances far enough to form a cavern within the downstream shell leading to a collapse of a portion of the downstream shell that exposes and results in a partial and progressive collapse of the core, exposing it to overtopping. The overtopping would then washout the embankment. Therefore, there would be expected to be considerable time available in the early phases of a piping process (phase one, or breach initiation) to intervene and take action, especially with the defensive measures that are designed into Jocassee Dam.

Figure 3 is adapted from the Xu and Zhang (2009) paper and illustrates the distinction between the two phases of the breaching process as defined by Xu and Zhang (2009): breach initiation and breach development. Outflow during the breach initiation phase is small, and in the case of a piping failure it is a flow through a developing pipe or seepage channel. In the breach development phase, the outflow and erosion develop more rapidly especially after initial collapse of the core.

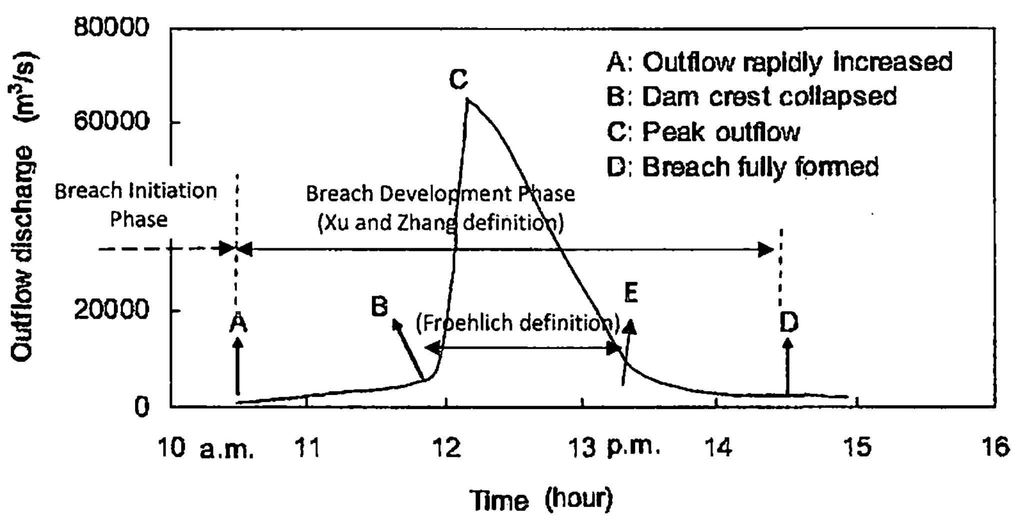


Figure 3. Definition of breach initiation and breach development phases in Xu and Zhang (2009) shown for observed discharge during the failure of

Teton Dam [Adapted from Xu and Zhang (2009)].

Figure 3 shows an example of the outflow hydrograph during the piping failure of Teton Dam in 1976. The time of 10:30 a.m. is a critical point that separates the breach initiation phase (ending at point A) from the breach development phase (from point A to point D). After 10:30 a.m. (point A) the rates of discharge and erosion of embankment materials from the pipe increased more rapidly. After the collapse of the dam crest at about 11:55 a.m. (point B), the breach developed rapidly due to overtopping of the collapsed dam crest and soon reached a peak discharge at about 12:15 a.m. (point C). The failure time associated with the breach development process, T_f, was approximately 4 hours.

Fell et al (2003), point out that it has not been possible to identify the time of initiation of internal erosion associated with piping, and so it has not been possible to estimate breach initiation phase times for historical dam failures. However, as indicated in Figure 3, it is the breach development time, T_f, between points A and D that is the predicted "failure time" in the Xu and Zhang (2009) method. However, the Xu and Zhang (2009) definition of failure time differs to that used in some other failure-time prediction equations, such as Froehlich (1995), in which it is defined as the period of higher outflow between points B and E or about 1.25 hours as indicated in Figure 3. The importance of this for our review is not that there are differences between the way that failure time is defined in different breach parameter prediction methods, but that there should be a consistency between the way that failure time is defined in the method that is used to estimate it and the way that it is applied in a breach model, which for Jocassee is the HEC-RAS model. This consistency is discussed in Section 6.4 as it applies to the Jocassee Dam.

Duke Energy and its consultant HDR Engineering have selected the dam breach methodology as proposed by Y. Xu and L. M. Zhang in their technical paper entitled "Breaching Parameters for Earth and Rockfill Dams" as published in the Journal of Geotechnical and Geoenvironmental Engineering, December 2009. This is a "state of knowledge" paper, especially for consideration of a piping breach in a zoned dam. The main objective of the paper is to develop robust empirical formulas with physical meaning for predicting dam breaching parameters.

6.0 Xu and Zhang (2009) Regression Model

6.1 Control Variables

The Xu and Zhang regression equations are based on an analysis that includes more recent failures than are included in earlier relationships, such as Froehlich (1995), and also include data from China that have not been previously available. Data from a total of 75 earth and rockfill dam failure cases were used to develop multiple regression equations to predict the following five breach parameters (dependent variables), which are divided into two groups, breach geometry and hydrograph:

- Breach Geometry
    - Breach depth (H_b)
    - Breach top width (B_t)
    - Average breach width (B_{ave})
- Hydrograph
    - Peak outflow rate (Q_p)
    - Breach development time or failure time (T_f)

The breach side slope, z, can be calculated from the breach depth (H_b), breach top width (B_t) and average breach width (B_{ave}).

Unlike previous relationships, Xu and Zhang included erodibility as a control (independent) variable and they found it to be the single most important of all the control variables that they considered. The complete list of five control variables that they used to estimate breach parameters is as follows:

- Dam height *($X_1 = H_d/H_r$, dam height H_d and a reference height H_r, where $H_r = 15$ m),*
- Reservoir shape coefficient *($X_2 = V_w^{1/3}/H_w$, volume of water above breach invert Vw and depth of water above the breach invert at the time of failure, Hw.),*
- Dam type *(with corewalls, concrete faced and homogeneous/zoned-fill),*
- Failure mode *(overtopping and seepage erosion/piping),*
- Dam erodibility *(high, medium and low).*

Xu and Zhang (2009) included rockfill dams in the homogeneous/zoned-fill dam type. Dam type, failure mode and dam erodibility are included in the regression analysis as virtual discrete variables that represent either the presence or absence of an attribute.

As mentioned above, Xu and Zhang (2009) found dam erodibility to be the most important control variable for predicting all five breach parameters. Dam erodibility is described as *a relative measure of erodibility based on the information on material compositions and compaction conditions, dam cross-sectional geometry, construction time, and other relevant pieces of construction information*. The three erodibility classes (low, medium or high) used in the Xu and Zhang equations refer to the technical lecture paper by Briaud (2008), whereby soils

and rocks are classified into various erosion resistance categories based on erosion velocity or shear stress, as shown in Figures 4a and b, respectively.
As summarized in the background discussion in Section 3.0, the Jocassee Dam is a modern designed and constructed dam with (b)(7)(F) [redacted] (b)(7)(F) [redacted] core and filters.

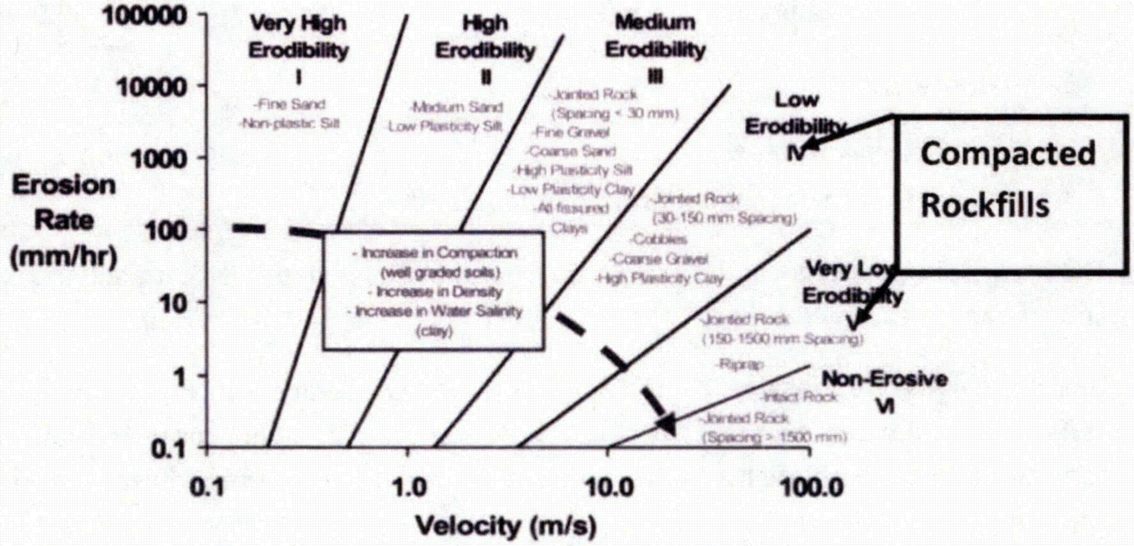


Figure 4a. Briaud (2008) erosion categories for soils and rocks based on velocity.

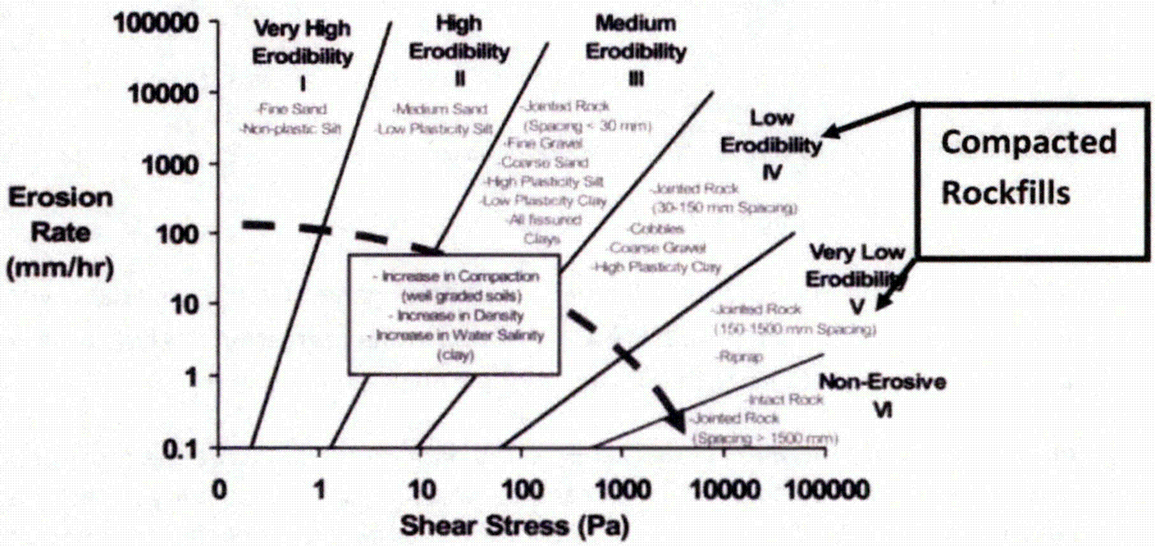


Figure 4b. Briaud (2008) erosion categories for soils and rocks based on shear stress.

zone. These considerations would conservatively place the Jocassee Dam embankment in the low erodibility category (IV) and likely in the very low erodibility category (V).

6.2 Prediction Equations and Confidence Intervals

Xu and Zhang (2009) developed two types of multiple regression equations for estimating mean values of the five breach parameters:

- Best Exact Prediction - based only on failure cases where all five control variables were available [see Table 4 in Xu and Zhang (2009) for numbers of failure cases for each breach parameter].
- Best Simplified Prediction – includes additional failure cases where not all five control variables were available [see Table 5 in Xu and Zhang (2009) for numbers of failure cases for each breach parameter].

Xu and Zhang (2009) present their final best exact and best simplified equations for all five breach parameters in Tables 4 and 5 of their paper, respectively. These tables also include the standard error of the regression, which can be used to calculate confidence intervals for breach parameter estimates in addition to mean predictions that are obtained directly from the regression equations. Xu and Zhang (2009) conducted a comparison with other breach prediction methods and demonstrated that their method provides a lower bias and standard error on predictions than other methods. They admit that since this comparison used data on which their model is based it may have had an advantage over other models included in the comparison, but they nevertheless conclude that the comparison is valid.

Xu and Zhang (2009) included two applications of their equations to actual dam failures for Banqiao and Teton dams. Their mean predictions are compared with the observed values of the breach parameters for both the best exact and the best simplified models in Tables 7 and 8 of their paper, respectively. In addition to the mean estimates, they include lower and upper bound estimates based on a 95% confidence interval. The meaning of such a confidence interval is that there is a 95% chance that the true values of the breach parameters are contained in the range between the lower and upper bound values. The lower and upper bound values for an additive regression model are equally spaced on either side of the mean estimate and this can be seen, within the limits of round off in the estimates, for the estimates of the breach depths given in Tables 7 and 8 of the paper. However, the spacing between the mean estimates and the lower and upper bounds for the remaining four breach parameters is very asymmetric. Specifically there are approximately two to three and a half fold differences between the mean and lower bound estimates and between the upper bound and mean estimates for all breach parameters except height of breach. Clearly the widths of the 95%

confidence intervals are large for all breach parameters and this is further discussed in the following paragraph. We return to this topic of the confidence intervals on the Xu and Zhang (2009) breach parameter estimates in Section 6.3 where we discuss how it applies specifically to the Jocassee breach parameter estimates that are the focus of this review.

The reason for the asymmetric confidence intervals relative to the mean estimates for all breach parameter predictions, other than breach depth, is that the nonlinear or multiplicative form of the regression equations that Xu and Zhang (2009) found best fit the historical breach data. To develop the regression models these multiplicative relationships were converted to an additive form, following the usual practice in regression analysis (Haan 1977), by applying a logarithmic transformation. To obtain confidence intervals, an assumption is made that the variability in the breach parameters that is not accounted for or explained by the variation in the values of the control variables is distributed according to a normal or Gaussian (bell curve) probability distribution. This unexplained variability is associated with all the breach parameter values in the data set of historical dam failure cases that were used to develop each regression equation [see Tables 4 and 5 in Xu and Zhang (2009)]. When the multiplicative form of regression equation is used, the assumption that the unexplained variability in the values of the historical breach parameters is normally distributed applies in the transformed or log-space. Thus, when the confidence intervals are transformed to their natural space using an inverse logarithmic transformation, they are significantly asymmetrical with the range between the mean and the upper bound typically much greater than the range between the mean and the lower bound.

6.3 Implementation of Xu and Zhang (2009) Method

We applied the Xu and Zhang (2009) regression equations to Jocassee Dam to obtain the mean estimates of the breach parameters and to compare them with the HDR estimates. We also estimated the confidence intervals on the estimated breach parameters using equations for the confidence intervals on a multiple regression that can be found in many textbooks (e.g. Haan 1977). To verify our spreadsheet for applying the Xu and Zhang regression equations we first reproduced, within a small round-off error, the mean and confidence interval estimates that are presented in the Xu and Zhang (2009) paper for Teton and Banquito dams. We then unsuccessfully attempted to reproduce the HDR mean estimates but after some additional checking the HDR estimates were revised. These final HDR estimates now closely match our estimates as is shown later in this section.

As discussed earlier, the Xu and Zhang equations incorporated the various erosion resistance categories (low, medium or high) based on the technical paper by Briaud (2008). Our interpretation, based on the rockfill materials, compaction and density, places the rockfill at Jocassee Dam between the very low and low erodibility categories (see Figures 4a and b).

Therefore, we recommend that the most appropriate and conservative breach parameters for Jocassee Dam should be based on using the low erodibility values.

Tables 1 and 2 show the input values and the breach parameter estimates from our application of Xu and Zhang (2009) to Jocassee Dam. Separate breach parameter estimates are included for the Best Exact and Best Simplified Prediction regression equations. Table 2 also includes a comparison with the final HDR mean estimates (in blue font) for low erodibility. A more detailed version of Table 2, which includes the values calculated for all regression parameters and references the regression equation numbers for the equations that we used from the Xu and Zhang (2009) paper, is contained in Appendix B.

Table 1. Input values used for Xu and Zhang (2009) application to Jocassee Dam.

Input	Elevation	Hw	Reservoir Volume
	ft msl	ft	acre-feet
Crest Elevation	1125		
Reservoir Elev	1110		
Breach bottom elevation			
Low erodibility	870	240	1,091,716
Dam Height	Hd =	385	ft
	Hr =	15	m
Dam Type	Homogeneous/zoned-fill		
Failure Mode	Seepage erosion/Piping		

Table 2. Breach parameters estimates from Xu and Zhang (2009) application to Jocassee Dam with comparison to HDR mean estimates.

Breach Parameter	Erodibility Category	Best Exact Prediction				Best Simplified Prediction			
		Mean	Mean	95% Confidence Interval		Mean	Mean	95% Confidence Interval	
		HDR*		Lower	Upper	HDR		Lower	Upper
Height of breach, Hb ft.	Low	255	253	168	337	255	255	176	334
Failure time, Tf hrs.	Low	(b)(7)(F)							
Breach top width, Bt ft.	Low	701	701	306	1,606	666	666	299	1,485
Average breach width, Bave ft.	Low	566	566	246	1,306	515	515	223	1,193
Peak outflow, Qp ft3/sec.	Low	(b)(7)(F)							

* Note that HDR used the Best Simplified Prediction for Height of Breach instead of the Best Exact Prediction

One reason for the small differences between the HDR estimates and our estimates shown in Table 2 is that HDR used the Best Simplified Prediction equation for height of the breach, H_b, in both the Best Exact and Best Simplified Predictions for the other breach parameters. We used the separate Best Exact and Best Simplified Predictions of H_b for the corresponding Best Exact and Best Simplified Predictions, respectively, for the other breach parameters. HDR's approach was based on the higher R^2 value obtained for the Best Simplified Prediction equation for H_b, whereas our approach followed the examples for Banqiao and Teton dams in the Xu and Zhang (2009) paper.

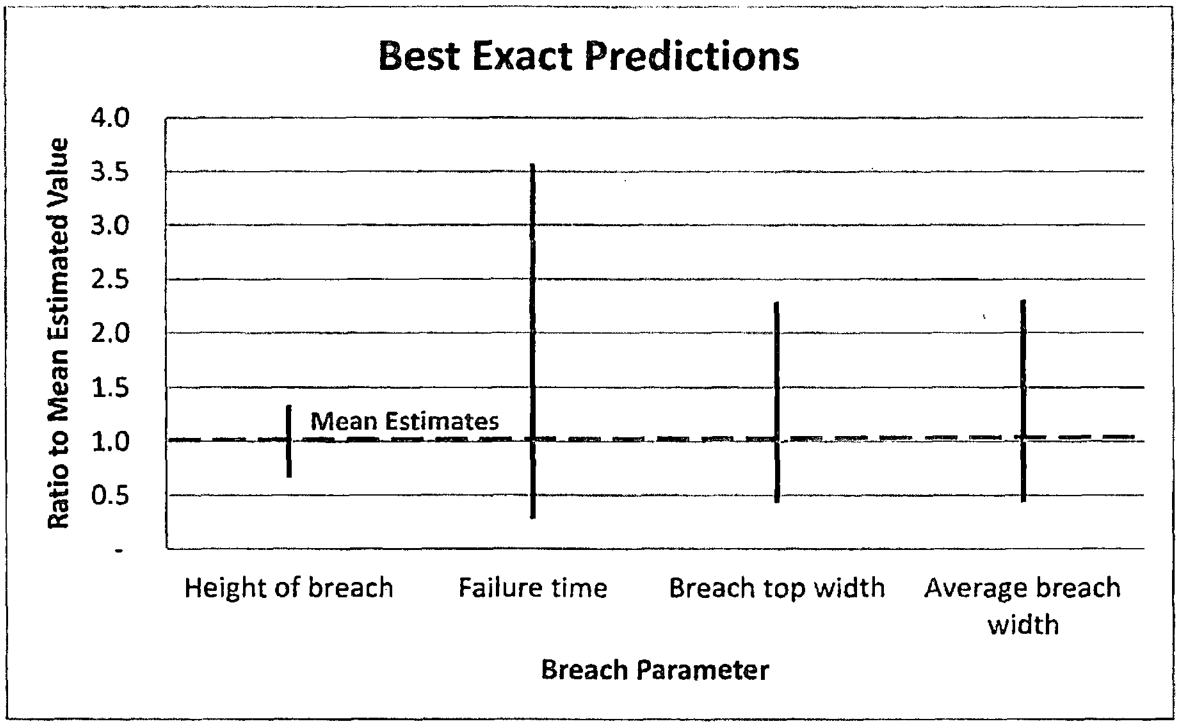
From Table 2 it can be seen that using low erodibility in the Best Exact Prediction equations, the mean failure time is estimated to be abou (b)(7)(F) the mean breach top width about 700 feet, the mean average breach width about 570 feet, the mean height of breach about 250 feet, and the mean peak discharge unde (b)(7)(F) cfs.

The confidence interval estimates for each breach parameter are shown in Table 2 as lower and upper bound estimates that define the width of the confidence interval both below and above the mean estimates. Similar to the examples for Banqiao and Teton dams, which we discuss in Section 6.2, the asymmetry in the confidence intervals can be seen for all breach parameters except height of the breach, which is symmetrical because of the additive form of the regression equations for height of the breach as explained in Section 6.2. The symmetry in breach parameter estimates for height of breach and the asymmetry in estimates for failure time, breach top width, and average breach width can be seen in Figures 5a and b. In these figures the confidence intervals for breach parameter estimates for Jocassee Dam are plotted as a ratio to the mean estimate for the Xu and Zhang (2009) Best Exact and Best Simplified Prediction equations, respectively. The asymmetry in the confidence intervals for failure time, breach top width, and average breach width can be seen by the longer lines above the mean estimate (plotted at a ratio = 1.0 shown by the blue dashed line) than below the mean estimate.

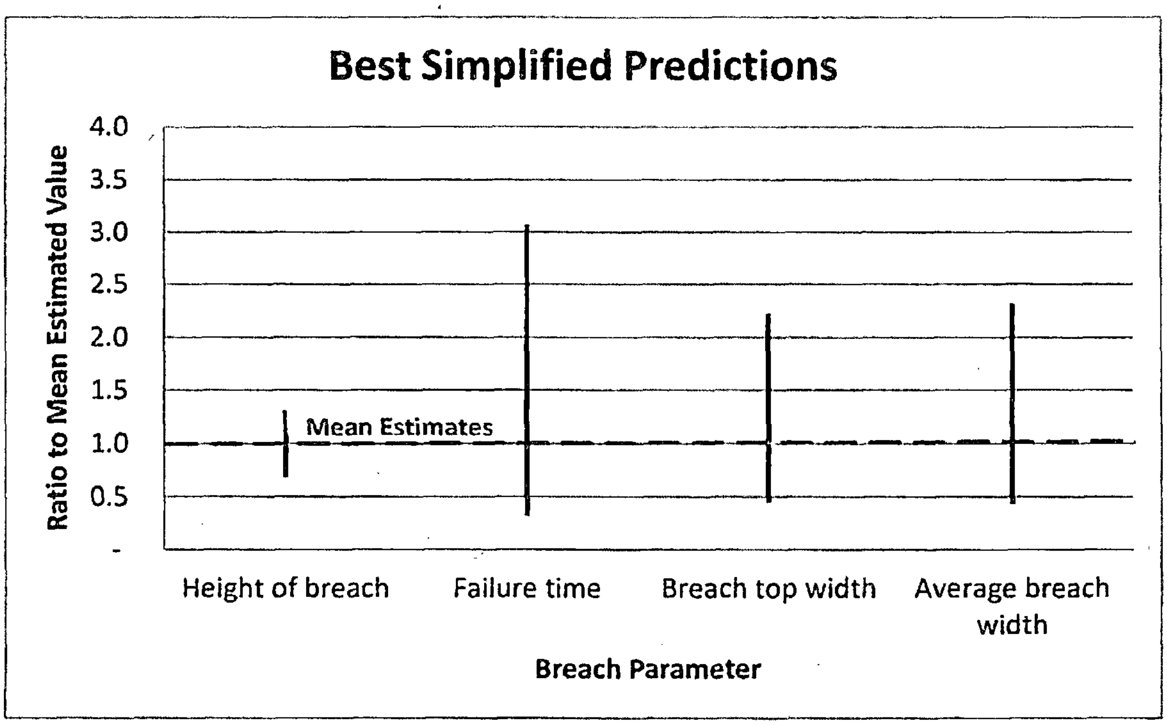
In Table 2 we have placed boxes around the values of the breach parameters that we recommend as being most applicable to Jocassee Dam. These are mean estimates based on low erodibility. The basis for using low erodibility is explained earlier in this section and is based on discussions in Sections 3.0 and 6.1. The basis for using mean estimates is discussed here.

In Section 6.2, we described Xu and Zhang (2009) mean and confidence interval breach parameter estimates for each breach parameter as follows:

- **Mean estimate**: Based on applying a regression equation to represent or "explain" the fraction R^2 of the variability in the observed breach parameter values in the data set of historical dam failures in terms of the variation in the values of the observed values control variables that were used to derive the regression equation.



a) Best exact predictions



b) Best simplified predictions

Figure 5. Relative width of confidence intervals for Xu and Zhang (2009) breach parameter estimates for Jocassee Dam based on low erodibility expressed as a ratio to the mean estimate (ratio = 1.0).

- **Lower and upper bound estimates** (Confidence interval): Based on the variability between observed breach parameter values in the data set of historical dam failures that is not represented or "unexplained" by the regression equation for the mean value. This unexplained variability is the fraction $(1 - R^2)$ of the variability in the observed breach parameter values in the data set of historical dam failures.

Graphically one can picture the unexplained variability in breach parameter estimates as being a scatter of points about the regression line representing the actual breach parameter values.

A key question is, "Where would one expect Jocassee Dam to fit in the range of the scatter or unexplained variation of breach parameter values for the data set of historical dam failures used by Xu and Zhang (2009)?" <u>Based on the fact that Jocassee Dam is a well designed and constructed rockfill dam that has incorporated modern design criteria and defensive design features (see Section 3.0) the breach geometry parameter estimates are expected to be in the range below the mean but above the lower bound confidence interval estimates.</u> Additional factors that support this range of geometric breach parameter estimates include the following:

- **Erodibility**: Based on the discussion in Section 6.1, Jocassee Dam would be expected to be in the lower part of the range of low erodibility as defined in the Xu and Zhang (2009) implementation of the Briaud (2008) erodibility classification system.

- **Uni-directional breach formation**: A breach from the very unlikely piping failure modes of piping through the foundation in the [redacted] would start [redacted] the dam and can only progress [redacted] of the dam and [redacted] in contrast to developing in two directions for most historical breaches. This would be expected to reduce the width of the breach because of the more erosion resistant and stabl[redacted] on one side of the breach, in addition to slowing the breach development, with the result that the peak breach flow rate would be reduced.

- **Deposition of eroded rockfill material and development of tailwater**: The rockfill material moved by the breaching process would simply ravel downstream by the flow through the breach and much of this material would be deposited a short distance downstream of the dam. This would cause a significant tailwater to develop that would reduce flow velocities through the breach and inhibit both downward and lateral development of the breach with the result that a narrower breach would be formed, taking a longer time to form, and resulting in a lower peak breach flow rate.

The combination of all these considerations provides strong evidence that the use of mean values of breach geometry parameter estimates for Jocassee Dam would be a conservative choice. In reality we would expect that the breach geometry parameter estimates for Jocassee

Dam are in the range between the mean and lower bound confidence interval estimates obtained from the Xu and Zhang (2009) regression equations.

Similar arguments made for breach geometry parameter estimates for Jocassee Dam being in the range between the mean and lower bound confidence interval estimates can be made for failure time, except that they would support a failure time estimate between the mean and the upper bound confidence interval estimate. Therefore, the use of the mean estimate for failure time is also considered to provide a conservative estimate of the failure time as defined by Xu and Zhang (2009) for the reasons stated above.

6.4 Use of Failure Time Estimate in HEC-RAS Model for Jocassee Dam

The importance of consistency between the definition of the Xu and Zhang (2009) failure time when using failure time estimates in the HEC-RAS breach model is pointed out in Section 5.0. Figure 6 contains breach hydrographs for headwater (blue line related to left scale * 1,000), tailwater (brown line related the left scale * 1,000) and breach discharge (green line related to the right scale) obtained by HDR for a piping failure of Jocassee Dam at a location immediately downstream of the internal boundary in HEC-RAS model that represents the dam. The rate of breach progression that is defined as a HEC-RAS input is shown by the black line (related to the left scale). Points A, B and D are equivalent to points that are defined in Figure 3 for the Teton Dam failure in Section 5.0.

To achieve a compatible implementation of the Xu and Zhang breach parameter estimates in HEC-RAS the Xu and Zhang mean breach failure time (between points A and D) and mean breach geometry estimates were input to the HEC-RAS model. The Xu and Zhang mean peak breach flow rate estimate was then closely matched by adjusting the piping coefficient to a value of 0.1, the breach weir coefficient to a value of 2.0, and the sinusoidal rate of breach progression relationship was adjusted between points B and D as shown in Figure 6.

The adjusted values of these coefficients appear to be reasonable in terms of representing flow through the rockfill material and flow through the breach following collapse of the dam crest of the rockfill dam, respectively. In addition the form of the resulting breach hydrograph also appears to be reasonable for a piping failure mode and closely matches the Xu and Zhang mean peak breach flow rate estimate. We consider this to be a *realistic but conservative* breach hydrograph that has good *defendablity* based on the validity of the Xu and Zhang (2009) method, the conservative nature of the mean breach parameter estimates, a piping failure mode initiating in the abutment, the deposition of rockfill immediately below the dam, the low erodibility of the rockfill material, and various characteristics of a modern dam that were include in the design and construction of Jocassee Dam.

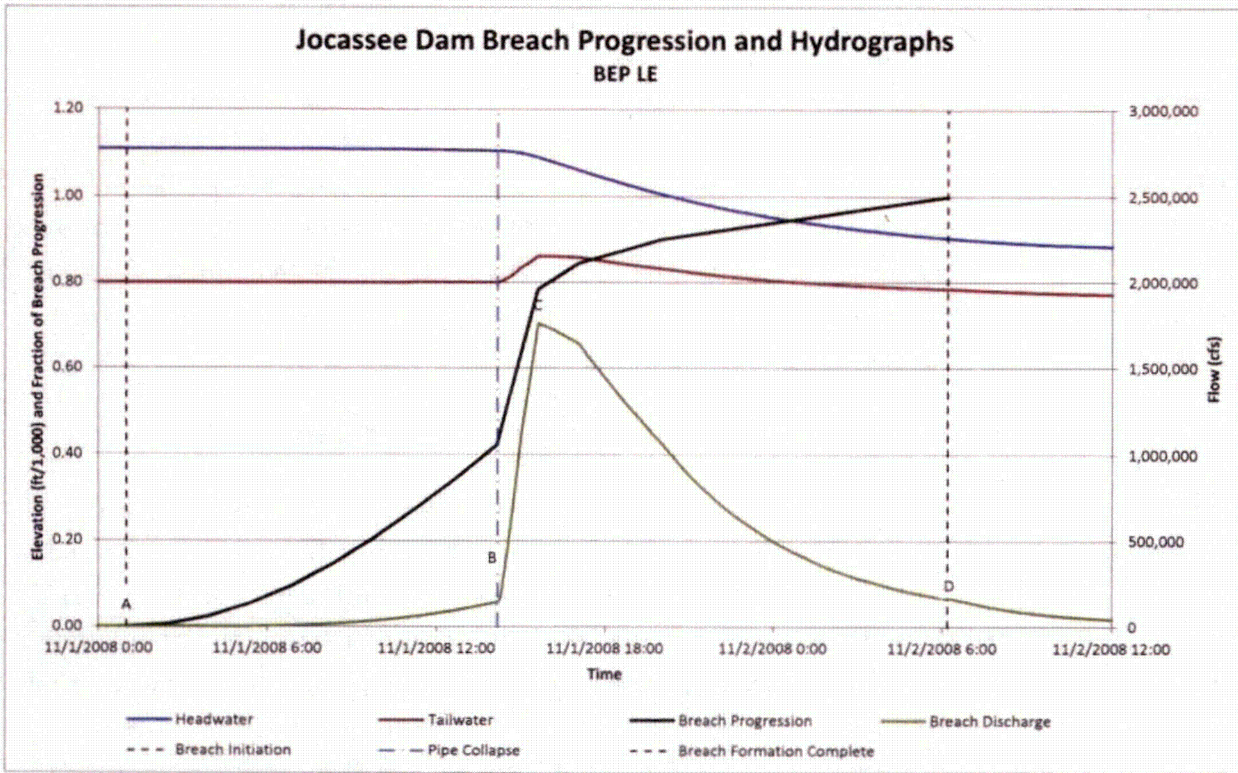


Figure 6. HDR's HEC-RAS Jocassee Dam breach hydrographs

7.0 Review of Keowee Breach Parameters

7.1 Overview

Section 7.2 provides a summary of some key features of Keowee Dam that affect its response to an upstream failure of Jocassee Dam. Section 7.3 is our review of the breach parameters proposed for Keowee Dam.

7.2 Keowee Dam

The Keowee Dam is an embankment dam consisting of (b)(7)(F) [redacted]. The embankment is approximately 3,300 feet in length and is confined between the (b)(7)(F) [redacted] and the natural high ground forming the (b)(7)(F) [redacted] The crest is at EL 815 or 15 feet above normal Keowee Lake level of EL 800. The soil materials contained in the dam are (b)(7)(F) [redacted] with regard to Briaud (2008) erodibility resistance categories used by Xu and Zhang (2009) as described in Section 6.1.

Given the (b)(7)(F) [redacted] nature of the Keowee Dam, overtopping i(b)(7)(F) [redacted] to result in a dam breach regardless of the rate of rise of the headwaters. Given the fact that a Jocassee Dam breach would cause a relatively rapid rise in headwaters at Keowee, it is anticipated that the flood waters would (b)(7)(F) [redacted] of the embankment.

7.3 Breach Parameters for Cascade Failure of Keowee Dam

The breach parameters for Keowee Dam should correspond to the breach process that would be expected to result at this dam for the breach flood wave from the postulated piping failure of Jocassee Dam. For overtopping dam failures, the empirical breach parameter methods, such as Xu and Zhang (2009), are based on case histories of overtopping dam failures resulting from natural inflow floods. These natural floods would be expected to have a slower rising limb than the flood wave generated by the failure of a major dam, such as Jocassee Dam. Typically a natural overtopping flood first results in overtopping of a low spot of the dam (accounting for wind and wave effects), or at a higher rate of overflow overtopping can occur at more than one low spot in the dam crest with lesser overflow over extensive portions of the crest. The rate of rise of the overtopping hydrograph for natural floods is likely to provide opportunity for a breach to develop at one of these low spots, or as a result of vulnerable conditions on the downstream face of the embankment dam. As the breach development progresses more rapidly it typically draws down the headwater level to below the dam crest level and thus focuses all the available erosive energy on developing a single breach rather than developing multiple breaches along the dam crest. The interactions between potential breaches initiating at multiple locations along an embankment dam are not considered in breach algorithms such as that in HEC-RAS. They are considered in the physically-based models developed by Wang and Bowles (2006a-d, 2007).

In contrast to the typical overtopping breach process associated with a natural flood that is summarized in the previous paragraph, a rapidly rising hydrograph resulting from the upstream failure of Jocassee Dam would (b)(7)(F) [redacted] Keowee Dam. This would be expected to not only initiate breaching at multiple locations along the dam crest, but (b)(7)(F) [redacted] since the initiation of breaches would likely not reduce the headwater quickly enough to focus flow into only a single breach. Thus, it is expected that following the upstream failure of Jocassee Dam, there would be a (b)(7)(F) [redacted] of the Keowee Dam.

The approach implemented by HDR for an overtopping failure of the Keowee Dam resulting from the postulated upstream piping failure of Jocassee Dam is as follows:

1. a trapezoidal breach of the main high section of Keowee Dam represented using the HEC-RAS breach algorithm with an average breach width of almost 900 feet, a failure time o (b)(7)(F) [redacted] and a sinusoidal breach progression relationship; and

2. an almost 1,900 feet wide rectangular "gate" in the west saddle dam that opens in 30 mins. to simulate the breach developing over essentially the entire remaining length of Keowee Dam.

This dimensionand failure time used to represent the breach of the highly erodible Keowee Dam appears to be reasonable for the rapidly rising overtopping flood wave resulting from the upstream failure of Jocassee Dam based on the considerations discussed above. The *Dam Overtop Trigger Elevation* of 817 feet msl corresponds to two feet of overtopping of Keowee Dam, which is likely much more overtopping than this highly erodible dam would withstand for this cascade failure mode. However, this trigger elevation can be viewed as a conservative assumption in the sense that the higher headwater and greater volume stored behind Keowee Dam for this trigger elevation would result in a higher peak breach flow rate downstream.

The HEC-RAS breach hydrograph provided by HDR for the cascade failure of Keowee Dam shows a very rapidly rising limb consistent with the failure times selected and the assumed breach dimensions that represent removal of most of the main embankment dam at Keowee.

In conclusion, the proposed breach parameters for Keowee Dam appear to be reasonable given the cascade failure mode and non-applicability of the empirical breach parameter estimation methods to this type of failure. Physically-based breach models (Morris et al 2012) may provide some useful insights into the rate and extent of development of the Keowee breach in the longer term.

8.0 Conclusions

For Jocassee Dam:

- Given the design, construction and excellent performance of the Jocassee Dam it is concluded that a piping failure of the Jocassee Dam is extremely unlikely to occur.
- Even in the extremely unlikely event that a piping failure mode initiates at Jocassee Dam, the potential for uncontrolled release of the Jocassee Reservoir through failure of the main section of Jocassee dam is even more remote. This is because the potential internal erosion and piping would progress very slowly, due to the protective filtering systems and/or the resistance of the rockfill to movement and collapse. The present monitoring and observation procedures and systems would indicate movement of materials and would give cause and time for intervention. This intervention could be

(b)(7)(F)

(b)(7)(F) If necessary, it could also be accomplished through a (b)(7)(F) (b)(7)(F) that would result in a flow rate that could be managed at Keowee Dam without threat to the Oconee N.S.

- The Xu and Zhang (2009) method that has been applied for Duke by HDR to estimate breach parameters for Jocassee Dam has been demonstrated to provide the best predictions of breach parameters amongst similar regression methods. It is, therefore, considered to be a "state-of-the-practice" published regression method for estimating piping dam breach parameters. Unlike most other methods, it includes erodibility as a control variable. This variable was found to be the most important variable in the regression Xu and Zhang (2009) method. The capability to consider erodibility is particularly important given that the dense rockfill in Jocassee Dam would be conservatively classified as low erodibility. See Figures 4a and b.

- A review of the analysis performed by HDR indicated that calculations using the breach parameters with consideration of "low erodibility" for the rockfill Jocassee dam reduces the mean estimates of the breach size and peak discharge and increases the failure time considerably compared with previous estimates based on other methods. These low erodibility mean estimates of the breach parameters are associated with the low erodibility of a rockfill dam, and they are considered reasonable and even conservative for Jocassee Dam. They are conservative because of the low to very low erodibility properties of Jocassee Dam such that it would be expected to be a more resistant dam to the breach process than other low erodibility dams for which failure data were used by Xu and Zhang (2009). In addition, the uni-directional breach formation, deposition barrier of moved rockfill material and development of tailwater during breach formation would lead to conservative estimates of breach parameters using the Xu and Zhang (2009) mean estimates.

For Keowee Dam:

- The proposed breach parameters for Keowee Dam appear to be reasonable given the cascade failure mode and non-applicability of the empirical breach parameter estimation methods to this type of failure.

9.0 Recommendations

The following recommendations are made based on our review:

1. We recommend that the Xu and Zhang (2009) breach parameter estimates for a piping failure of Jocassee Dam should be based on low erodibility mean estimates (see values in boxes in Table 2) as *realistic but conservative* values.

2. We recommend that the breach parameters proposed by HDR for Keowee Dam should be used as *realistic but conservative* values for a cascade failure of that dam as the result of the failure of Jocassee Dam.

3. While the Xu and Zhang (2009) regression equation approach presented herein is presently the "state-of-the-practice" and provides for the appropriate range of breach parameters to represent a piping failure at Jocassee Dam; in the longer term, we recommend that future consideration be given to a physically-based model. When available, this type of model may provide additional insights to better define the conservative nature of the present evaluation by considering the material properties, site geometry, reservoir capacity-stage relationship, the uni-directional breach development starting at a relatively high elevation in a (b)(7)(F) [redacted] and other factors such as the effect of the tailwater development due to deposition of rock downstream of the breach. It is recommended that the application of this type of model should be considered using bounding values of model inputs to better understand and appreciate the beneficial nature of the physical features on limiting breach development for the Jocassee Dam.

4. For Keowee Dam, in the longer term, we also recommend that future consideration be given to a physically-based model. This type of model may provide additional insights considering the characteristics of a Jocassee breach flood wave arriving at Keowee. It is recommended that the application of this type of model should also be considered using bounding values of model inputs to better understand the physical limitations on breach development for the Keowee Dam as a validation of the estimates reviewed in this report.

We note that at present the HR-BREACH physically-based breach model, which could be applied in the future in response to Recommendations 3 and 4, is undergoing further development and testing for a piping failure condition for a zoned rockfill dam, such as Jocassee; however, in the future such research and verification may be available.

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Appendix A

Descriptions of Potential Piping Failure Modes for Jocassee Dam

The potential piping failure modes identified for Jocassee Dam during the RAC PFMA are summarized below followed by a comment on breach by sloughing and unraveling.

JPFM 5a1: Piping Through the Foundation
(Natural Dam) in the (b)(7)(F)

The location of JPFM 5a1 is in the (b)(7)(F) area where there is no rockfill dam and a "Natural Dam" overlying the bedrock foundation retains the reservoir. Figures A.1 and A.2 show sections through the (b)(7)(F) of the main Jocassee Dam. The section forms a "Natural Dam" in the locations where the ground surface of the section is above the dam crest elevation. The location of this "Natural Dam" section is shown on Figure A.3 and labeled as "JPFM 5a1". The failure mode is summarized in the following two paragraphs.

The reservoir rises to a critical level at which time a seepage path develops through fractures in the weathered rock. The seepage passes into the residual soil which forms the downstream slope of the natural dam. The seepage initiates movement of the residual soil as well as infilled soil in the joints and fractures of the weathered bedrock.

The eroding material moves toward an unprotected exit near the toe of the natural dam and beyond the core/cutoff trench of the rockfill dam; the unprotected exit allows the movement of material to continue. As the infilled joint material continues to erode the seepage velocities and the flow rate increase. The fractures remain open and the internal erosion process progresses. The process may or may not be detected but successful intervention does not take place. The internal erosion process eventually begins to increase uplift pressures under the downstream slope of the natural dam causing sloughing or slope instability to occur.

JPFM 5a2: Piping Through the Foundation
(b)(7)(F)
Near the (b)(7)(F)

Failure mode JPFM 5a2 is in the (b)(7)(F) area but is located where there is a (b)(7)(F) section of the rockfill dam overlying the bedrock foundation. If the same mechanism described above for failure mode JPFM 5a1 occurs at a location where the Main Dam is underlain by the natural dam then the seepage may exit into the Rockfill shell of the main dam and cause failure of the dam by unraveling. At this location the main dam is abou (b)(7)(F) The location of (b)(7)(F) [redacted]

Figure A.1. (b)(7)(F) [redacted] **section (from page 1 of 3**
of the HDR (b)(7)(F) [redacted] **Geological Study).**

(b)(7)(F) [redacted]

Figure A.2. Sketch of piping through rock foundation
in the (b)(7)(F) [redacted] **failure mode JPFM 5a-1.**

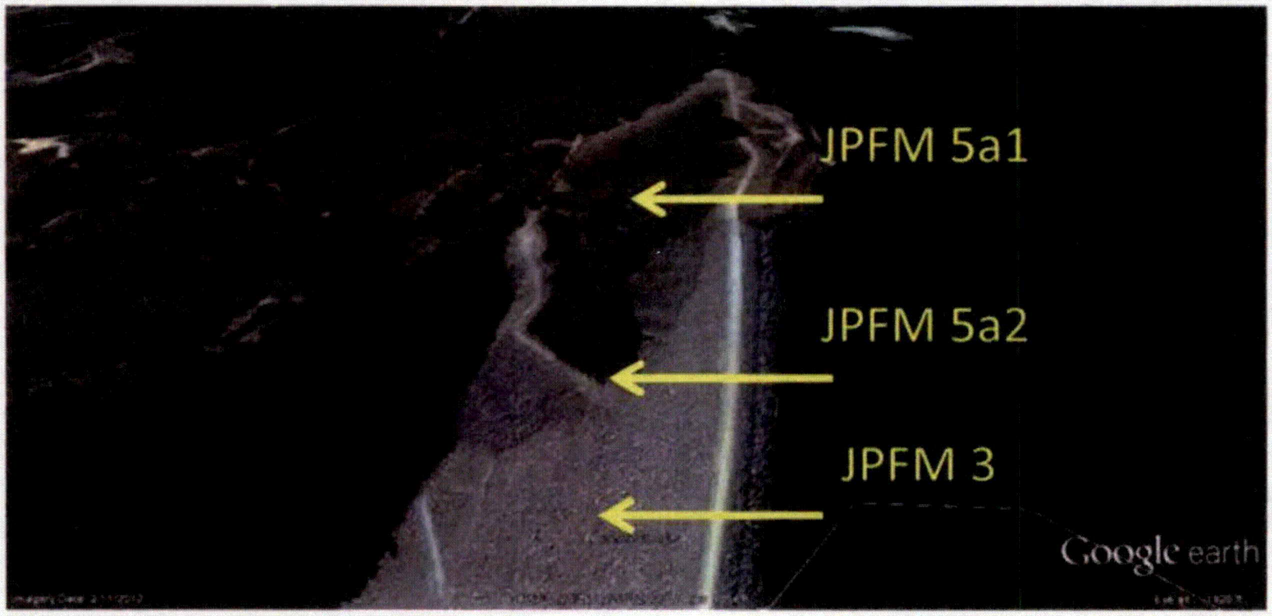

Figure A.3. Google view showing the locations of failure modes JPFM 5a1, 5a2 and 3 in the (b)(7)(F) [redacted]

thi (b)(7)(F) [redacted] is shown on Figure A.3 and is labeled as "JPFM 5a2." The process of piping through the foundation for JPFM 5a1 and 5a2 is essentially the same. For both cases the flow is through joints and fractures in the bedrock, eroding infilled material, and does not emerge from the bedrock fractures until well downstream beyond the location of the cutoff trench and core projected into th (b)(7)(F) [redacted] The primary difference between JPFM 5a1 and 5a2 is that for JPFM 5a2 the seepage exits from the foundation into the downstream rock shell of the dam causing unraveling[1] of the shell, and for JPFM 5a1 the seepage exits from the foundation into the downstream natural slope causing sloughing and slope instability.

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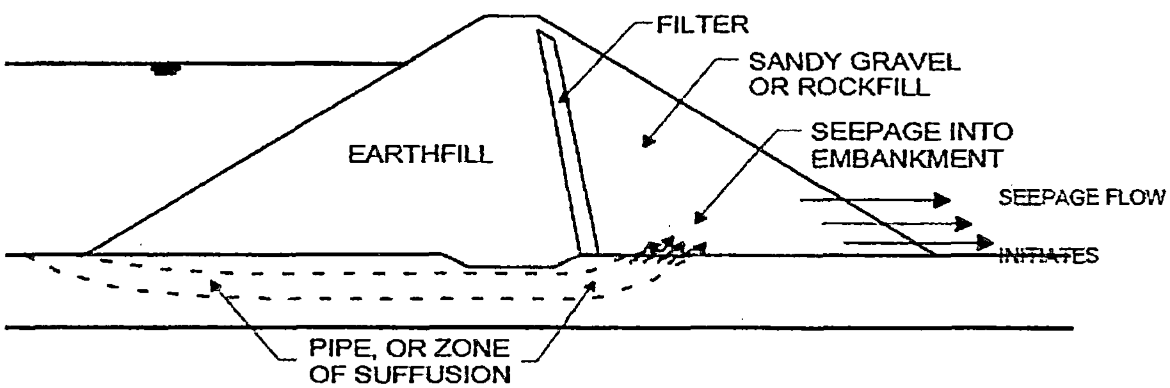
[1] **Comment on breach by sloughing and unraveling**

> *"For breach to occur by unraveling or sloughing of the downstream face of the embankment, the increased seepage due to internal erosion and piping in the foundation would have had to discharge into the downstream zone as shown in* the figure below.
>
> *For sloughing to occur, the downstream face would have to be relatively steep, and the shoulder material a cohesionless soil, probably sandy gravel, or gravely sand, possibly with some silty fines. The process would have to be allowed to continue until it gradually eroded away the crest and allowed the reservoir to overtop.*
>
> *Unraveling usually relates to the progressive removal of individual rocks by fairly large seepage flows flowing through the downstream rockfill. Foster and Fell (1999) indicate that in the database they assembled, there was only one case of piping failure in an embankment with breach by unraveling* (After Reclamation-USACE-URS-UNSW 2008).

JPFM 5b: Piping Through the Foundation (Natural Dam) at the Interface Between the Weathered Rock in the (b)(7)(F) and the Core Material

The failure mode in the Piping and Seepage toolbox that fits this failure mechanism the best seems to be piping through the embankment Initiating Mechanism IM15. The following sequence of events describes this failure mode:

1. Occurrence of a threshold pool level which creates a gradient greater than the critical gradient of core or foundation material that exists at the interface between the (b)(7)(F) (b)(7)(F) foundation material.
2. A seepage path exists along the interface with the select core material.
3. Movement of the select core material initiates.
4. The select core material moves into an exit point beyond the flared filter material and into the rockfill shell or the downstream face of the (b)(7)(F)
5. A pipe (crack) develops in the select core material or the residual foundation soil material and progresses upstream to the upstream shell.
6. The core material is capable of supporting a roof and does not collapse into the pipe (crack).
7. Core or foundation material continues to move through the pipe (crack) in the select core material.



Breach by unraveling or sloughing of the downstream slope (Reclamation-USACE-URS-UNSW 2008).

8. The upstream filter material fails to fill the pipe (crack) and to plug the exit into the downstream shell.

9. The seepage may or may not be detected but intervention is not successful.

10. A breach develops by sloughing and unraveling or by sinkhole development.

JPFM 4: Piping Through Rock Foundation in the (b)(7)(F) [redacted]

The description of the piping mechanism for piping through the rock foundation in the (b)(7)(F) [redacted] (b)(7)(F) [redacted] of the dam in the location shown in Figure A.4 is essentially the same as for the piping failure mode JPFM 5a2 in the (b)(7)(F) [redacted] of the dam. Figures A.1 and A.2 for the (b)(7)(F) [redacted] are also applicable for the (b)(7)(F) [redacted].

JPFM 3: Piping Through Embankment

For this failure mode the initiation of the flaw occurs by "Cross Valley Differential Settlement" as described by the initiating mechanism IM1 in the Piping and Seepage Toolbox. The schematic drawings of the failure mechanism in the toolbox are shown below as Figures A.5 and A.6.

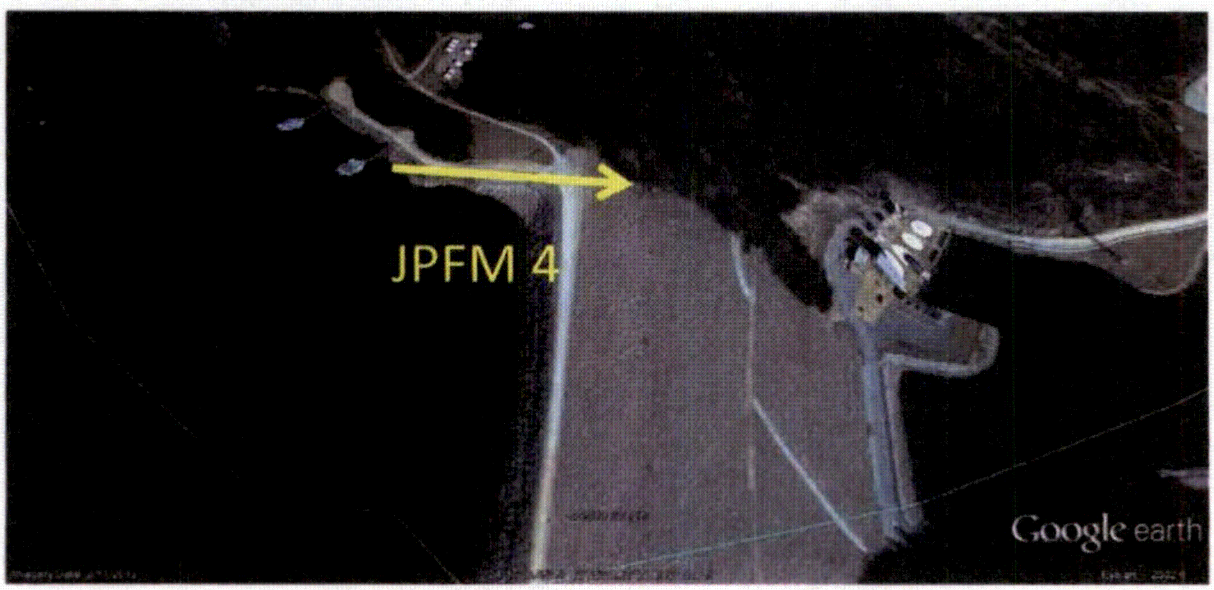


Figure A.4. Location of foundation piping failure mode JPFM 4 in the (b)(7)(F) [redacted]

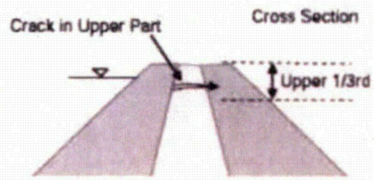


Figure A.5. Location of the transverse crack in the upper third of the dam section.

IM1 – Cross Valley Differential Settlement

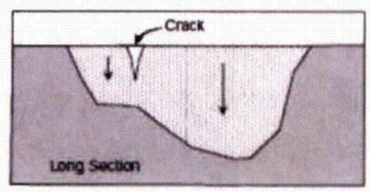


Figure A.6. Cross Valley Differential Settlement causes transverse cracking in the upper third of the embankment.

The reservoir rises to a critical level to initiate seepage through a crack in the core caused by Cross Valley Settlement (Initiating Mechanism IM1). There is sufficient velocity flowing through the crack that core material along the sides of the crack begins to move toward the downstream side of the core. There is a flaw in the filter that aligns with the crack so that the material is not stopped at the filter. Erosion continues to occur and material from the upstream filter does not fill the crack. The internal erosion is not detected and continues to develop. As the material erodes, a void develops in the core material. The void eventually collapses and a sink hole develops at the crest of the dam or upstream of the crest. The process may or may not be detected but successful intervention does not take place. Breach of the dam occurs by sloughing or unraveling or by sinkhole development.

The location of a breach for an embankment piping failure mode would occur near the [redacted] where there is a break in grade in the foundation profile as schematically shown in Figure A.6. The same discussion as given in the footnote on breach by sloughing and unraveling for failure mode JPFM 5a2 applies to this embankment piping failure mode.

Appendix B

Breach Parameters Estimates from Xu and Zhang (2009) Application to Jocassee Dam with Comparison to HDR Mean Estimates

Breach Parameter	Best Exact Prediction: Eqn. No.	Regression Parameters	Mean HDR*	Mean	95% Confidence Interval Lower	95% Confidence Interval Upper	Best Simplified Prediction: Eqn. No.	Regression Parameters	Mean HDR	Mean	95% Confidence Interval Lower	95% Confidence Interval Upper
Height of breach, Hb ft.	Eqn. 9	Zone-filled Dams b3 0.132 Piping b4 0.236 Low b5 0.031	255	253	168	337		Low C1 0.858	255	255	176	334
Failure time, Tf hrs.	(b)(7)(F)											
Breach top width, Bt ft.	Eqn. 14	Zone-filled Dams b3 -0.089 Piping b4 -0.239 Low b5 -0.289	701	701	306	1,606		Piping b4 -0.262 Low b5 -0.288	666	666	299	1,485
Average breach width, Bave ft.	Eqn. 15	Zone-filled Dams b3 -0.226 Piping b4 -0.389 Low b5 -0.391	566	566	246	1,306		Piping b4 -1.747 Low b5 -1.268	515	515	223	1,193
Peak outflow, Qp ft3/sec.	(b)(7)(F)											

* Note that HDR used the Best Simplified Prediction for Height of Breach instead of the Best Exact Prediction

~~Contains Security Sensitive Information.~~
~~Withhold from public disclosure under 10CFR 2.390.~~

ENCLOSURE 2

Board of Consultants/FERC EFM Review Meeting report

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April 4, 2013

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Subject: Oconee Nuclear Station
Board of Consultants/FERC EFM Review Meeting
March 12-14, 2013

Dear Mr.Arnold:

INTRODUCTION

The subject of the Board meeting consisted of the potential effects of a "sunny day" failure of Jocassee Dam on the Oconee Nuclear Station (ONS) and the protection measures being designed to protect the Station. The protection measures that were discussed are based on certain breach parameters that resulted in a failure time of (b)(7)(F) refer to herein as "current" breach parameters. The Board response to Question 1 deals with a study of alternative breach parameters being conducted by Duke Energy and its consultants that leads to substantially longer failure time of (b)(7)(F) referred to herein as "new" breach parameters

The Board responses to Questions 2 through 6 address the investigations and analyses required for the necessary modifications to protect the Oconee Nuclear Station against flooding caused by a failure of Jocassee Dam, assuming the current breach parameters of the dam consistent with a failure time of (b)(7)(F) (b)(7)(F). Duke Energy refers to the use of the current parameters as path 1.

BOARD QUESTION 1: ***Are breach parameters reasonable?***

The Board of Consultants has been asked to comment on the new breach parameters for Jocassee Dam based upon information provided by Duke Energy and CB&I.

Field trips were taken to the dam sites. Presentations were given of the computer simulations of potential failure of Jocassee Dam and the routing of the flood through Lake Keowee and Little River. Runs were made using 1-D and 2-D HEC-RAS routing along with several scenarios of the breach parameters. The breach parameters considered were those of Froehlich (1995) and Xu and Zhang (2009). The Board considered the Xu and Zhang parameters to be "state of the art" based on their more extensive data of dam failures and breach development. Xu and Zhang found dam erodability to be the most important control variable for predicting their breach parameters, which had not been considered in the current breach parameters that led to the (b)(7)(F) failure time. Noting that Jocassee Dam has been monitored since 1972 when it was constructed, there is no indication of excessive seepage that would indicate potential piping or movement of any soil or internal erosion. . Ehasz and Bowles (2013) evaluated the Xu and Zhang breach parameters with respect to Jocassee and Keowee dams. They reported the breach parameter of low erodability as estimates for Jocassee Dam would provide a conservative estimate of the failure time as defined by Xu an Zhang. The Board agrees with the low erodability assessment for Jocassee Dam.

As noted by Ehasz and Bowles, the failure times for the current and new parameters are not defined in the same manner. The failure time of (b)(7)(F) for the current parameters is defined in accordance with Froehlich and corresponds to the period of time during which the great majority of the outflow takes place. The failure time of (b)(7)(F) for the new breach parameters follows the Xu and Zhang definition that includes the total time for the breach development phase. The new parameters would have a failure time of about (b)(7)(F) using the Froehlich definition.

. The Board is not aware of any piping failures in properly constructed central core-rockfill dams that have been in operation successfully for extended periods of time, i.e. 40 years with a relatively constant net head. Furthermore, the breach parameters evaluated by Xu and Zhang as well as those determined by Froehlich are related to the "development phase of the breach", which takes place once the piping process has advanced sufficiently to trigger sliding, sloughing of the shells and breaching of the core. However, in our case it might be pertinent to review the history of the Jocassee dam performance to comment on the "warning phase" during which the piping process is initiated and developed. The Jocassee dam reservoir was filled in 1972 and most seepage detected corresponded to abutment springs in the upper 100 ft below reservoir level. Leakage in the (b)(7)(F) has remained at steady flow of about 20 to 30 gpm. On the (b)(7)(F) leakage is higher, 700 to 800 gpm, and increased only during two episodes in 1977 and 1978, when seeping water with concurrent development of subsidence features were detected on the right abutment downstream of the dam axis. In both cases it was determined that seepage took place through soil-filled joints in partially weathered rock. Additional grouting with sodium silicate was carried out in the affected areas, seepage was substantially reduced and has remained constant to date. Also, filtered wells were installed in the subsidence areas. It is pertinent to note that during the 1978 subsidence event the reservoir level was lowered 15 ft below normal pool level as a precaution. The volume of water released corresponded to a significant portion of the total reservoir volume.

In the board's view, the dam performance to date indicates that:

a) Unusual seepage events were promptly detected and timely and successful intervention was implemented. The springs appeared in the right abutment rock, about 300 ft downstream of the crest of the dam.
b) The "preferential seepage paths" susceptible to piping developed mostly along in-situ, weathered, materials, outside the footprint of the dam; and took place over an extended period of time that allowed for a lowering of 15 ft of the reservoir level.
c) The preferential paths have been grouted & plugged successfully since no seepage increase has been reported after 1978.
d) The areas of concern are heavily instrumented, i.e. piezometers and weir instruments

Based on these considerations, it is plausible to argue that installation of automatic data loggers in the existing instruments, combined with frequent inspections would allow for a timely detection of signs of distress. An action plan can be implemented to initiate spillway discharge to reduce the reservoir volume prior to an eventual "dam breach" which will significantly ameliorate the impact on Keowee Dam and the Oconee Nuclear Station. The potential for reducing the reservoir level is not considered in the breach parameters estimates and thus it further supports the fact that the new "breach parameters" are conservative.

Breach Development phase- A revised breach failure time of (b)(7)(F) has been proposed for Jocassee Dam. A relevant precedent to support this assessment can be the failure of Hell Hole Dam in California in 1964. The dam was designed as inclined- core rockfill dam. In December 1964, the last lift of dumped rockfill, 220 ft high (crest elevation 4470 ft), had been placed when an intense storm deposited 18 in. of rain at the dam site. The core, at a lower elevation than the crest was overtopped at 1:30 pm on December 21 and flow commenced through the rockfill; water emerged from the downstream toe at 3:00 pm on December 22, and a maximum water level was reached at 8:20 am on December 23. By 3:30 pm on December 23, most of the water impounded behind the rockfill had flowed through the washout removing about 700,000 cu yd of rock downstream. A conservative assumption for the breach initiation when the flow emerged at the downstream toe would result in a 24 hour long breach time for the 220 ft high dumped rockfill at Hell Hole Dam. Jocassee Dam has a higher height of breach, 255 ft, but the rockfill was compacted in (b)(7)(F) which makes it more resistant to erosion, although the Hell Hole rockfill size was larger. More significantly however, Jocassee Dam volume is 11,000,000 cu yd, which is an order of magnitude larger than the Hell Hole Dam removed volume. This comparison is qualitative but indicates that the (b)(7)(F) breach time proposed for Jocassee Dam is conservative.

Since Jocassee Dam has been operating and has been monitored since 1972 and will continue to be monitored and inspected, the Board considers the Xu and Zhang breach parameters that result in the longer breach hydrograph period are conservative for routing the flood downstream through Keowee Dam.

BOARD QUESTION 2: ***Please comment on the proposed subsurface investigations. What modifications do you suggest for the proposed investigations?***

The proposed field and laboratory geotechnical investigation programs for the ONS and for Keowee Dam are reasonable and should provide the parameters needed for the design of the Fukushima External Flood Mitigation Project (EFM), path 1. Like any geotechnical investigation one should adjust the program as information is developed. Given the time required to perform additional subsurface investigations because of site clearance needs, in the Board's opinion the scope of the initial program is appropriately more extensive than would likely be eventually needed. We suggest replacement of some of the borings with CPT soundings. For example along the crest of the intake canal dike one could add CPTs next to two of the proposed borings for the purpose of calibrating the soil classification inferred from the CPTs. Then the rest of the borings could be replaced with CPTs assuming that the two pairs of Borings/CPTs indicate soils which lend themselves to CPT soundings.

It might also be useful to review existing records of embankment materials testing during construction, i.e. grain size distributions, compaction tests, in-situ permeability tests and other field observations. Also the specification requirements for placement of embankment materials should be reviewed. Also, it would be useful to have for Keowee Dam a description of current embankment conditions, including pertinent cross-sections with seepage line elevations. During the site visit it was noticed that the seepage line intersected the downstream slope above the toe, especially in an area near the west corner. Significant seepage was also observed in the rock cut at the (b)(7)(F) [redacted] This information may be helpful in selecting critical areas for additional exploration.

Also, it would be beneficial to have:
a) The estimates of the current factor of safety of the embankment slopes.
b) The range of actual fluctuations of reservoir pool elevation since operations started.
c) Estimated peak ground accelerations during the 1979 earthquake.
d) History of concrete spillway discharge to help assessing the potential for erosion downstream.

BOARD QUESTION 3: ***Please provide your input for our in-process documents on safe drilling procedures near dams and contingencies for appearance of internal erosion and excessive borehole sloughing.***

The proposed in-process procedures address cases in which boreholes encounter artesian conditions or loss of drilling fluid and the responses to these conditions while drilling and while grouting. The proposed actions are reasonable. It should be made clear that the documents address the responses of the field engineer/geologist and not of the driller, and that the action of "calling Engineer" refers to the Engineer in overall charge of the geotechnical work for the project. The field engineer/geologist should have the primary responsibility to monitor that proper drilling procedures are followed and to detect any unusual events, and should be familiar with the main objectives of the investigations, drilling procedures in general and the specific requirements for drilling in embankment dams.

While the responses to artesian conditions are well planned, it is important to anticipate and plan for their possible occurrence. Artesian conditions are not likely to be found when drilling from the crest of the embankment but can be found when drilling at the downstream toe. Such a condition can be anticipated if there is knowledge of piezometric levels at the location of the proposed borings. If there are existing piezometers their readings should be reviewed. If there are no piezometers in the general area, we suggest that a CPT sounding with pore pressure measurements capabilities (piezocone) be pushed stopping at various depths until the pore pressure reading stabilizes. In this manner one would get a clear picture of the piezometric conditions and whether there is a vertical gradient, and specifically whether artesian conditions exist at any depth, and if so how high above the ground surface would the water level in the borehole rise. Then drilling can be made with the appropriate length of casing above the ground surface to prevent flow from the ground into the borehole. The CPT equipment should have the capability of grouting as the CPT is withdrawn.

Loss of drilling fluid can occur when either the boring encounters very pervious materials or the pressure in the drilling fluid become high enough to cause cracks to develop in the ground. Losing drilling fluid into very pervious materials can be stopped by driving casing below the bottom of such materials before recharging the borehole with drilling fluid. There are no significant consequences to such loss of fluid if it is detected quickly and prompt action is taken. On the other hand loss of fluid due to cracking of the ground should be avoided, particularly within the body of an embankment dam or in its foundation. If safe drilling procedures are followed, cracking of the ground is highly unlikely. General recommendations for drilling in embankments are presented in The Corps of Engineers publication ER-1110-1807 "Engineering Design Procedures for Drilling in earth Embankments". It is recommended to use casing and to keep the casing no more than 5 ft above the bottom of the borehole to minimize the area of the embankment soil on which the drilling fluid pressures are applied. It is important to keep the drilling pressures at the bottom of the borehole at a minimum. One of the causes of excessive drilling fluid pressures at the bottom of the borehole is blockage to the flow of the return of drilling fluid, thus return should not occur along the outside of the casing since this annular space can be easily blocked. Return should be between the drilling rods and the casing. Advancing the borehole too fast in clayey soils can result in clay chunks in the return fluid that can block the annular space between the drilling rods and the casing and lead to high drilling pressures. The COE publication recommends that the borehole be advanced no faster than 1 ft/min. It is important to monitor closely the pressure at which the drilling fluid is being pumped into the drilling rods, any sudden increase in pressure may be an indication of blockage and thus the advance of the borehole should be stopped until the pressure returns to normal levels. Rapidly raising and lowering the drilling rods should be avoided because it causes a pumping action with increases in drilling fluid pressure at the bottom of the borehole.

Cracking is more likely when drilling through the embankment in areas where the horizontal normal stresses on upstream-downstream vertical planes can be expected to be lower than for at- rest conditions, and thus it is prudent to avoid drilling at such locations. Such locations can be identified by examining a longitudinal profile along the crest of the dam and the change in the height of the dam and in the depth of compressible soils beneath the dam, if any. For this purpose an example of soils to be considered compressible would be clays that are normally consolidated or become normally consolidated under the weight of the dam. At locations where the height of the dam or the depth of compressible soils changes abruptly there would have been significant differential settlements along the dam during construction which would have reduced horizontal stresses and thus drilling should be avoided anywhere close to such locations. It is not possible to develop rules that would quantify what an abrupt change is since it would depend on how compressible the embankment material and foundations soils are and the length over which the abrupt change occurs. For the general conditions where borings are planned for the EFM project a depth change with a slope of more than about 2H:1V may be considered abrupt. The Board examined the longitudinal profile for the West Saddle Dam and Keowee Dam and in our opinion no such locations are present. The Board recommends that the longitudinal profile along the Intake Canal Dike be examined for potential conditions unfavorable for drilling.

BOARD QUESTION 4: ***Given flooding defined in the SER, is RCC overtopping protection feasible and can it be implemented safely on an embankment retaining a normal pool with a functioning powerhouse? Should the design team consider other overtopping protection methodologies?***

The Board believes that RCC slope protection can be constructed on the downstream slope and the crest of Keowee Dam without damaging the embankment, provided that the work is done with appropriate care. The RCC armoring layer should be constructed on a permeable bedding layer to prevent uplift pressures beneath it. The upstream edge of the RCC armoring layer on the crest of the embankment can be protected against uplift due to water pressures beneath it by tying it into a row of driven sheet piles along the upstream edge. Using sheet piles for this purpose rather than a concrete wall will avoid having to excavate into the top of the dam. At the lower edge of the armoring layer on the downstream slope, the RCC can be extended horizontally beyond the toe of the embankment to form a stilling basin of appropriate length. The downstream end of this stilling basin can be protected against back-cutting erosion by a row of driven sheet piles. This row of sheet piles should have "windows" formed by omitting every third or fourth pile, or by using shorter piles at the same intervals to avoid blocking flow within layered soils beneath the slab. Drain holes through the stilling basin, angling upward in the downstream direction, will prevent buildup of uplift pressures beneath the RCC slab.

For the design of the RCC overtopping structure it is important to avoid that the overtopping protection spillway begins operation before the original capacity of the service spillway is achieved. In the Board's view, we should try to maintain the current crest of the embankment constant to maximize the hydraulic capacity of the existing spillway before operation of the RCC overtopping structure is triggered. The installation of the overtopping structure can lead to introducing a new potential mode of failure, embankment erosion, for a more frequent event than the maximum capacity of the existing service spillway. In this regard, the RCC design should also consider the operation of the RCC overtopping protection structure for floods with return periods shorter than those anticipated with the Jocassee dam failure.

While other overtopping protection technologies such as riprap could also provide resistance against erosion, the Board believes that armoring the embankment with RCC will afford robust erosion protection, and is a more appropriate choice considering the high flow velocities associated with the postulated overtopping event.

BOARD QUESTION 5: ***What analyses are critical to the design of these modifications? What analysis methods and design standards does the Board recommend?***

Safe design of the proposed modifications will require several types of geotechnical analyses, including those listed below. The Board recommends the following analysis methods, and the use of design standards in accordance with the practices and policies of the U. S. Army Corps of Engineers:

- Evaluation of precedent. Key parameters for the proposed overtopping structure, including the maximum unit discharge, cfs/ft, maximum overflow height, and Maximum height of dam. These need to be compared with performance of observed precedent in other RCC overtopping structures.
- Slope stability. Limit equilibrium analyses using two computer programs, such as Slope/W and SLIDE are suitable for slope stability analyses. Using two programs is recommended as the most effective and efficient method of checking results. The Keowee reservoir loading surcharge triggered by failure of the Jocassee dam is likely to increase the pore pressures within the embankment. Given the existing high seepage line and (b)(7)(F) [redacted] it would be pertinent to make sensitivity analysis to assess the factor of safety of the downstream slope. If the RCC is included in the stability analysis it should be assumed that the RCC is cracked and its strength is only the frictional resistance.
- Seepage. Finite element seepage analysis programs such as Seep/W and Slide are appropriate for seepage analyses. Although some zones within the cross sections analyzed may be partially saturated, the Board recommends using conventional seepage analysis procedures, where the quantity of seepage and the pore pressures above the phreatic surface are assumed to be zero. The quantity of seepage above the phreatic surface is very small, and it is conservative to neglect the soil suction within this zone. In addition, analyzing only saturated flow avoids the complication involved in estimating initial suction conditions, soil-moisture characteristic curves, and the relationship between soil suction and hydraulic conductivity for partially saturated soils. Depending on the duration of the surcharge, the quality of the RCC upstream cut-off barrier might need to be upgraded to minimize additional embankment seepage which can develop further pore pressures in the downstream slope. A thorough evaluation of the potential for this increase and the impact on the stability of the embankment is required at this stage.
- Sliding. The Board agrees that limit equilibrium spreadsheet calculations will be appropriate for analysis of safety against sliding.
- Toe erosion. The Board agrees that the potential for erosion caused by flow parallel to slopes can be approached using river erosion technology, and the methods for evaluating scour that have been developed recently by Briaud, 2008.
- Side Slope Erosion. Permissible water velocities for flow to avoid erosion of soils of various grain size are available in the report of the ASCE Special Committee on Irrigation Research (1926), and in hydraulic engineering textbooks.
- Protection Liftoff. Negative pressures from the overflowing water can be minimized by care during construction of the RCC to insure that the profile is smooth at joints between sections. Where the RCC overlay is stepped, the flow will be tumbling, with no significant negative pressures. Because the protection will experience only one flow episode, cavitation is unlikely to be a problem.
- Excavation Dewatering. The Board agrees that well flow formulas will be appropriate for analysis of flows and pressures due to dewatering, and suggests that a dewatering contractor be engaged if significant dewatering is required.
- Pressures on Buried Structures. For rigid buried structures such as reinforced concrete box culverts and vaults, the Board recommends the use of earth pressure distributions like those developed by the U. S. Army Corps of Engineers. For flexible buried conduits, the Board recommends the use of charts and formulas developed for design of flexible metal culvert structures (Duncan and Drawsky, 1983).

BOARD QUESTION 6: ***Proposed erosion control measures at the embankment toe of Keowee Dam and along plant slopes exposed to flowing water.***

Even though the RCC will reduce the velocity of the overtopping flow and dissipate some of the energy, the toe of the slope of Keowee Dam should be protected from erosion. Several options are available, such as a riprap blanket, continuing the RCC, or constructing a concrete plunge pool or stilling basin that would extend along the entire length of the embankment toe. The RCC stilling basin alternative was discussed in the Board's answer to Question 4. The riprap protection alternative should also be considered. Quarry or natural rock material for riprap appears to be readily available. Because of the height of the overtopping water and the height of the Keowee embankment, the velocities will be high and riprap rock size will be large. Depending on the overtopping discharge and the thickness of the riprap, flow conditions will be interstitial and overflowing the riprap blanket. Neglecting friction, velocities at the toe of the dam would be approximately 75 ft/s. Actual velocities would be lower based upon estimated friction factor for the riprap. Design of the scour protection would be based on extrapolation of data from riprap tests conducted by the U. S. Army Corps of Engineers (Maynord, 1988) in channels and included in the Corps <u>Engineering Manual No. 1110-2-1601</u> (1994) and tests for the Bureau of Reclamation at Colorado State University (Mishra, 1998) for riprap on steeply sloping channels.

Riprap armoring would be placed along the toe immediately downstream of the piling cutoff wall. To be conservative, the riprap should be designed for the estimated velocity at the toe of the embankment and extend 300 to 400 feet downstream. The riprap should be placed to a minimum depth of 2 to 2.5 times D_{50} of the riprap for about 100 feet from the toe. The rock should be of relatively well graded sizes and the voids filled with a smaller dumped rock while the layers are placed. The riprap layer beyond 100 feet should have a minimum depth of 1.5 to 2.0 D_{50}. The river channel downstream from Keowee Dam will scour during the overtopping event. The riprap layer at the toe of the dam is placed to stop and/or reduce the erosion at the toe and into the embankment. Riprap can also be placed at locations where embankment slopes intersect and where flow might be parallel to the embankment slope.

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