Duke Energy®

DAVE BAXTER
Vice President
Oconee Nuclear Station

Duke Energy
ONO1VP / 7800 Rochester Highway
Seneca, SC 29672

864-873-4460
864-873-4208 fax
dabaxter@dukeenergy.com

November 30, 2009

U.S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, D.C., 20555-001

Subject: Duke Energy Carolinas, LLC
Oconee Nuclear Station, Units 1, 2, and 3
Renewed Facility Operating License, DPR-38, DPR-47, and DPR-55
Docket Numbers 50-269, 50-270, and 50-287
Oconee External Flood Analyses and Associated Corrective Action Plan

References:

1. Duke Letter From Dave Baxter to NRC Document Control Desk, "Final 60-Day Response to Reference 2 [NRC Letter dated April 30, 2009]," dated July 9, 2009
2. Duke Letter From Dave Baxter to NRC Document Control Desk, "Request for Extension of Duke Response Time to Referenced Letter [NRC Letter dated April 30, 2009]" dated May 20, 2009
3. NRC Letter From Joseph G. Giitter to Dave Baxter, "Evaluation of Duke Energy Carolinas, LLC (Duke), September 26, 2008, Response to Nuclear Regulatory Commission (NRC) Letter dated August 15, 2008, Related to External Flooding at Oconee Nuclear Station, Units 1, 2, and 3 (Oconee) (TAC Nos. MD8224, MD8225, and MD8226), dated April 30, 2009
4. Duke Letter From Dave Baxter to NRC Document Control Desk, "Interim 30-Day Response to Reference 2 [NRC Letter dated April 30, 2009]" dated June 10, 2009
5. Duke Letter From Dave Baxter to NRC Document Control Desk, "Response to 10 CFR 50.54(f) Request [NRC Letter dated August 15, 2008]," dated September 26, 2008
6. NRC Letter From Joseph G. Giitter to Dave Baxter, "Information Request Pursuant to 10 CFR 50.54(f) Related to External Flooding Including Failure of the Jocassee Dam, at Oconee Nuclear Station, Units 1, 2, and 3 (Oconee) (TAC Nos. MD8224, MD8225, and MD8226), dated August 15, 2008

A001
NRR

~~This letter contains security sensitive information.~~
~~Withhold from Public Disclosure under 10 CFR 2.390 (d)(1)~~

On August 15, 2008, the NRC issued a request for information pursuant to Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, Section 50.54(f) regarding the protection against external flooding at Oconee including a postulated failure of the Jocassee Dam. Duke responded to the NRC letter on September 26, 2008. In that response, Duke stated that it would update the 1992 DAMBRK model and convert to the Hydrologic Engineering Center – River Analysis System (HEC-RAS) model to predict flood heights at the Standby Shutdown Facility (SSF) following a postulated failure of the Jocassee Dam.

By letter dated April 30, 2009, (Reference 3) the NRC responded to Duke's letter dated September 26, 2008. The response states, in part:

> The NRC staff agrees that a study with the more advanced [HEC-RAS] model and sensitivity analyses would be beneficial because of the uncertainty involved in predicting dam failure and resultant flood levels at Oconee. Dam design operating parameters, including reservoir level, should be used as input to the inundation study to support the safety of the Oconee facility. The sensitivity analyses should include varying key parameters that can affect the on-site flood height (e.g., breach size, reservoir levels, and time to dam failure) individually and in combination over a sufficient range to provide an understanding of how changes impact the flood height estimates. As appropriate, the sensitivity analyses should also consider FERC guidelines or other applicable industry standards as potential methods for representing appropriate ranges for the sensitivity analyses. Regarding the inundation study, Duke must provide adequate technical justification for the various input parameters used in the study. Regarding the sensitivity analyses, Duke must provide adequate technical justification for the selection of parameters to be varied and the range of variability for those parameters.
>
> . . . As discussed in our November 5, 2008, meeting, the NRC staff expects that the analyses discussed above which would establish an adequate licensing basis for external flooding and the technical basis for the proposed closure, to be completed by November 2009. Should Duke find that additional modifications are necessary, that schedule should also be provided by November, 2009.

Duke responded to the April 30, 2009, letter by letters dated May 20, 2009 (Reference 2), and July 9, 2009 (Reference 1). In these responses, Duke provided a project milestone schedule that called for completing the external flood inundation studies and sensitivity analyses and providing a corrective action plan for resolution of the Oconee Nuclear site external flood potential by November 2009.

Duke provides the following three attachments which include information due to the NRC by November 2009, as requested by the letter dated April 30, 2009, and the aforementioned project milestone schedule:

- Attachment 1, "Technical Justification for Inundation Study Input Parameters," provides the technical justification for the various input parameters used in the inundation study.
- Attachment 2, "Technical Justification for Sensitivity Analysis Input Parameters," provides the technical justification for the selection of parameters to be varied in the sensitivity analyses and the range of variability for those parameters.
- Attachment 3, "External Flood Corrective Action Plan," identifies actions that may be considered necessary to provide additional reasonable assurance of adequate protection following a postulated failure of the Jocassee Dam, and an associated feasibility/constructability milestone schedule.

The sensitivity and inundation analyses described in Attachments 1 and 2 and the actions identified in Attachment 3 will be used as inputs into Duke's Engineering Change Program. The Duke Engineering Change Program is applicable to both safety and non-safety related structures, systems, and components and requires thorough and comprehensive evaluations consistent with NRC requirements, appropriate industry codes, and good engineering practice to ensure appropriate design control when identifying, designing, and implementing proposed changes to the Oconee Nuclear Station.

In general, the Duke Engineering Change Program requires consideration of numerous factors when evaluating the feasibility and constructability of projects that may impact the safe operation of the Oconee Nuclear Station. Factors that may be considered include the time needed for design, review, approval, and procurement of needed materials; the availability of specialized equipment; unit operational conditions required for implementation; and insights gained from probabilistic approaches. Duke recognizes that the NRC does not believe that risk insights will demonstrate that the probability of a failure of the Jocassee Dam is so low that it does not need to be considered in Oconee's external flooding analyses. Nonetheless, Duke is obligated to consider phenomenological insights gained from the Utah State University/RAC study when formulating the final corrective action plan. The failure to consider that information would not be consistent with 10CFR50 Appendix B, the Duke Engineering Change Program, or good engineering practice.

Modifications deemed necessary will be implemented commensurate with the safety significance of the issue and prioritized relative to other significant risk reduction construction projects currently in progress at the Oconee Nuclear Station.

Since this letter contains commercially sensitive information, Duke hereby requests the NRC withhold the letter from public disclosure pursuant to 10 CFR 2.390 (d)(1), "Public inspections, exemptions, requests for withholding."

If you have any questions on this matter, please contact Jeff Thomas, Fleet Regulatory Compliance Manager, at 704-382-3438 or Bob Meixell, Oconee Regulatory Compliance at 864-873-3952.

Sincerely,

Dave Baxter, Vice President
Oconee Nuclear Station

Attachments

1. Technical Justification for Inundation Study Input Parameters
2. Technical Justification for Sensitivity Analysis Input Parameters
3. External Flood Corrective Action Plan

bc:

Mr. Luis Reyes
USNRC – Region II
Sam Nun Atlanta Federal Center
Suite 23T85, 61 Forsyth St., SW
Atlanta, GA 30303-8931

Mr. J.F. Stang, Project Manager
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Mail Stop O-8 G9A
Washington, DC 20555

Mr. Andrew Sabisch
NRC Senior Resident Inspector
Oconee Nuclear Station

Susan E, Jenkins, Manager, Infectious and Radioactive Waste Management
Bureau of Land and Waste Management
Department of Health & Environmental Control
2600 Bull Street, Columbia, SC 29201

Attachment 1

Technical Justification for Inundation Study Input Parameters

Introduction

This document contains the justification for those breach parameters used in the inundation analysis following a postulated failure of the Jocassee Main Dam, as requested by the Nuclear Regulatory Commission. The intent of the inundation analysis is to determine the water levels at the Oconee Nuclear Plant and the effect of these levels on the safe operation of the plant. Failure of the Jocassee Main Dam is postulated for analysis purposes only; the results of the inundation analysis do not imply a likelihood of occurrence.

The recommended breach parameters are presented first for the Jocassee Main Dam and the downstream Keowee impoundment structures. The justifications for the recommended parameters are presented afterward, starting on page 2.

Jocassee Parameters

- Reservoir Elevation: 1110 ft. MSL
- Bottom Breach Elevation: 800 ft. MSL
- Bottom Breach Width: 425 ft.
- Side Slopes, West Slope: (1.55:1); East Slope: (0.7:1)
- Time to Failure: 2.8 hrs
- Piping Elevation: 1020 ft. MSL
- Failure Progression: Sine Wave

Keowee Parameters

Keowee Reservoir Elevation: 800 ft. MSL

West Saddle Dam

- Bottom Breach Elevation: 795 ft. MSL
- Bottom Breach Width: 1680 ft.
- Gate Slope, both sides of opening: 0:1
- Time to Failure: 0.5 hrs.
- Overtopping Trigger: 817 ft. MSL (2 ft. above crest)
- Failure Progression: Sine Wave

Main Dam

- Bottom Breach Elevation: 670 ft. MSL
- Bottom Breach Width: 500 ft.
- Side Slopes, both sides of breach: (1:1)
- Time to Failure: 2.8 hrs.
- Overtopping Trigger: 817 ft. MSL (2 ft. above crest)
- Failure Progression: Sine Wave

Attachment 1
Technical Justification for Inundation Study Input Parameters

Oconee Intake Dike

- Bottom Breach Elevation: 715.5 ft. MSL
- Bottom Breach Width: 200 ft.
- Side Slopes, both sides of breach: (1:1)
- Time to Failure: 0.9 hrs.
- Overtopping Trigger: 817 ft. MSL (2 ft. above crest)
- Failure Progression: Sine Wave

Little River Dam

- Bottom Breach Elevation: 670 ft. MSL
- Bottom Breach Width: 290 ft.
- Side Slopes, both sides of breach: (1:1)
- Time to Failure: 1.9 hrs.
- Overtopping Trigger: 817 ft. MSL (2 ft. above crest)
- Failure Progression: Sine Wave

Justification

Jocassee Parameters

Reservoir Elevation

Due to the degree to which the Jocassee reservoir elevation is controlled, the full pond reservoir elevation, 1110 ft. Mean Sea Level (MSL) was adopted. The normal operating range of the reservoir is between 1108 and 1110 ft. MSL. Should the level of the reservoir reach levels above 1108 ft. MSL, the Duke Hydro Department would take steps to prevent the level rising above 1110 ft. MSL. No concurrent probable maximum flood level was assumed.

Bottom Breach Elevation

Available technical papers do address bottom breach elevation. Mtalto and Lia[1] note that the final breach depth is approximately 70% of the dam height. From Duke drawing J-26, the height of Jocassee from the crest to the foundation is 375 ft. (1125 ft. -750 ft.). Taking 70% of the height and subtracting from the crest elevation equals 862.5 ft. MSL. A bottom elevation of 800 ft. MSL would then be conservative. Consideration was given to the material of the dam 'dropping out' of the inundation flow and thus limiting the breach bottom elevation. The extent to which the material would 'drop out' out was difficult to

[1] Mtalto, Malisa J, University of Dar Es Salaam- Tanzania; Lia, Leif, Norwegian University of Science and Technology, "Physical Hydraulic Modeling and Non-Cohesive Homogeneous Embankment Exposed to Through and Over Flows", Copyright PennWell Corp., 2009 Water Power Conference.

quantify because the majority of Jocassee is composed of random rock, and the degree to which the material would 'drop out' is related to the mass of the material, which would also be random. Thus the breach bottom elevation was conservatively determined to be 800 ft. MSL. To compensate for any material 'dropping out' and in turn affecting the flow, Manning's n was assigned a higher value immediately downstream of Jocassee in the HEC-RAS input.

<u>Breach Size</u>

There are a number of technical papers available that address breach size. The components that define the breach size are the bottom breach width (or alternatively the average breach width), the bottom breach elevation (with the crest elevation of the dam) and the breach side slopes. The justification for the bottom breach elevation was addressed above. The breach size is used as an input into HEC-RAS. It should be noted that the breach size is estimated based on historic dam failure data for the final or ultimate breach dimensions, which are the result of passing the complete breach hydrograph through the breach section. Chauhan et. al.[2] noted, "Observations of actual dam failures show that breaches continue to grow during the falling limb of the hydrograph. Thus, the peak breach flow rate would not be expected to occur when the breach size is at its maximum." HEC-RAS grows the breach to its final shape over the breach formation time and stops growing after that time, irrespective of the decreasing portion of the breach hydrograph still passing through the breach. Thus, when using the HEC-RAS breach model, the timing of the peak breach flow rate will usually coincide with the maximum breach size. Chauhan et. al. continue "Therefore, it is commonly understood that the use of breach parameters obtained from empirical approaches will lead to significant overestimates of peak breach flow rates, at least for dams with large reservoirs," such as the case for Jocassee. "This bias is expected to occur for all empirical breach parameter estimation approaches that use the final breach width and final breach formation time."

Thus, a balanced approach must be used that evaluates the peak break flow rates versus the postulated breach size. Froehlich[3] provides an empirical approach for estimating the average breach width and the peak flow rate. The estimated average breach width can then be used to determine the postulated breach size. The postulated breach size can be used as input to HEC-RAS and a peak flow rate calculated. By comparing the HEC-RAS calculated peak

---

[2] Chauhan, Sanjay S. (Research Assistant Professor of Civil and Environmental Engineering, Utah State University), Bowles, David S. (Professor of Civil and Environmental Engineering, Utah State University), and Anderson, Loren R. (Professor and Head of Civil and Environmental Engineering, Utah State University) "Do Current Breach Parameter Estimation Techniques Provide Reasonable Estimates For Use in Breach Modeling?"

[3] Froehlich, David C, "Embankment Dam Breach Parameters and Their Uncertainties" Journal of Hydraulics Engineering, Vol. 134, No. 12, December 1, 2008, American Society of Civil Engineering.

Attachment 1

Technical Justification for Inundation Study Input Parameters

flow rate with the estimated peak flow rate from Froehlich, an adjusted breach size can be determined.

The average breach width estimated by Froehlich is calculated by the following formula:

$$B = .27\, k_o\, V_w^{.33} \qquad (1)$$

Where k_o is a factor for the method of failure, and equals 1.0 for piping failures. V_w is the volume of water in the reservoir at the time of failure, in cubic meters.

The volume of water in the Jocassee reservoir at full pond conditions (1110 ft. MSL) is 1,160,296.0 acre-ft. This converts to a value of 1,431,203,905 cubic meters. Using this volume, B = 284.0 m, or approximately 930 ft. Using a 0.7 to 1 side slope as recommended by Froehlich for non overtopping failure modes, the bottom breach width equates to approximately 702.5 ft.

Froehlich[4] also predicts the peak flow through the breach by the following equation:

$$Q_p = .607\, V_w^{.295}\, H_w^{1.24} \qquad (2)$$

Where V_w is the volume of water in the reservoir at the time of failure in cubic meters and H_w is the height of the water above the breach in meters.

Using the volume calculated before, and noting that the height of the water above the breach is approximately 94.5 m, the peak flow is calculated as 85,819.8 cubic meters per second (m^3/s) or 3,030,699 cubic feet per second (cfs).

As noted, three Jocassee breach sizes were evaluated by HEC-RAS for input to the 2D model. These three cases are as follows:

<u>Case 1</u>

- Reservoir Elevation: 1110 ft. MSL
- Bottom Breach Elevation: 800 ft. MSL
- Bottom Breach Width: 250 ft.
- Time to Failure: 2.6 hrs
- Side Slopes, both sides of breach: (1:1)
- Piping Elevation: 1020 ft. MSL
- Failure Progression: Sine Wave

[4] Froehlich, David C, 1995b. "Peak Outflow from Breach Embankment Dam", Journal of Water Resources Planning and Management, Vol. 121, No. 1, p. 90-97

Attachment 1

Technical Justification for Inundation Study Input Parameters

Case 2

- Reservoir Elevation: 1110 ft. MSL
- Bottom Breach Elevation: 800 ft. MSL
- Bottom Breach Width: 425 ft.
- Side Slopes, West Slope: (1.55:1); East Slope: (0.7:1)
- Time to Failure: 2.8 hrs
- Piping Elevation: 1020 ft. MSL
- Failure Progression: Sine Wave

Case 3

- Reservoir Elevation: 1110 ft. MSL
- Bottom Breach Elevation: 800 ft. MSL
- Bottom Breach Width: 600 ft.
- Side Slopes, West Slope: (1.55:1); East Slope: (0.7:1)
- Time to Failure: 3.0 hrs
- Piping Elevation: 1020 ft. MSL
- Failure Progression: Sine Wave

The HEC-RAS results for these three cases indicated a peak flow rate as follows:

Case 1 – 4,760,000 cfs

Case 2 – 5,440,000 cfs

Case 3 – 6,300,000 cfs

A comparison of the HEC-RAS calculated peak breach flow rates with the Froehlich estimated peak flow rate indicates that HEC-RAS, with the given input breach sizes, over predicts the peak flow rates by as much as 200% (Case 3). Since the HEC-RAS peak flow rate for the Case 2 breach size provides a substantial margin, when compared to the Froehlich estimated peak flow rate, the Case 2 breach size was adopted.

Side Slopes

Froehlich recommends 0.7:1 side slopes for the final breach shape. In order to incorporate the probable piping failure mode on the west abutment, the side slope of 1.55:1 was adopted for the west slope. The east slope was adopted to be 0.7:1 in accordance with Froehlich. These side slopes result in a larger projected breach area than if the Froehlich recommended side slopes were used for both side slopes, and therefore are conservative.

Attachment 1
Technical Justification for Inundation Study Input Parameters

Time to Failure

Available technical papers do address time to failure. Time to failure is defined as the time between the onset of piping to the attainment of the full breach size. Froehlich estimated a time to failure of 2.2 hrs. Walder and O'Connor[5] estimated a time to failure of 3.04 hrs. A value of 2.6 hours was used for Case 1. This is the average of the Froehlich estimated time to failure and the Walder and O'Connor estimated time to failure. A value of 3.0 hours was used for Case 3. For Case 2, a value of 2.8 hours was used. The justification for this value recognizes that breach size and time to failure are related. As such, a larger breach would have a longer time to failure versus a smaller breach. The value of 2.8 hours was adopted, since the Case 2 breach size is larger than the Case 1 breach size, and thus the time to failure for Case 2 is slightly larger than the Case 1 time to failure.

Initial Piping Elevation

A piping elevation of 1020 ft. MSL was adopted to coincide with the elevation of the actual seepage on the West Abutment.

Failure Progression

Available technical papers do not address failure progression. The sine wave progression was adopted due to the more reasonable representation of the material loss through the breach provided by the sine wave progression versus the linear progression. The sine wave progression predicts a gradual increase in material loss as the breach begins to develop with a more pronounced increase as the breach grows before tapering off to its maximum size. The sine wave method of progression is more conservative considering a piping breach that would result in a rapid collapse of the crest of the Jocassee dam as the pipe develops.

Keowee Parameters[6]

Reservoir Elevation

Due to the degree to which the Keowee reservoir elevation is controlled, the full pond reservoir elevation, 800 ft. MSL was adopted. No concurrent probable maximum flood level was assumed.

---

[5] Walder, Joseph S., Cascades Volcano Observatory, United States Geological Survey, Vancouver, Washington; and O'Connor, Jim E., United States Geological Survey, Portland, Oregon. "Methods for Predicting Peak Discharge of Floods Caused by Failure of Natural and Constructed Earthen Dams", Water Resources Research, Vol. 33, No. 10, Pages 2337-2348, October 1997.

[6] All of the Keowee impoundment structures, e.g. the Main Dam, the West Saddle Dam, the Oconee Intake Dike, and the Little River Dam are of similar construction and material.

Attachment 1
Technical Justification for Inundation Study Input Parameters

West Saddle Dam

Bottom Breach Elevation

The adopted bottom breach elevation is based on the actual toe elevation (795 ft. MSL) of the majority of the dam. See Duke drawing K-30.

Bottom Breach Width

The adopted bottom breach width of 1680 ft. is based on the assumption that ~80% of the west saddle dam fails following overtopping.

Side Slopes

Given the relatively short size of the structure (20 ft. high), the breach side slopes are assumed to be vertical. Side slopes for a structure of this height is not a significant parameter.

Time to Failure

The adopted time to failure of 0.5 hours is based on the assumption that the dam will rapidly fail once overtopped due to rapidly increasing and high velocity flow over the downstream face of the dam (See discussion under Overtopping Trigger, below)

Overtopping Trigger

The adopted overtopping trigger of 2 ft. is based on several references and conclusions reached after completion of the initial HEC-RAS studies. The Agricultural Research Service, Hydraulic Engineering Research Unit, Stillwater, Oklahoma conducted actual scale model tests and references a static overtopping depth of 2 ft. Jamieson and Ferentchak[7] noted that breach development at an earthen dam would begin at about one foot of overtopping depth. A conservative value of 2 ft. was adopted based on these references and additional HEC-RAS model results that indicated velocities at the crest and at the downstream face of the dam were 2-4 times the accepted velocity exposure limits for steep grassed slopes.

Failure Progression

Sine Wave failure progression adopted for all embankment structures, see justification given under Jocassee parameters.

[7] Jamieson, Stephen L. and Ferentchak, James A. "Using Erosion Rate to Refine Earth Dam Breach Parameters"

Attachment 1

Technical Justification for Inundation Study Input Parameters

Main Dam

Bottom Breach Elevation

The adopted bottom breach elevation is based on the normal level of the Keowee tailrace of 670 ft. MSL. The actual prepared bed elevation at the Keowee main dam is approximately 650 ft. MSL.

Bottom Breach Width

Using Froehlich, the estimated bottom breach is 1028 ft., well beyond the actual width of the Keowee Main Dam. The actual bottom width between the abutments of 500 ft. was adopted.

Side Slopes

Side slopes were based on Froehlich (1:1) for an overtopping failure mode.

Time to Failure

The adopted time to failure is based on a proportional factor applied to the Froehlich estimated time to failure value of 5 hrs. This approach recognizes that time to failure and breach size parameters are related. The proportional factor is equal to the average breach width for a 1:1 side slope breach, with the aforementioned bottom breach width of 500 ft. with a height of 145 ft. (815 – 670) divided by the average breach width estimated by Froehlich. The average breach width calculated based on the above noted parameters is 645 ft. Froehlich estimated an average breach width of 1173 ft. Thus, the proportional factor is 0.571 (645/1173). Multiplying the proportional factor by the Froehlich estimated time to failure of 5 hours yields 2.8 hours. The main dam's time to failure is used as a base for determining the time to failure of the remaining earthen structures surrounding the Keowee reservoir due to the similarity of the material used in the dams and the common construction practices.

Overtopping Trigger

See overtopping trigger discussion under West Saddle Dam.

Failure Progression

Sine Wave failure progression adopted for all embankment structures, see justification given under Jocassee parameters.

Attachment 1

Technical Justification for Inundation Study Input Parameters

Oconee Intake Dike

Bottom Breach Elevation

The adopted bottom breach elevation is based on the actual toe elevation (715.5 ft. MSL) of the portion of the intake dike facing east. The downstream side of the dike facing north is assumed to be armored in the model and thus failure of this portion of the dike due to overtopping is not assumed.

Bottom Breach Width

Based on Froehlich, the estimated bottom breach width would be 1028 ft., well beyond the actual width of the Oconee Intake Dike. Due to physical constraints, the adopted width is 200 ft.

Side Slopes

Side slopes were based on Froehlich (1:1) for an overtopping failure mode.

Time to Failure

The adopted time to failure is based on a proportional factor applied to the projected area of the Keowee Main Dam breach. The projected area of the Keowee Main Dam breach referenced above is 93,525 sq. ft. The projected area of the adopted Oconee Intake Dike breach, described above, is 29,800 sq. ft. The proportional factor is then 29,800/93,525 = 0.3186. Recognizing that the time to failure is proportional to the breach size, and the adopted time to failure of the Keowee Main Dam is 2.8 hrs, the time to failure of the Oconee Intake Dike is then 0.3186 x 2.8 hrs. = 0.9 hrs.

Overtopping Trigger

See overtopping trigger discussion under West Saddle Dam.

Failure Progression

Sine Wave failure progression adopted for all embankment structures, see justification given under Jocassee parameters.

Little River Dam

Bottom Breach Elevation

The adopted bottom breach elevation is based on the normal level of the Keowee tailrace of 670 ft. MSL.

Bottom Breach Width

Froehlich estimated a bottom breach width of 1028 ft., far beyond the actual width of the Little River main dam. Due to physical constraints, the adopted width is 290 ft.

Attachment 1
Technical Justification for Inundation Study Input Parameters

Side Slopes

Side slopes based on Froehlich (1:1) for an earthen dam.

Time to Failure

The adopted time to failure is based on a proportional factor determined from the projected area of the Keowee Main Dam breach. The projected area of the Keowee Main Dam breach referenced above is 93,525 sq. ft. The projected area of the adopted Little River Dam breach, described above, is 63,075 sq. ft. The proportional factor is then 63,075/93,525 = 0.6744. Recognizing that the time to failure is proportional to the breach size, and the adopted time to failure of the Keowee Main Dam is 2.8 hrs, the time to failure of the Oconee Intake Dike is then 0.6744 x 2.8 hrs. = 1.9 hrs.

Overtopping Trigger

See overtopping trigger discussion under West Saddle Dike.

Failure Progression

Sine Wave failure progression adopted for all embankment structures, see justification given under Jocassee parameters.

Attachment 2

Technical Justification for Sensitivity Analysis Input Parameters

Introduction

This document contains a technical discussion of the selected parameters and their variation used in the HEC-RAS sensitivity analyses. Justification of the selected parameters and the variation of those parameters is also included. The intent of the sensitivity analyses was to determine those parameters that would have a significant effect on the results of the HEC-RAS inundation analysis following a postulated failure of the Jocassee Main Dam. Significance in this case is defined as affecting the potential water level at the Oconee Nuclear Site. The significance of the effects was determined by varying those selected parameters and gauging the results. 101 HEC-RAS analysis cases were evaluated for sensitivity of the parameters. The Jocassee failure is postulated for analysis purposes only; the results of the inundation analysis do not imply a likelihood of occurrence.

Literature Search

A number of authors have written papers regarding breach parameters. The majority of the papers are based on historical dam failure data and the analysis of that data. Referenced papers are given as footnotes throughout this document. A review of the literature resulted in a suggested list of parameters deemed important by the authors of those technical papers. In general, the parameters of importance are those that describe the shape of the breach and the timing of the breach. The timing of the breach or the time to failure of the breach is defined as the time beginning at initial erosion of the downstream face of the dam to the attainment of the full breach size.

Parameter Selection

In addition to the selection of the breach size and time to failure parameters, other parameters were selected based on the analysis assumptions. For example, since the issue regards a postulated 'sunny day' failure of the Jocassee Dam, only those failures modes that could occur were considered, given the design and construction of Jocassee. Since Jocassee is designed to pass a Probable Maximum Flood (PMF) without overtopping the main dam, overtopping failures were not considered. Likewise seismic induced failures were not considered since Jocassee is seismically designed. A Potential Failure Modes Analysis (PFMA) completed in 2004 listed a piping failure through the West Abutment as the predominate failure mode affecting the structural stability of the Jocassee Dam. As such, the inundation analyses focused on the initiation of the Jocassee breach caused by a piping type failure. Based on this, the elevation of the initial piping became an important parameter that should be considered.

Other parameters were selected based on site details. For example, the Oconee Nuclear site is protected from normal flooding by the impoundment capability of the Keowee Main Dam, the West Saddle Dam, and the Oconee Intake Dike. Early analysis indicated that these structures would be overtopped following a postulated breach of the Jocassee Dam. Therefore, it is important to understand whether these structures fail, how they fail, and the timing of their failure. The other Keowee reservoir impoundment structure, the Little River Dam is located several miles away from the Oconee Nuclear Site, and as such, does not directly protect the site from normal flooding. However, if it were to fail, it could help relieve water away from the Oconee Nuclear Site. Therefore, it is important to understand whether the structure failed, how fast it failed, and the timing of the failure.

Attachment 2

Technical Justification for Sensitivity Analysis Input Parameters

Other parameters were chosen based on expert opinion. Manning's n is an example of this. Manning's n is a measure of the roughness, or friction, in the flow channel. A small Manning's n in the channel downstream of Jocassee would result in a faster conveyance of water toward the Keowee impoundment structures and the Oconee Nuclear Site than a large Manning's n. Similarly, a small Manning's n in the channel downstream of Keowee and the site would result in a faster conveyance of water away downstream of the Oconee Nuclear Site than a large Manning's n. Therefore, Manning's n became an important parameter for consideration.

The parameters were divided into four groups. Group 1 included all of the parameters that affect the postulated breach size at Jocassee, the time to failure, the failure initiator, and the progression of the failure. Group 2 included all of the parameters that affect the failure of the Keowee Main Dam, that is, the postulated breach size, the time to failure, and the failure initiation point. Also, included in Group 2 is whether the West Saddle Dam failed, and whether any bypass flow around the Keowee Main Dam occurred. Group 3 included the timing of the failure of the other Keowee reservoir impoundment structures. The breach size(s) and the failure initiation point of the other Keowee reservoir impoundment structures were held constant for all analysis cases. This was done to limit the analysis cases evaluated to a manageable number. Group 4 regards the possible combinations of Manning's n in the various parts of the flow basin under consideration. Each of these parameter groups are described in more detail below, with a justification for the selected parameter and the variation of the parameter.

Group 1 - Jocassee Parameters

Reservoir Elevation

The reservoir elevation and hence the water volume in the reservoir are important for determining the potential breach size, the timing of the failure, and the peak flow out of the breach. The first HEC-RAS analysis duplicated the 1992 Emergency Action Plan (EAP) DAMBRK analysis, and accordingly the reservoir elevation was held at 1108 ft. Mean Sea Level (MSL). For the remaining 100 HEC-RAS analysis runs the Jocassee reservoir elevation was held at 1110 ft. MSL. The reservoir level was held at this level due to the problem definition. As discussed before the failure postulation of Jocassee is noted as a 'sunny day' failure. Thus PMF induced or seismic induced failures were not considered. Also, due to the degree to which the Jocassee reservoir elevation is controlled, the full pond reservoir elevation of 1110 ft. MSL is reasonable. The normal operating range of the reservoir is between 1108 and 1110 ft. MSL. Should the level of the lake reach levels above 1108 ft. MSL, the Duke Hydro Department would take steps to prevent the level rising above 1110 ft. MSL. In addition, the elevation of the top of the Jocassee spillway gates is 1110 ft. MSL. Any rise in the reservoir beyond this elevation would result in surcharge flow over the spillway gates.

Attachment 2
Technical Justification for Sensitivity Analysis Input Parameters

Bottom Breach Elevation

The general shape of a postulated breach is trapezoidal. The trapezoid is defined by the base width, the height, and the inclination of the sides. Alternatively, the average breach width may be substituted for the base width. The height of the trapezoid is defined as the difference in the crest of the dam and the bottom breach elevation. The bottom breach elevation was varied from the nominal excavation depth of the dam during construction to an upper limit based on empirical evidence provided by Mtalto and Lia[8]. Mtalto and Lia note that the final breach depth of the breach is approximately 70% of the dam height. From Duke drawing J-26, the height of Jocassee from the crest to the foundation is 375 ft. (1125 ft. -750 ft.). Taking 70% of the height and subtracting from the crest elevation equals 862.5 ft. MSL. The variation of this parameter ranges from the lower elevation of 750 ft. MSL to an upper elevation of 850 ft. MSL, in 50 foot increments. The upper elevation is bounded by the Mtalto and Lia elevation of 862.5 ft. MSL.

Bottom Breach Width

There are a number of technical papers available that address breach width. Froehlich[9] provides an empirical approach for estimating the average breach width. The estimated average breach, along with the side slopes, and breach height can then be used to determine the bottom breach width.

The average breach width estimated by Froehlich is calculated by the following formula:

$$B = .27\, k_o\, V_w^{.33} \qquad (1)$$

Where k_o is a factor for the method of failure, and equals 1.0 for piping failures.
V_w is the volume of water in the reservoir at the time of failure, in cubic meters.

The volume of water in the Jocassee reservoir at full pond conditions (1110 ft. MSL) is 1,160,296.0 acre-ft. This converts to a value of 1,431,203,905 cubic meters. Using this volume, B = 283.6 m, or approximately 930 ft. Using a 0.7 to 1 side slope as recommended by Froehlich for non overtopping failure modes, the bottom breach width equates to approximately 702.5 ft.

---

[8] Mtalto, Malisa J, University of Dar Es Salaam- Tanzania; Lia, Leif, Norwegian University of Science and Technology, "Physical Hydraulic Modeling and Non-Cohesive Homogeneous Embankment Exposed to Through and Over Flows", Copyright PennWell Corp., 2009 Water Power Conference.

[9] Froehlich David C, "Embankment Dam Breach Parameters and Their Uncertainties" Journal of Hydraulics Engineering, Vol. 134, No. 12, December 1, 2008, American Society of Civil Engineering.

Attachment 2

Technical Justification for Sensitivity Analysis Input Parameters

A review of the actual dimensions of the Jocassee dam indicate that bottom breach width calculated via Froehlich is not possible at the bottom breach elevation of 800 ft. MSL. Per Duke drawing J-26, the maximum possible width at elevation 800 ft. MSL is 600 ft. At a slightly higher elevation of 850 ft. MSL, the maximum width is 650 ft.

The original 1992 EAP DAMBRK analysis used a bottom breach width of 250 ft. So the lower bound width was set to this value, and the upper bound set at 650 ft. The upper bound value of 650 ft. was used only when the bottom breach elevation was selected as 850 ft. MSL. A value of 625 ft. was used only when the bottom breach elevation was selected as 825 ft. MSL. The variation in this bottom breach width parameter ranges from 250 ft. to 650 ft., with specified values of 250, 425, 500, 600, 625 and 650 ft.

<u>Side Slopes</u>

As discussed before, the side slopes help define the breach trapezoidal area. Froehlich estimated a 0.7 horizontal to 1 vertical (0.7:1) side slopes for the final breach shape for non-overtopping failure modes. As noted before, a piping initiated failure was the only failure mode considered for the 'sunny day' failure of Jocassee, so the 0.7:1 side slope would be appropriate.

Less steep side slopes were explored in the sensitivity analyses since they resulted in larger projected breach areas than the 0.7:1 side slope trapezoid. It was also recognized that in order to capture the location of the West Abutment seepage, the west side slope must be raked more to the horizontal. Since the East Abutment seepage is significantly less in magnitude than the West Abutment seepage, such a modification of the east side slope was not necessary. To capture the west side seepage, the slope was raked to 1.55:1. This is essentially the slope of the natural ground of the west side of the dam. Therefore, in order to capture the difference in seepage between the two abutments, the combinations of side slopes for the east and west sides of the postulated breach were also varied.

For the cases where the side slopes were equal on both sides of the postulated breach, the slope values of 0.9:1, 1:1, and 1.5:1 were used. For the cases where the side slopes were unequal, the slope combinations of 1.55:1 (west) and 1:1 (east); and 1.55:1 (west) and 0.7:1 (east) were used. These values are fully justified based on the Froehlich estimated value of 0.7:1.

<u>Time to Failure</u>

The time to failure is defined as the time between the onset of piping to the attainment of the full breach size. Given the size of the Jocassee reservoir and the previously estimated breach height, Froehlich estimates a time to failure of 2.2 hrs. Walder and O'Connor[10] estimate a

---

[10] Walder, Joseph S., Cascades Volcano Observatory, United States Geological Survey, Vancouver, Washington; and O'Connor, Jim E., United States Geological Survey, Portland, Oregon. "Methods for Predicting Peak Discharge of Floods Caused by Failure of Natural and Constructed Earthen Dams", Water Resources Research, Vol. 33, No. 10, Pages 2337-2348, October 1997.

time to failure of 3.04 hrs. The sensitivity study explored time to failure values on both the low side and high side of these values. The time to failure parameter was varied from 1 hour to 5 hours in 1 hour increments. Also included were the time to failure values of 2.6 and 2.8 hours.

Piping Elevation

During the literature search, no technical papers were found that predicted the probable piping elevation. The HEC-RAS software package allows input of this parameter, so variation in this parameter was pursued. The 1992 EAP DAMBRK analysis assumed a piping elevation of 940 ft. MSL. As noted before, the sensitivity study attempted to capture the potential failure mode associated with seepage through the West Abutment. The West Abutment seepage is generally located at elevation 1020 ft. MSL. Thus, the initial piping elevation was varied at the specific values of 940 ft. and 1020 ft. MSL. The difference in elevation between the two values is significant enough to discern differences in the predicted water level at the Oconee Nuclear Site.

Failure Progression

During the literature search, no technical papers were found that predicted the probable failure progression. The HEC-RAS software package allows input of this parameter, so variation in this parameter was pursued. The linear and the sine wave failure progressions were explored in the sensitivity analysis to understand if this is a significant parameter in regards to the predicted water level at the Oconee Nuclear Site.

Group 2 - Keowee Parameters

Reservoir Elevation

The reservoir elevation of the Keowee Reservoir was held constant at 800 ft. MSL for all of the sensitivity HEC-RAS runs. This is justified due to the degree to which the Keowee Reservoir elevation is controlled. In addition, since the Jocassee failure is termed as a 'sunny day' failure, no concurrent probable maximum flood level was assumed for the Keowee Reservoir.

Main Dam Bottom Breach Elevation

The Keowee Main Dam is postulated to fail as the inundation flow following a postulated failure of Jocassee overtops the structure. Bottom breach elevations of 670 ft. MSL and 700 ft. MSL respectively were explored in the sensitivity analysis. The 670 ft. MSL value corresponds to the base of the Keowee dam. The value of 700 ft. MSL is bounded by the Mtalto and Lia noted limitation on the final breach height equaling 70% of the dam height. The height of the Keowee Main Dam is 145 ft. (Crest elevation of 815 ft. MSL – Base elevation of 670 ft. MSL). So, 70% of the height would be 101.5 ft. Subtracting this value from the crest elevation equals 713.5 ft. MSL.

Attachment 2

Technical Justification for Sensitivity Analysis Input Parameters

Bottom Breach Width

As noted before, there are a number of technical papers available that address breach width. Froehlich provides an empirical approach for estimating the average breach width. The estimated average breach, along with the side slopes, and breach height can then be used to determine the bottom breach width.

Froehlich estimated an average breach width of 1173 ft. A review of drawing K-30 indicates that the physical size of the dam limits the maximum average breach width to approximately 897 ft. In addition the same drawing indicates that the maximum bottom breach width is approximately 500 ft.

Bottom breach widths of 500 and 650 ft. respectively were explored in the sensitivity analysis. The 500 ft. value was used in combination with the breach bottom elevations of 670 and 700 ft. MSL respectively. However, the 650 ft. value was only used in combination with the breach bottom elevation of 700 ft. MSL due to the physical constraints of the Main Dam.

These bottom breach widths are justified by historical information that earthen dams fail once overtopped to the extent predicted.

Side Slopes

As noted before, the side slopes help define the breach trapezoidal area. Froehlich estimated a 1 horizontal to 1 vertical (1:1) side slope for the final breach shape for overtopping failure modes. Flatter side slopes were also explored in the sensitivity analysis. The use of the flatter slopes allowed postulation of larger projected area breaches of the Keowee Main Dam. In some cases unequal side slopes were explored. This again allowed postulation of larger projected area breaches of the Keowee Main Dam.

For the cases where the side slopes were equal on both sides of the postulated breach, the slope values of 1:1 and 1.5:1 were used. For the cases where the side slopes were unequal, the slope combinations of 3.45:1 (west) and 2.03:1 (east) were used.

The slope value of 1:1 is fully justified by the Froehlich estimated side slope given above. The larger side slopes and hence larger breach sizes were explored to understand the effect of allowing more flow through the Keowee Main Dam breach and its possible effect on the level of water at the Oconee Nuclear site.

Time to Failure

The Froehlich estimated time to failure value of the Keowee Main Dam is 5 hrs. Failure times of less than 5 hours were explored in the sensitivity analysis. Time to failures values of 2 hours, 2.4 hours, 2.8 hours, and 4 hours were explored in the sensitivity analysis. This approach recognizes that time to failure and breach size parameters are related. Since the possible size of the Keowee Main Dam breach is less than the Froehlich estimated average breach, a proportional factor, equal to the average breach width for a 1:1 side slope breach, with a 500 ft. bottom breach width, and a height of 145 ft. (815 ft. – 670 ft.) divided by the average breach width estimated by Froehlich was determined. The average breach width

calculated based on the above noted parameters is 645 ft. Froehlich estimated an average breach width of 1173 ft. Thus, the proportional factor is 0.571 (645/1173). Multiplying the proportional factor by the Froehlich estimated time to failure of 5 hours yields 2.8 hours.

Overtopping Trigger

The Agricultural Research Service, Hydraulic Engineering Research Unit, Stillwater, Oklahoma conducted actual scale model tests and references a static overtopping depth of 2 ft. Jamieson and Ferentchak[11] noted that breach development at an earthen dam would begin at about 1 ft. of overtopping depth. Overtopping failure trigger at elevations 815.5 (0.5 ft. above crest) and 817 (2 ft. above the crest) ft. MSL respectively were explored in the sensitivity study. The 815.5 ft. MSL elevation was explored because it would result in a shorter failure time of the Keowee Main Dam (all other parameters held constant) and could result in higher water levels at the Oconee Nuclear Site. The 817 ft. MSL elevation is justified by Jamieson and Ferentchak.

Failure Progression

See the discussion of the Jocassee parameters.

Failure of West Saddle Dam

Cases were explored in the sensitivity analysis assuming the West Saddle Dam failed after being overtopped and cases were explored in analysis assuming the West Saddle Dam did not fail. Both cases were explored to understand whether failing the West Saddle Dam would affect the water level at the Oconee Nuclear Site, either from a backwater affect in the Keowee tailrace if the West Saddle Dam failed or by increased overtopping level at the Oconee Intake Dike if the West Saddle Dam did not fail. For those cases where the West Saddle Dam was assumed to fail, the following parameters to model the failure were used:

- Bottom Breach Elevation: 795 ft. MSL
- Bottom Breach Width: 1680 ft.
- Gate Slope, both sides of opening: 0:1
- Time to Failure: 0.5 hrs.
- Overtopping Trigger: 817 ft. MSL (2 ft. above crest)
- Failure Progression: Sine Wave

The justification for these failure parameters are listed below:

Bottom Breach Elevation

The adopted bottom breach elevation is based on the actual toe elevation (795 ft. MSL) of the majority of the dike.

[11] Jamieson, Stephen L. and Ferentchak, James A. "Using Erosion Rate to Refine Earth Dam Breach Parameters"

Attachment 2
Technical Justification for Sensitivity Analysis Input Parameters

Bottom Breach Width

The adopted bottom breach width of 1680 feet is based on the assumption that ~80% of the west saddle dike fails following overtopping.

Side Slopes

Given the relatively short size of the structure (20 ft. high), the breach side slopes are assumed to be vertical. Side slopes for a structure of this height is not a significant parameter.

Time to Failure

The adopted time to failure of 0.5 hours is based on the assumption that the dike will rapidly fail once overtopped due to rapidly increasing and high velocity flow over the downstream face of the dam.

Overtopping Trigger

See discussion for the Keowee Main dam

Bypass Flow

During a review of the predicted headwater elevations at the Keowee Main Dam, it was discovered that the high water elevations could result in water flowing through a topographically low area east of the Keowee Main Dam. Thus, cases were explored in the sensitivity analysis assuming that this flow did occur and cases were explored assuming that this flow did not occur. Both conditions were explored to understand the effect of the bypass flow on the water levels at the Oconee Nuclear Site.

Group 3 – Other Keowee Impoundment Structure Parameters

Oconee Intake Dike

In those cases where the Oconee Intake Dike was overtopped, a breach was assumed to develop on the portion of the dike facing east. The downstream side of the dike facing north is assumed to be armored in the model and thus failure of this portion of the dike due to overtopping is not assumed. The assumption that the dike would fail when overtopped is based on historical data that indicates that earthen dams fail when significantly overtopped. The time to failure parameter for the Oconee Intake Dike was explored in the sensitivity analysis. The estimated time to failure of the dike is based on a proportional factor determined from the projected area of the Keowee Main Dam breach. The projected area of the Keowee Main Dam breach referenced above is 93,525 sq. ft. The projected area of the Oconee Intake Dike breach defined by the parameters given below is 29,800 sq. ft. The proportional factor is then 29,800/93,525 = .3186. Recognizing that the time to failure is proportional to the breach size, and the estimated time to failure of the Keowee Main Dam is 2.8 hrs, the time to failure of the Oconee Intake Dike is then estimated to be 0.3186 x 2.8 hrs. = 0.9 hrs.

Attachment 2

Technical Justification for Sensitivity Analysis Input Parameters

The Oconee Intake Dike failure times of 0.8 hours, 0.9 hours, 1 hour, 1.2 hours, and 2 hours were explored in the sensitivity analysis. The intent of this exploration was to determine the sensitivity of the water level(s) at the Oconee Nuclear Site to the varying failure times.

The remaining failure parameters regarding the Oconee Intake Dike were held constant in the sensitivity analysis in order to limit the case perturbations. The failure parameters for the Oconee Intake Dike are as follows:

- Bottom Breach Elevation: 715.5 ft. MSL
- Bottom Breach Width: 200 ft.
- Side Slopes, both sides of breach: (1:1)
- Overtopping Trigger: 817 ft. MSL (2 ft. above crest)
- Failure Progression: Sine Wave

The justification for these failure parameters are listed below:

Bottom Breach Elevation

The adopted bottom breach elevation is based on the actual toe elevation (715.5 ft. MSL) of the portion of the intake dike facing east.

Bottom Breach Width

Froehlich estimated the bottom breach width of 1028 ft., well beyond the actual width of the Oconee Intake Dike. Due to physical constraints, the adopted width is 200 ft.

Side Slopes

Side slopes were based on Froehlich (1:1) for an overtopping failure mode.

Overtopping Trigger

See overtopping trigger discussion under Keowee Main Dam.

Failure Progression

See the discussion failure progression under the Jocassee Parameters.

Little River Dam

In those cases where the Little River Dam was overtopped, a breach was assumed to develop. The assumption that the dam would fail when overtopped is based on historical data that indicates that earthen dams fail when significantly overtopped. The time to failure parameter for the Little River Dam was explored in the sensitivity analysis. The estimated time to failure of the dam is based on a proportional factor determined from the projected area of the Keowee Main Dam breach. The projected area of the Keowee Main Dam breach referenced above is 93,525 sq. ft. The projected area of the Little River Dam breach, as defined by the parameters below, is 63,075 sq. ft. The proportional factor is then 63,075/93,525 = 0.6744. Recognizing that the time to failure is proportional to the breach size, and the estimated time

Attachment 2

Technical Justification for Sensitivity Analysis Input Parameters

to failure of the Keowee Main Dam is 2.8 hrs, the estimated time to failure of the Little River Dam is then 0.6744 x 2.8 hrs. = 1.9 hrs.

The Little River Dam failure times of 1 hour, 1.6 hours, 1.9 hours, 2.4 hours, and 4 hours were explored in the sensitivity analysis. The intent of this exploration was to determine the sensitivity of the water level(s) at the Oconee Nuclear Site to the varying failure times.

The remaining failure parameters regarding the Little River Dam were held constant in the sensitivity analysis in order to limit the case perturbations. The failure parameters for the Little River Dam are as follows:

- Bottom Breach Elevation: 670 ft. MSL
- Bottom Breach Width: 290 ft.
- Side Slopes, both sides of breach: (1:1)
- Overtopping Trigger: 817 ft. MSL (2 ft. above crest)
- Failure Progression: Sine Wave

The justification for these failure parameters are listed below:

Bottom Breach Elevation

The adopted bottom breach elevation is based on the normal level of the Keowee tailrace of 670 ft. MSL.

Bottom Breach Width

Froehlich estimated a bottom breach width of 1028 ft., far beyond the actual width of the Little River main dam. Due to physical constraints, the adopted width is 290 ft.

Side Slopes

Side slopes were based on Froehlich (1:1) for an overtopping failure mode.

Overtopping Trigger

See overtopping trigger discussion under Keowee Main Dam.

Failure Progression

Sine Wave failure progression adopted for all embankment structures, see justification given under Jocassee Parameters.

Group 4 – Manning's n

The original March 2009 HEC-RAS Model (Model Run 1) generally utilized a Manning's roughness coefficient (n value) of 0.035 for the main channel and 0.08 for overflow areas throughout the model. These values were repeated in Model Runs 2-63.

Manning's n value in most streams decreases with an increase in stage and discharge and generally approaches 0.02 in deep reservoirs. Additional model runs were performed with

Attachment 2
Technical Justification for Sensitivity Analysis Input Parameters

Manning's n values of 0.02 and 0.025 for the main reservoir channels for Jocassee, Keowee, and Hartwell reservoirs. Individual HEC-RAS river reach cross sections were reviewed and Manning's n values were revised from 0.035 to 0.02 (early runs) and 0.025 (later runs) for water depths in excess of 60 ft. Upstream reservoir headwaters and tributary n values remained at 0.035.

In addition, the Manning's n values were revised from 0.035 to 0.07 in the respective tailrace reaches below Jocassee Dam, Keowee Main Dam, ONS Intake Canal Dike, and Little River Dam to account for roughness associated with displaced dam breach material. The affected reach lengths below each dam were limited to the base width dimension of the respective dams, followed by a second base width dimension to allow transition from 0.07 to 0.02. For example, the base width of Jocassee Dam is approximately 1,300 ft. at elevation 800 ft. MSL. Therefore, the Jocassee tailrace would have a Manning's n value of 0.07 for a distance of 1,300 ft. below the downstream toe of the dam and an additional 1,300 ft. to transition from 0.07 to 0.02. The Manning's n transition was assumed linear. In general the Manning's n value revisions correspond with the following models:

- Model Run 1-63: 0.035
- Model Run 64-76: 0.02
- Model Run 77-101: 0.025

Attachment 3

External Flood Corrective Action Plan

Duke continues to aggressively address NRC concerns relative to external flood potential of the Oconee Nuclear Station. To date, the following actions have been completed:

- Extended the height of the Standby Shutdown Facility (SSF) flood walls and gate.
- Provided interim external flood mitigation guidance for the Technical Support Center (TSC) personnel.
- Converted the original 1992 DAMBRK model to the Army Corp of Engineer's Hydrologic Engineering Center – River Analysis System (HEC-RAS) software package.
- Completed sensitivity studies using HEC-RAS as described in Attachment 2.
- Completed 2D model studies as described in Attachment 1.

As an interim measure, Duke will reevaluate the external flood mitigation guidance completed in February of 2009 to incorporate current perspectives gained following the HEC-RAS sensitivity studies and the subsequent 2D inundation studies.

Duke is considering, based on perspectives gained during the HEC-RAS sensitivity studies and subsequent 2D inundation studies, potential modification(s) that include (but are not limited to) the following:

- Flood mitigation device located in the Oconee Intake Canal
- Flood wall located on the north rim of the Oconee Intake Canal Dike
- Assured integrity of portion of the Oconee Intake Canal Dike to preclude failure of the dike toward the Standby Shutdown Facility (SSF)
- Water retaining structure/wall/raised roadbed in swale adjacent to the World of Energy Visitor's Center.
- Downstream modifications (earth removal) to flood zone to smooth flow through the Keowee Tailrace.
- Protection of the SSF for higher inundation levels.

Potential modifications and the implications of those modifications on the site need to be evaluated in an orderly manner consistent with Duke's Engineering Change Program. Planning horizons for some of the larger scope projects under consideration could take several years. Duke has developed an expedited schedule for determining the scope of potential modifications and whether such modifications can be constructed. A list of the planning steps that describe the feasibility/constructability process for the potential modifications is provided below. Tentative dates are also included. Continuation of periodic status meeting(s) with the NRC will be scheduled as needed. Modifications deemed necessary will be implemented commensurate with the safety significance of the issue and prioritized relative to other significant risk reduction construction projects currently in progress at the Oconee Nuclear Station.

Attachment 3
External Flood Corrective Action Plan

1. Finalize design breach parameters. Before the scope of the project can be determined, a common understanding of those breach parameters that directly affect the results of the inundation analysis must be achieved between Duke and the NRC. In order to expedite the schedule, this must occur in the early stages of the project.

   Tentative completion date: 12/15/09.

2. Determine Level of Protection to be afforded for Structures, Systems and Components (SSC's) necessary to reach safe shutdown. This step first determines what structures, systems and components are to be protected and secondly, what level of protection should be afforded. Consideration will be given to those SSCs that need to withstand the inundation and those SSCs that aren't required for initial event mitigation but may be needed once the inundation has receded.

   Tentative completion date: 2/1/10.

3. Determine initial feasibility/constructability of potential alternatives and design concepts prioritized based on safety significance. Alternatives will be determined for each source of inundation (e.g., from overtopping the intake dike, from overtopping the swale, backwater from the tailrace) as they apply. Alternatives will then be combined into design concepts for further evaluation of safety benefits, complexity, and costs.

   Tentative completion date: 4/1/10.

4. Determine if design concepts can provide the needed level of protection for the selected SSC's. This involves reanalysis of the inundation models to understand the level of protection that the design concepts can provide. The design concepts will also be evaluated for the potential to create additional inundation flow paths.

   Tentative completion date: 5/1/10.

5. Evaluate the design concepts for their potential impact to other non flood related design requirements. Impacts to site security, plant operation and maintenance, and non-flood design functions will be evaluated.

   Tentative completion date: 7/1/10

6. Finalize feasibility/constructability of potential alternatives and design concepts and order in terms of safety benefits, complexity, and costs for presentation to Corporate Management.

   Tentative completion date: 10/1/10.

Attachment 3

External Flood Corrective Action Plan

7. Secure corporate approval for design and implementation. After selecting the appropriate design concept, this step will provide the results of the study and recommendations for design and implementation in order to obtain funding.

   Tentative Completion date: 11/1/10.

8. Set modification Design and Implementation schedules. This step establishes and communicates the modification(s) design schedules and implementation schedules.

   Tentative Completion date: 11/15/10.

Finally, depending on the modifications chosen and their implementation, the Oconee Updated Final Safety Analysis Report (UFSAR) will be updated as necessary, in accordance with the Duke Engineering Change Program.