Tennessee Valley Authority, 1101 Market Street, LP 5A, Chattanooga, Tennessee 37402-2801

October 17, 2008

10 CFR 52.79

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555

In the Matter of	)	Docket No. 52-014 and 52-015
Tennessee Valley Authority	)	

BELLEFONTE COMBINED LICENSE APPLICATION – RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION – SEISMIC DESIGN PARAMETERS

Reference: Letter from Brian Hughes (NRC) to Andrea L. Sterdis (TVA), Request for Additional Information Letter No. 110 Related to SRP Section 03.07.01 for the Bellefonte Units 3 and 4 Combined License Application, dated August 7, 2008.

Letter from Andrea L. Sterdis (TVA) to Document Control Desk (NRC), Bellefonte Combined License Application – Response to Request for Additional Information – Seismic Design Parameters, dated September 5, 2008.

This letter provides the Tennessee Valley Authority's (TVA) supplemental response to the Nuclear Regulatory Commission's (NRC) request for additional information (RAI) items included in the reference letter.

A response to each NRC request in the subject letter is addressed in the enclosure which also identifies any associated changes that will be made in a future revision of the BLN application.

If you should have any questions, please contact Tom Spink at 1101 Market Street, LP5A, Chattanooga, Tennessee 37402-2801, by telephone at (423) 751-7062, or via email at tespink@tva.gov.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on this 17th day of Oct, 2008.

JABailey

Jack A. Bailey
Vice President, Nuclear Generation Development

Enclosure
cc: See Page 2

D085
NRO

cc: (w/Enclosure)
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Enclosure
TVA letter dated October 17, 2008
RAI Responses

Responses to NRC Request for Additional Information letter No. 110 dated August 7, 2008
(6 pages, including this list)

Subject: Seismic design parameters in the Final Safety Analysis Report

RAI Number	Date of TVA Response
03.07.01-01	September 5, 2008
03.07.01-02	This letter – see following pages
03.07.01-03	September 5, 2008

Associated Additional Attachments / Enclosures	Pages Included
Attachment 03.07.01-02A	4 pages

NRC Letter Dated: August 7, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 03.07.01-02

3.7.2.12 Methods for Seismic Analysis of Dams

Add the following text to the end of DCD Subsection 3.7.2.12: (BLN COL 3.7-1)

> "The evaluation of existing and new dams whose failure could affect the site interface flood level specified in DCD Subsection 2.4.1.2, is included in Subsection 2.4.4."

RAI: The staff requires clarification about the seismic classification of dams and the analysis methods and acceptance criteria that have been applied in the "evaluation of existing and new dams whose failure could affect the site interface flood level specified in DCD Subsection 2.4.1.2,....." The staff requests the applicant to provide the following information for BLN COL 3.7-1:

(a) Are there any Seismic Category I dams associated with the Bellefonte site? If so, describe the analysis methods and acceptance criteria that have been applied to confirm they do not collapse under the GMRS.

(b) What organization has jurisdictional responsibility for the dams whose failure in an earthquake could affect the site flood level? Is there an established seismic design basis for these dams? If so, please describe it.

(c) In estimating the maximum site flood level, including seismic effects on these dams, have all dams been assumed to fail under the effects of the site GMRS? If not, describe in detail the technical basis for making determinations of complete failure, partial failure, and no failure under the effects of the site GMRS.

BLN RAI ID: 2212

BLN RESPONSE:

(a) There are no seismic Category I dams associated with the Bellefonte site.

(b) The Tennessee Valley Authority has jurisdictional responsibility for the majority of dams whose failure in an earthquake could affect the site flood level at BLN. Table 1 of this response lists dams upstream of Bellefonte including the dam owner. Tables 2.4.4-205 and 2.4.4-206 of the BLN Units 3 and 4 FSAR have a listing of TVA and non-TVA upstream dams that include facts about these dams and reservoirs including storage volume.

The Aluminum Company of America (ALCOA) projects located below Fontana Dam (Calderwood, Cheoah, and Chilhowee), along with non-TVA dams Nantahala and Santeetlah, were considered as part of the seismic analysis of Fontana and these non-TVA dams were assumed to fail in the seismic dam failure scenarios. One other non-TVA project (Thorpe), located on a tributary to the Little Tennessee River, was not considered. However, the potential impact of failure of the Thorpe project would be considered minimal in comparison to Fontana failure based on the relative storage volume of the two dams. At the top of its gates, Thorpe has a storage volume of 70,810 acre-feet, compared to over 1,800,000 acre-feet of combined storage in Fontana, Calderwood, Cheoah, Chilhowee, Nantahala and Santeetlah. The postulated failure of any of the other non-TVA dams would have minimal impact given their small storage volume, which ranges from 183 to 34,711 acre-feet. The technical basis for the evaluations related to the Bellefonte site is described in response to item c below.

There is not an established seismic design basis for these dams. However, TVA operates its Dam Safety Program (DSP) consistent with the Federal Guidelines for Dam Safety, which includes seismic provisions. The TVA DSP conducts technical studies and engineering analyses to assess the hydrologic and seismic integrity of agency dams and verifies that they can be operated in accordance with Federal Emergency Management Agency (FEMA) guidelines. These guidelines were developed to enhance national dam safety such that the risk to loss of life and property damage is minimized. As part of the TVA DSP, inspection and maintenance activities are carried out on a regular schedule to confirm the dams are maintained in a safe condition. Instrumentation to monitor the dams' behavior was installed in many of the dams during original construction. Other instrumentation has been added since and is still being added as the need arises or as new techniques become available.

The remaining dams are privately owned and regulated by the Federal Energy Regulatory Commission (FERC). Owners of private dams that produce power are required to demonstrate to FERC that their dams have acceptable seismic performance.

Table 1

Dams Upstream of Bellefonte

Dam Name	**Owner**
Nickajack	TVA
Chickamauga	TVA
Watts Bar	TVA
Fort Loudoun	TVA
Apalachia	TVA
Hiwassee	TVA
Chatuge	TVA
Ocoee #1	TVA
Ocoee #2	TVA
Ocoee #3	TVA
Blue Ridge	TVA
Nottely	TVA
Melton Hill	TVA
Norris	TVA
Tellico	TVA
Fontana	TVA
Cherokee	TVA
Douglas	TVA
Raccoon Mountain	TVA
Watauga	TVA
Fort Patrick Henry	TVA
South Holston	TVA
Wilbur	TVA

Boone	TVA
Nolichucky	TVA
John Sevier	TVA
Calderwood	ALCOA
Cheoah	ALCOA
Chilhowee	ALCOA
Santeetlah	ALCOA
Nantahala	Nantahala Power and Light (Duke Energy)
Thorpe	Nantahala Power and Light (Duke Energy)
Bear Creek	Nantahala Power and Light (Duke Energy)
Cedar Cliff	Nantahala Power and Light (Duke Energy)
Mission	Nantahala Power and Light (Duke Energy)
Queens Creek	Nantahala Power and Light (Duke Energy)
Wolf Creek	Nantahala Power and Light (Duke Energy)
East Fork	Nantahala Power and Light (Duke Energy)
Tuckasegee	Nantahala Power and Light (Duke Energy)
Walters	Progress Energy

(c) The estimation of the maximum site flood level has included seismic effects on the dams, as described in FSAR Subsection 2.4.4. The resulting flood levels of seismically induced dam failure scenarios are substantially less than the PMF described in section 2.4.3. Additional technical basis for making determinations of the extent of failure under the effects of the largest historic earthquake for the dam site are provided below.

The standard method of computing stability of concrete structures is used. The maximum base compressive stress, average base shear stress, the factor of safety against overturning, and the shear strength required for a shear-friction factor of safety of 1 are determined. To find the shear strength required to provide a safety factor of 1, a coefficient of friction of 0.65 is assigned at the elevation of the base under consideration. The analyses for earthquakes are based on the pseudo-static analysis methods as given in Hinds, et.al (1945), with increased hydrodynamic pressures determined by the method developed by Bustmantle and Flores (1966). These analyses include applying masonry inertia forces and increased water pressure to the structure resulting from the acceleration of the structure horizontally in the upstream direction and simultaneously in a downward direction. The masonry inertia forces are determined by a dynamic analysis of the structure which takes into account amplification of the accelerations through the structure above the foundation input acceleration.

No reduction of hydrostatic or hydrodynamic forces because of the decrease of the unit weight of water from the downward acceleration of the reservoir bottom is included in this analysis. Although accumulated silt on the reservoir bottom would dampen vertically traveling waves, the effect of silt on structures is not considered. Embankment analysis was made using the standard slip circle method, except for Chatuge and Nottely Dams where the Newmark method for dynamic analysis of embankment slopes was used. The effect of the earthquake is taken into account by applying the appropriate static inertia forces to the dam mass within the assumed slip circle.

In the analysis, the embankment design constants used, including shear strength of the materials in the dam and the foundation, are the same as those used in the original static stability analysis. Although detailed dynamic soil properties are not available, a value for seismic amplification through the soil has been assumed based on previous studies pertaining to TVA nuclear plants. These studies have indicated maximum amplification values slightly in excess of two for a rather wide range of shear wave velocity to soil height ratios. For these analyses, a straight-line variation is used with acceleration at the top of the embankment being two times the top of rock acceleration.

The SSE and OBE are defined as having maximum horizontal rock acceleration levels of 0.18 g and 0.09 g, respectively.

Evaluation ground motions were developed based on attenuating the peak ground acceleration associated with the SSE or OBE from the scenario epicenter to the dam. Single and multi-dam failure scenarios were considered. The conclusion of complete, partial or non-failure of a particular dam can vary with the level of attenuated ground motion and to a lesser extent the headwater and tailwater elevations for the scenario considered. The table in Attachment 03.07.01-02A summarizes the results for each dam for three of the multi-dam failure scenarios that produce the highest water elevations at Bellefonte.

The failure scenario for Norris Dam includes the subsequent overtopping failure of Melton Hill Dam.

Ocoee 2 and 3 have very little storage (less than 5,000 acre-feet combined) and these dams would be overtopped by the Blue Ridge failure flood wave as described in scenario one.

For scenario one, the north embankment at Chickamauga would fail due to overtopping, but would not fail in scenario two or three. The Nickajack embankment would fail in all three scenarios.

Structural calculations were not completed for Chickamauga and Nickajack dams. Instead, Chickamauga and Nickajack dams were considered to be removed instantly and completely both singly and simultaneously at the critical moment in the one-half Bellefonte PMF. The Raccoon Mountain pumped storage dam was not analyzed specifically because its small capacity would not add measurably to the Bellefonte flood level.

TVA has determined that additional analysis is necessary to address the Tellico Dam seismic failure scenarios given that the uppermost portion (at and above elevation 780 feet) of the non-overflow section was shown to potentially fail at the OBE level (0.09g). While as a matter of engineering judgment TVA determined that the dam does not fail at 0.09g, a literal interpretation of the calculation suggests that the uppermost portion (at and above elevation 780 feet) of the non-overflow section potentially fails at the OBE level (0.09g); no stability analysis was performed for lower accelerations. This additional analysis is being performed to confirm the conclusion that "resulting flood levels of seismically induced dam failure scenarios is substantially less than the PMF described in Subsection 2.4.3." The results of the additional analysis will be provided in a future supplement to this response.

References

Bustamante, J.I., and A. Flores, "Water Pressure in Dams Subject to Earthquakes," Journal of the Engineering Mechanics Division, ASCE Proceedings, October 1966.

Hinds, J.C., P. William, and J.D. Justin, "Engineering for Dams," Volume II, Concrete Dams, John Wiley and Sons, Inc, 1945.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

Subsection 2.4.4.1 will be revised in a future revision to reflect the results of additional analysis for Tellico Dam.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

Attachment 03.07.01-02A

Attachment 03.07.01-02A

(4 pages including this cover sheet)

BLN Potential Dam Failures

Supporting Information for FSAR Subsection 2.4.4

Scenario 1. Fontana, Hiwassee, Apalachia and Blue Ridge Dams - OBE + 1/2 PMF

Dam	Horizontal PGA	Construction Type	Extent of Failure	Supporting Basis & Comments
Fontana	0.09g	Concrete	Partial	See discussion on page 2.4-30; Section of dam (950 feet long) that contains longitudinal joints is conservatively assumed to fail at OBE PGA due to uncertainty about stability of these joints under dynamic loading
Hiwassee	0.09g	Concrete	Complete	No analysis - assumed to fail
Apalachia	0.07g	Concrete	Complete	No analysis - assumed to fail
Chatuge	0.08g	Embankment	None	See discussion on page 2.4-35 of FSAR R0; Newmark method of analysis indicates no significant deformation at PGA of 0.09g
Nottely	0.05g	Embankment	None	See discussion on page 2.4-27 of FSAR R0
Ocoee No. 1	0.03g	Concrete	None	Attenuated acceleration judged to be too low to cause failure; if failure occurred, the contribution to flooding levels at Bellefonte would be negligible compared to flood waters released by failure of the other dams involved in this scenario
Blue Ridge	0.04g	Embankment	Complete	No analysis - assumed to fail
Fort Loudoun	0.04g	Concrete	None	Factor of Safety > 1 for overturning; Resultant within spillway base; powerhouse base completely in compression
Fort Loudoun	0.04g	Embankment	None	See discussion on page 2.4-34 of FSAR R0
Tellico	0.04g	Concrete	Complete	Fails by overtopping from Fontana flood wave
Tellico	0.04g	Embankment	Complete	Fails by overtopping from Fontana flood wave
Watts Bar	0.03g	Concrete	None	See discussion on page 2.4-33 of FSAR R0; spillway and powerhouse sections do not fail at PGA <= 0.09g
Watts Bar	0.03g	Embankment	Partial	Saddle dam fails completely by overtopping; main embankment does not fail - slip circle analysis factor of safety >1.5
Nantahala			Complete	No analysis - assumed to fail; See discussion on page 2.4-31 of FSAR R0
Santeetlah			Complete	No analysis - assumed to fail; See discussion on page 2.4-31 of FSAR R0
Cheoah			Complete	No analysis - assumed to fail; See discussion on page 2.4-31 of FSAR R0
Calderwood			Complete	No analysis - assumed to fail; See discussion on page 2.4-31 of FSAR R0
Chilhowee			Complete	No analysis - assumed to fail; See discussion on page 2.4-31 of FSAR R0

Scenario 2. Norris, Cherokee and Douglas Dams - SSE + 25 year flood

Dam	Horizontal PGA	Construction Type	Extent of Failure	Supporting Basis & Comments
Norris	0.15g	Concrete	Partial	Failed in maximum cross section; other sections would not fail because they are smaller and have soil support on downstream side
Norris	0.15g	Embankment	None	Soil on the downstream side of the theoretical embankment cross section would prevent failure
Cherokee	0.09g	Concrete	Partial	Spillway - would not fail below elevation 1010 feet because dam has adequate capacity to resist computed shear and compressional stresses, all computed factors of safety > 1, and resultant is within base; Non-overflow - resultant inside base at elevation 910 feet.; resultant outside base at 981.5 feet and higher so failure assumed at these elevations
Cherokee	0.09g	Embankment	Partial	South embankment fails at maximum cross section (Factor of Safety=0.85) along circle intercepting crest and phreatic surface but does not intercept upstream reservoir so failure would be slow, but complete; saddle dams and north embankment have much lower heights so they would not fail just as smaller sections of south embankment slip circle analysis do not indicate failure
Douglas	0.09g	Concrete	Partial	Resultant falls within base of non-overflow section up to elevation 899 feet; Spillway in compression at elevation 815 feet and resultant within base; analysis follows findings of similar dam Cherokee; see discussion on page 2.4-29 of FSAR R0
Douglas	0.09g	Embankment	None	Saddle dam 1slip circle analysis factor of safety =1.0 for PGA = 0.09g
Fort Loudoun	0.08g	Concrete	Complete	Overtopping from flood wave causes failure
Fort Loudoun	0.08g	Embankment	Complete	Overtopping from flood wave causes failure
Tellico	0.08g	Concrete	None	The uppermost portion (at and above elevation 780 feet) of the non-overflow section was shown to potentially fail at the OBE level (0.09g); no stability analysis was performed for lower accelerations. Engineering judgement concluded that the dam would not fail. This issue has been inserted into the Corrective Action Program.
Tellico	0.08g	Embankment	None	Slip circle analysis assuming an acceleration of 0.09g has factor of safety greater than 1.0
Fontana	0.05g	Concrete	None	Attenuated acceleration judged to be too low to cause failure
Watts Bar	0.03g	Concrete	None	See discussion on page 2.4-33 of FSAR R0; spillway and powerhouse sections do not fail at PGA <= 0.09g
Watts Bar	0.03g	Embankment	Complete	Saddle dam fails completely by overtopping; main embankment does not fail - slip circle analysis factor of safety >1.5

Scenario 3. Cherokee and Douglas Dams - OBE + 1/2 PMF

Dam	Horizontal PGA	Construction Type	Extent of Failure	Supporting Basis & Comments
Cherokee	0.09g	Concrete	Partial	Spillway - would not fail below elev. 1010 feet because dam has adequate capacity to resist computed shear and compressional stresses, all computed factors of safety > 1, and resultant is within base; Non-overflow - resultant inside base at elev. 910 feet; resultant outside base at 981.5 feet and higher so failure assumed at these elevations
Cherokee	0.09g	Embankment	Partial	South embankment fails at maximum cross section (Factor of Safety=0.85) along circle intercepting crest and phreatic surface but does not intercept upstream reservoir so failure would be slow, but complete; saddle dams and north embankment have much lower heights so they would not fail just as smaller sections of south embankment slip circle analysis do not indicate failure
Douglas	0.09g	Concrete	Partial	Resultant falls within base of non-overflow section up to elevation 899 feet; Spillway in compression at elevation 815 feet and resultant within base; analysis follows findings of similar dam Cherokee; see discussion on page 2.4-29 of FSAR R0
Douglas	0.09g	Embankment	None	Saddle dam 1slip circle analysis factor of safety =1.0 for PGA = 0.09g