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

May 25, 2010

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:
1. 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
2. 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
3. Letter from Jack A. Bailey (TVA) to Document Control Desk (NRC), Bellefonte Combined License Application – Response to Request for Additional Information – Seismic Design Parameters, dated October 17, 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 Reference 1 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.

D085
NRO

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

Executed on this 25th day of May, 2010.

Jack A. Bailey

Jack A. Bailey
Vice President, Nuclear Generation Development

Enclosure
cc:
S. P. Frantz, Morgan Lewis
M.W.Gettler, FP&L
R. C. Grumbir, NuStart
P. S. Hastings, NuStart
P. Hinnenkamp, Entergy
R. H. Kitchen, PGN
M. C. Kray, NuStart
A. M. Monroe, SCE&G
C. R. Pierce, SNC
L. Reyes, NRC/R11
A. Zinke, NuStart
M. M. Comar, NRC HQ
A. H. Fetter, NRC HQ

cc: (W/o Enclosure)

G. P. Arent, EQB 1A - WBN
J. A. Bailey, LP 5A - C
A. L. Sterdis, LP 5A - C
E. J. Vigluicci, WT 6A - K
C. C. Chandler, WT 6A - K
S. A. Vance, WT 6A - K
P.D. Maddux, LP 5A-C

EDMS, WT CA - K

Responses to NRC Request for Additional Information letter No. 110 dated August 7, 2008
(3 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	October 17, 2008
03.07.01-03	This letter – see following pages

Associated Additional Attachments / Enclosures Included		Pages
Attachment 02.04.04 - 1A	Revision to FSAR 2.4.4	32 Pages
Attachment 02.04.04 - 1B	FSAR 2.4.16 Proposed Revision (References)	6 Pages
Attachment 02.04.04 - 2	Table and Figure Change Roadmap	3 Pages

NRC Letter Dated: August 7, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 03.07.01-03

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) See RAI 110 Supplement 1 ref (3) 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 October 17, 2008

(b) See RAI 110 Supplement 1 ref (3) Letter from Jack A. Bailey (TVA) to Document Control Desk (NRC), Bellefonte Combined License Application – Response to Request for Additional Information – Seismic Design Parameters, dated October 17, 2008

(c) This response supplements the information provided in RAI 110 Supplement 1 ref (3) Letter from Jack A. Bailey (TVA) to Document Control Desk (NRC), Bellefonte Combined License Application – Response to Request for Additional Information – Seismic Design Parameters, dated October 17, 2008

TVA has reanalyzed site flood levels from potential dam failures. The analysis has confirmed that the resulting flood levels of seismically induced dam failure scenarios are substantially less than the PMF described in FSAR Section 2.4.3. The results of this analysis are included in a proposed revision to FSAR Section 2.4.4.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:
COLA Part 2, FSAR Sections 2.4.4, and 2.4.16 will be revised from the text that is currently in BLN COLA Revision 2 submitted Letter from Andrea L. Sterdis (TVA) Document Control Desk (NRC) BELLEFONTE COMBINED LICENSE APPLICATION –REQUIRED UPDATES OF SAFETY ANALYSIS AND DEPARTURES REPORTS - Submittal No. 5, dated December 15, 2009.

To read:
Insert Proposed FSAR Subsections 2.4.4, and 2.4.16

ASSOCIATED ATTACHMENTS/ENCLOSURES:

Attachment 02.04.04 - 1A	Revision to FSAR 2.4.4	32 Pages
Attachment 02.04.04 - 1B	FSAR 2.4.16 Proposed Revision (References)	6 Pages
Attachment 02.04.04 - 2	Table and Figure Change Roadmap	3 Pages

Attachment 02.04.04 - 1A

Revision to FSAR Section 2.4.4

(32 Pages including Cover Sheet)

LIST OF TABLES

Bellefonte Nuclear Plant, Units 3 & 4
COL Application
Part 2, FSAR

LIST OF FIGURES

BLN COL 2.4-2 2.4.4 POTENTIAL DAM FAILURES

The procedures referred to in Regulatory Guide (RG) 1.59[a], Appendix A, were followed for evaluating potential flood levels from seismically induced dam failures. In accordance with this guidance, seismic dam failure is examined using the two specified alternatives: (1) the Safe Shutdown Earthquake (SSE) coincident with the peak of the 25-year flood and a 2-year wind speed applied in the critical direction, and (2) the Operating Basis Earthquake (OBE) coincident with the peak of the one-half PMF or the 500-year flood, whichever is less, and a 2-year wind speed applied in the critical direction.

There are 17 major dams above BLN whose failure could influence plant site flood levels. Dam locations with respect to the BLN site are shown in Figure 2.4.1-205. These include Boone, Chatuge, Cherokee, Douglas, Fontana, Hiwassee, Norris, Nottely, South Holston, Tellico, Watauga, Blue Ridge, Ocoee No. 1, Fort Loudoun, Watts Bar, Chickamauga, and Nickajack. These were examined individually, and in combinations, to determine if failure might result from a seismic event and, if so, would failure concurrent with storm runoff create maximum flood levels at the plant (Reference 254). The earthquake postulated in these dam failure analyses was a deterministic earthquake based on the largest historic earthquake to occur in the area consistent with the requirements of 10 CFR 100.23d(3), but differs from the probabilistic earthquake required by 10 CFR 100.23d(1) for new plant design as discussed in Section 2.5. These analyses are adequate and bounding for BLN based on the following considerations, which are discussed in greater detail later in this introductory subsection:

- As required by General Design Criterion 2, the largest historic earthquake to occur in the area was used to pseudo-statically evaluate the dams. Current information from the TVA Dam Safety Program (DSP) demonstrates seismic ruggedness of concrete gravity dams. Also, TVA DSP has completed dynamic stability analysis on Fontana and Hiwassee dams using probabilistic earthquake response spectra, which envelope the BLN OBE spectra, and were shown by this recent analysis to withstand high seismic demand.

- One-half PMF was conservatively used in the analysis. Because the one-half PMF is significantly larger than the 500-year flood, it bounds the lesser 500-year flood permitted by the guidance.

- The combined event probability of exceedance of 1 x 10^{-6} required by the guidance is bound by the combined events considered in the analyses.

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[a] The material previously contained within Appendix A of Regulatory Guide (RG) 1.59 was replaced by American National Standards Institute (ANSI) Standard N170-1976. This ANSI standard has since been replaced by ANSI/American Nuclear Society (ANS) standard 2.8-1992 (Reference 203). The procedures described in ANSI/ANS 2.8-1992 were followed for evaluating potential flood levels from seismically induced dam failures.

- The seismically induced flood elevations required by the guidance are bounded by the PMF elevation determined in the analysis.

From the seismic dam failure analyses made for TVA's operating nuclear plants, it was determined that three separate, combined events have the potential to create maximum seismic-caused flood levels at BLN. These events are as follows:

1. The simultaneous failure of Fontana, Hiwassee, Apalachia, and Blue Ridge dams in the OBE during one-half PMF.
2. The simultaneous failure of Norris, Cherokee, Douglas, and Tellico dams in the SSE during a 25-year flood.
3. The simultaneous failure of Cherokee, Douglas, and Tellico dams in the OBE during one-half PMF.

Tellico has been added to events 2 and 3, which was not included in the original analyses for TVA's operating nuclear plants. It was included because the seismic analysis of Tellico is not conclusive. Therefore, Tellico was postulated to fail.

<u>Seismic Ruggedness of Concrete Gravity Dams</u>

The plant site and upstream reservoirs are located in the Southern Appalachian Tectonic Province and, therefore, are subject to moderate earthquake forces. The upstream dams whose failure has the potential to cause critical flood levels at BLN were investigated to determine if failure from seismic events would endanger plant safety.

General Design Criterion 2, *Design Basis for Protection against Natural Phenomena*, of 10 CFR Part 50, Appendix A, requires that the design bases for structures, systems, and components important to safety shall reflect the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin. To satisfy this criterion, the earthquake event used for the TVA dam failure analyses was based on the largest historic earthquake to occur in the Southern Appalachian Tectonic Province - the 1897 Giles County, Virginia, earthquake. This earthquake was estimated to have had a body wave magnitude (m_b) of 5.8. The SSE for these studies was conservatively established as having a maximum horizontal acceleration of 0.18 g and a simultaneous maximum vertical acceleration of 0.12 g. The OBE was established as one-half SSE, therefore having a maximum horizontal acceleration of 0.09 g and a simultaneous maximum vertical acceleration of 0.06 g.

The TVA DSP, which is consistent with the Federal Guidelines for Dam Safety (Reference 207), 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 potential for 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. Based on the implementation of the DSP, TVA has confidence that its dams are safe against catastrophic destruction by any natural forces that could be expected to occur.

Concrete gravity dams similar to many TVA dams have performed very well during earthquakes throughout the world. Only one concrete gravity dam, the Shih-Kang Dam in Taiwan, is known to have failed because of an earthquake. This dam failure was caused by the fault rupture crossing directly beneath the dam and offsetting portions of the dam 29 ft. vertically and 6.5 ft. horizontally (Reference 246). Surface ruptures such as this are not expected to occur in the BLN area, as discussed in Section 2.5, or beneath any of the dams upstream of the BLN site. Worldwide, no other concrete gravity dams have failed due to earthquakes and only a few concrete dams have experienced any damage due to earthquakes, although dams have been subjected to earthquakes with Modified Mercalli (MM) intensities ranging from VIII to IX and ground accelerations have been measured as high as 0.51 g perpendicular to the dam axis and 0.36 g peak vertical acceleration (Reference 207).

Therefore, based on the known seismic ruggedness of these concrete dams, the analyses assuming partial or total failure of the concrete dams in the controlling combined event scenario are conservative.

Since the implementation of the TVA DSP in 1982, additional analyses and studies have been completed on several TVA dams based on a priority ranking. The TVA DSP has recently completed a dynamic stability analysis of Fontana and Hiwassee dams using probabilistic earthquake spectra. Other TVA dams have pseudo-static stability analysis performed, while others have no unique stability analysis but are compared to other analyzed dams. The results of these studies under the TVA DSP, as further discussed below, have shown that Fontana, Norris, and Hiwassee would not catastrophically fail during the defined seismic events.

A comparison of the BLN OBE[b] to the response spectra used in the TVA DSP analyses for Fontana and Hiwassee is made in Figure 2.4.4-201. These analyses envelope the BLN OBE demand and therefore provide further evidence that the assumption of catastrophic failure of Fontana and instant disappearance of Hiwassee Dam is extremely conservative. In addition, these analyses give confidence of the ability of other similar TVA dams to withstand the high frequency demand of the BLN Ground Motion Response Spectra (GMRS). Apalachia, Fort Patrick Henry, Melton Hill, and Ocoee No. 3, which are concrete gravity dams,

b. The BLN OBE for this comparison is defined as 1/2 Ground Motion Response Spectra (GMRS). The GMRS is discussed in Subsection 2.5.2 and shown in Figure 2.5-290.

were not analyzed for stability because these dams are not of concern for detailed design studies due to their small storage volume. Cherokee and Douglas are concrete gravity dams with embankments with limited stability analyses. Due to the lack of any known structural deficiencies, and the relatively low seismic hazard for these dams, performance of a detailed seismic evaluation for these dams was not considered necessary.

One-Half PMF versus 500-Year Flood

The procedures prescribed to by the guidance require seismic dam failure to be examined using the SSE coincident with the peak of the 25-year flood, and the OBE coincident with the peak of one-half the PMF or 500-year flood, whichever is less. The analyses consider a more severe one-half PMF instead of the 500-year flood; therefore, these analyses bound those prescribed by the Regulatory Guide.

Probability of Exceedance

The cumulative annual probability of exceedance for each of the alternative combinations is tabulated in Table 2.4.4-201. These exceedance probabilities are calculated for the SSE/OBE used in the analyses. The cumulative annual probabilities are 1.3×10^{-8} and 2.4×10^{-9} which satisfies the acceptance level of 1.0×10^{-6} set forth by Regulatory Guide 1.59.

2.4.4.1 Dam Failure Permutations

According to guidance, seismic dam failure is to be examined using the SSE coincident with the peak of the 25-year flood, and OBE coincident with the peak of one-half PMF or 500-year flood, whichever is less. The guidance also specifies a 2-year wind speed applied in the critical direction.

All references to SSE and OBE in this subsection are based on previous dam failure analyses made for TVA's operating nuclear plants, and refer to SSE and OBE as defined per 10 CFR Part 100, Appendix A, consistent with guidance of Regulatory Guide 1.59, in accordance with the requirements of 10 CFR 100.23d(3).

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 method provided by Hinds in Reference 208, with increased hydrodynamic pressures determined by the method developed by Bustamante and Flores in Reference 204. 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 above the foundation rock.

No reduction of hydrostatic or hydrodynamic forces due to the decrease of the unit weight of water from the downward acceleration of the reservoir bottom is included in this analysis.

Waves created at the free surface of the reservoir by an earthquake are considered of no importance. Based upon studies by Chopra in Reference 205 and Zienkiewicz in Reference 247, it is judged that before waves of any significant height have time to develop, the earthquake is over. The duration of earthquake used in this analysis is in the range of 20 to 30 seconds.

Although accumulated silt on the reservoir bottom would dampen vertically traveling waves, the effect of silt on structures is not considered. The effect of silt is judged to be minimal, if any, as the accumulation rate is slow, as measured by TVA for many years (Reference 253).

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 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. The three critical multiple structure failure scenarios are described in further detail below.

1. Fontana, Hiwassee, Apalachia, and Blue Ridge Dams

Fontana, Hiwassee, Apalachia and Blue Ridge Dams could fail when the OBE is located within a football-shaped area located between Fontana and Hiwassee dams (Figure 2.4.4-202). Failure scenarios for Fontana, Hiwassee, Apalachia, and Blue Ridge dams include postulated simultaneous failure of non-TVA dams on the Little Tennessee River, Cheoah, Calderwood and Chilhowee and on its tributaries, Nantahala and Santeetlah dams.

The OBE event produces maximum ground accelerations of 0.09 g at Fontana, 0.09 g at Hiwassee, 0.07 g at Apalachia, 0.08 g at Chatuge, 0.05 g at Nottely, 0.03 g at Ocoee No.1, 0.04 g at Blue Ridge, 0.04 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. Figure 2.4.4-203 shows the postulated condition of Fontana Dam after failure. Hiwassee, Apalachia, and Blue Ridge dams are postulated to completely fail. Chatuge is judged not to fail in this defined OBE event.

Nottely Dam is a rock-fill dam with large central impervious rolled fill core. The maximum attenuated ground acceleration at Nottely in this event is only 0.054 g. A field exploration boring program and laboratory testing program of samples obtained in a field exploration was conducted. During the field exploration program, standard penetration test blow counts were obtained on both the embankment and its foundation materials. Both static and dynamic (cyclic) triaxial shear tests were made. The Newmark Method of Analysis utilizing the information obtained from the testing program was used to determine the structural stability of Nottely Dam. It is concluded that Nottely Dam can resist the attenuated ground acceleration of 0.054 g with no detrimental damage.

Ocoee No.1 Dam is a concrete gravity structure. The maximum attenuated ground acceleration is 0.03 g. Based on past experience of concrete dam structures under significantly higher seismic ground accelerations, the Ocoee No. 1 Dam is judged to remain stable following exposure to a 0.03 g base acceleration with amplification.

Ocoee No. 1 and Ocoee No. 3 dams, downstream of Blue Ridge Dam, would be overtopped and were postulated to completely fail at their respective maximum headwater elevations. Ocoee No. 2 has no reservoir storage and was not considered.

Fort Loudoun and Watts Bar spillways would remain operable. Although Tellico Dam is postulated to fail seismically, downstream impacts of the failure of Tellico by overtopping are judged to be more severe than the seismic failure of Tellico. Therefore, for the analysis of this scenario, the seismic failure of Tellico Dam is not postulated. The Fontana failure wave would overtop and fail the Tellico embankment. Water would transfer through the connecting canal into Fort Loudoun, but it would not be sufficient to overtop the dam or to prevent overtopping failure of Tellico Dam. The maximum headwater at Fort Loudoun would reach elevation 832.48 ft. msl, 4.52 ft. below the top of the dam. Watts Bar headwater would reach elevation 765.20 ft. msl, 4.80 ft. below the top of dam. The west saddle dike at Watts Bar with a top elevation of 757.00 ft. msl would be overtopped and breached. The saddle dike is postulated to fail completely to elevation 750.00 ft. msl.

The discharge from Watts Bar Dam and the failed saddle dike combined with the combined failure flow of Hiwassee, Apalachia, and Blue Ridge dams would overtop Chickamauga Dam. Both embankments at Chickamauga Dam would breach. The north embankment at Nickajack Dam would be overtopped and breached.

The peak discharge at the BLN site produced by the OBE failure of Fontana, Hiwassee, Apalachia, and Blue Ridge coincident with the one-half PMF is 924,024 cfs. The peak elevation is 617.17 ft.

2. Norris, Cherokee, Douglas, and Tellico Dams

In order to fail these four dams (Norris, Cherokee, Douglas, and Tellico), the SSE epicenter must be confined to a relatively small circular area about 10 mi. wide and 20 mi. long. Figure 2.4.4-204 shows the location of the SSE and its attenuation. The SSE event produces maximum ground accelerations of 0.15 g at Norris, 0.09 g at Cherokee and Douglas, 0.08 g at Fort Loudoun and Tellico, 0.05 g at Fontana, and 0.03 g at Watts Bar. Norris, Cherokee, Douglas, and Tellico dams are postulated to fail. Fort Loudoun and Watts Bar are judged not to fail. Fontana Dam was excluded on the basis of its distant location from the cluster of dams under consideration.

For the postulated SSE failures of Cherokee and Douglas, the portions judged to remain and debris arrangements are shown in Figure 2.4.4-205 and Figure 2.4.4-206, respectively. The SSE failure configurations are the same as that shown for OBE conditions. The failure configuration of Norris Dam is shown in Figure 2.4.4-207. The center 833-ft. failure section of Norris Dam includes the spillway and intake portions of the dam. The resulting debris downstream would occupy the valley cross section with a top elevation of 970 ft. msl. The discharge rating for this controlling debris section at Norris was developed from a 1:150 scale hydraulic model at the TVA Engineering Laboratory and was verified by mathematical analysis. Figure 2.4.4-208 shows the postulated condition of Tellico after failure. However, Tellico was failed completely as the portions judged to remain are relatively small.

Because Tellico is postulated to fail seismically, transfer of water from Fort Loudoun through the connecting canal into the Tellico Reservoir will occur. The flood from the failure of Cherokee and Douglas dams upstream would overtop and breach the Fort Loudoun south embankment and marina saddle dam. The flood from the failure of Norris would overtop Melton Hill Dam, and the dam was postulated to completely fail at headwater elevation 817 ft. msl. The headwater at Watts Bar Dam would reach elevation 768.66 ft. msl, 1.34 ft. below top of dam. The west saddle dike and the east embankment wall at Watts Bar would be overtopped and breached. Chickamauga Dam would be overtopped and both embankments breached. The north embankment at Nickajack Dam would be overtopped and breached.

The peak discharge at the BLN site produced by the SSE failure of Norris, Cherokee, Douglas, and Tellico dams coincident with the 25-year flood is 985,734 cfs. The peak elevation is 618.28 ft.

3. Cherokee, Douglas, and Tellico Dams

The postulated simultaneous failure of Cherokee, Douglas, and Tellico dams could occur when the OBE is located between Cherokee and Douglas, which are approximately 15 mi. apart.

The results of the Cherokee Dam stability analysis in the OBE indicate the spillway is stable at the foundation base elevation 900 ft. Analyses made for other elevations above 900 ft. indicate the resultant forces fall outside the base at elevation 1010 ft. msl. The spillway is postulated to fail at that elevation. The non-overflow dam is embedded in fill to elevation 981.5 ft. msl and is considered stable below that elevation. However, stability analysis indicates failure will occur above the fill line. The powerhouse intake is massive and backed up by the powerhouse. Therefore, it is judged to be stable in the OBE.

Analysis was made for the highest portion of the south embankment using the same shear strengths of material as were used in the original analysis. The resulting factor of safety is less than 1. Therefore, the south embankment is postulated to fail. Because the north embankment and saddle dams 1, 2, and 3 are generally about one-half or less as high as the south embankment, they are judged to be stable in the OBE.

All debris from the failure of the concrete portion is postulated to be located downstream in the channel at elevations lower than the remaining portions of the dam, and therefore, will not obstruct flow. Figure 2.4.4-205 shows the postulated condition of the Cherokee Dam after failure.

The upper part of the Douglas spillway is approximately 12 ft. higher than Cherokee, but the amplification of the rock surface acceleration is the same. Therefore, based on the Cherokee analysis, it is projected that the Douglas spillway will fail at elevation 937 ft. msl, which corresponds to the postulated failure elevation of the Cherokee spillway.

The Douglas non-overflow dam is similar to that at Cherokee and is embedded in fill to elevation 927.5 ft. msl. The spillway is considered stable below that elevation. However, based on the Cherokee analysis, it is postulated to fail above the fill line in the OBE. The powerhouse intake is massive and backed up downstream by the powerhouse. Therefore, it is considered stable. Results of the analysis of the saddle dams indicate a factor of safety of 1. Therefore, the saddle dams are considered to be stable for the OBE.

All debris from the failed portions is postulated to be located downstream in the channel at elevations lower than the remaining portions of the dam, and therefore, will not obstruct flow. Figure 2.4.4-206 shows the portions of Douglas postulated to fail and the portions judged to remain.

Tellico was postulated to completely fail in this combination.

Although transfer of water from Fort Loudoun into Tellico through the connecting canal will occur, the flood from the failure of the Cherokee and Douglas dams upstream will still overtop and breach the Fort Loudoun south embankment and marina saddle dam. At Watts Bar Dam, the headwater would reach elevation 763.10 ft. msl, 6.90 ft. below top of dam. The west saddle dike at Watts Bar would be overtopped and breached. A complete washout of the dike was assumed. The headwater at Chickamauga Dam would reach elevation 705.80 ft. msl, 0.20 ft. below top of dam. The north embankment at Nickajack Dam would be overtopped and breached.

The peak discharge at the BLN site produced by the OBE failure of Cherokee, Douglas, and Tellico dams coincident with the one-half PMF is 778,217 cfs. The peak elevation is 614.25 ft.

Two additional events were evaluated in addition to the three controlling seismic events discussed in Subsection 2.4.4 above. The two additional events are:

4. Simultaneous failure of Norris, Douglas, Fort Loudoun, and Tellico Dams in the SSE during a 25-year flood.

5. Failure of Norris and Tellico Dams in the OBE during one-half the PMF.

4. Norris, Douglas, Fort Loudoun, and Tellico Dams

Figure 2.4.4-209 shows the location of an SSE and its attenuation which produces 0.12 g at Norris, 0.08 g at Douglas, 0.12 g at Fort Loudoun and Tellico, 0.07 g at Cherokee, 0.06 g at Fontana, and 0.04 g at Watts Bar. Cherokee is judged not to fail at 0.07 g. Watts Bar is judged not to fail at 0.04 g. Fontana Dam is also judged not to fail at 0.06 g, (excluded on the basis of its distant location from the cluster of dams under consideration).

The postulated SSE failure configuration of Norris Dam is shown in Figure 2.4.4-207. The SSE postulated failure of Douglas Dam is judged to be as previously discussed for the OBE.

The results of the Fort Loudoun Dam stability analysis indicate the spillway section will fail. Based on the analyses of Cherokee and Douglas, the entire spillway section is projected to fail above elevation 750 ft. msl, as well as the bridge supported by the spillway piers. The results of the slip circle analysis for the highest portion of the embankment indicate a factor of safety less than 1.

No analysis was made for the powerhouse under SSE. However, an analysis was made for the OBE with no water in the units, a condition believed to be extremely remote to occur. Because stresses were low and a large percentage of the base was in compression, it is considered that the addition of water in the units would be a stabilizing factor, and the powerhouse section is judged not to fail.

The postulated SSE failure configuration of Fort Loudoun Dam is shown in Figure 2.4.4-210.

Tellico Dam was postulated to fail completely.

The flood from the failure of Norris would overtop Melton Hill Dam, and the dam was postulated to completely fail at headwater elevation 817 ft. msl. The postulated failure combination results in Watts Bar headwater elevation of 764.26 ft. msl, 5.74 ft. below top of dam. The west saddle dike would be overtopped and breached. A complete washout of the dike was assumed. The maximum headwater would reach elevation 705.74 ft. msl at Chickamauga Dam, 0.26 ft. below top of dam. The north embankment at Nickajack Dam would be overtopped and breached.

The peak discharge at the BLN site produced by the SSE failure of Norris, Douglas, Fort Loudoun, and Tellico dams coincident with a 25-year flood is 767,664 cfs. The peak elevation is 613.62 ft.

5. Norris and Tellico Dams

The results of the Norris Dam stability analysis in the OBE for a typical spillway block and typical non-overflow section of the maximum height indicate only a small percentage of the spillway base is in compression. This structure is postulated to fail. The high non-overflow center section with a small percentage of the base in compression and with high compressive and shearing stresses is also postulated to fail. Tellico Dam is also postulated to fail in the OBE event.

The center 665-ft. failure section of Norris Dam includes the spillway and intake portions of the dam. Based on stability analysis, the remaining non-overflow section is judged to withstand the OBE. The resulting debris downstream would occupy the valley cross section with a top elevation of 970 ft. msl. The discharge rating for this controlling debris section was developed from a 1:150 scale hydraulic model at the TVA Engineering Laboratory and was verified by mathematical analysis. Figure 2.4.4-211 shows the postulated condition of the dam after failure.

Tellico was conservatively postulated to fail completely in this event.

The Norris failure wave would overtop Melton Hill Dam. Melton Hill Dam was postulated to completely fail when the flood wave reached headwater elevation 817 ft. msl. The headwater at Watts Bar Dam would reach elevation 763.42 ft. msl, 6.58 ft. below top of dam. The west saddle dike at Watts Bar would be overtopped and breached. A complete washout of the dike was assumed. Chickamauga headwater would reach 704.65 ft. msl, 1.35 ft. below top of dam. The north embankment at Nickajack Dam would be overtopped and breached.

The peak discharge at the BLN site produced by the OBE failure of Norris and Tellico dams coincident with the one-half PMF is 757,289 cfs. The peak elevation is 613.69 ft.

Additional potential structure failures analyzed in the original study in the OBE and SSE are discussed below. These scenarios are non-controlling and would result in flood levels significantly less than those described.

6. Chickamauga and Nickajack Dams

The Chickamauga and Nickajack dams were not analyzed structurally for an OBE and are judged to fail instantly and completely, both singly and simultaneously during the one-half PMF. Although a reevaluation has not been performed for this scenario, flood levels from simultaneous failure of both dams would not be a controlling event based on previous studies.

7. Watts Bar Dam

Stability analysis of Watts Bar Dam powerhouse and spillway sections in the OBE result in the judgment that these structures will not fail. Dynamic analysis of the concrete structures resulted in the determination that the base acceleration is amplified at levels above the base. The slip circle analysis of the earth embankment section results in a factor of safety greater than 1, and the embankment is judged not to fail.

An evaluation was not made for Watts Bar Dam in SSE conditions. Previous evaluations determined that if the dam is arbitrarily removed instantly, the flood levels would not be controlling.

8. Fort Loudoun Dam

Stability analysis of Fort Loudoun Dam powerhouse and spillway sections indicate these structures will not fail in the OBE. Slip circle analysis of the earth embankment results in a factor of safety greater than 1, and the embankment is judged not to fail.

No hydrologic routing for the single failure of Fort Loudoun in the SSE, including the bridge structure, was made because its simultaneous failure with Tellico, Norris, and Douglas, is controlling.

9. Tellico Dam

Results of the stability analyses in the OBE for a typical spillway block and the earth embankment indicates acceptable factors of safety against overturning. The

OBE stability analysis of the Tellico non-overflow section at elevation 799.3 ft. (geometric break point) indicates a resultant outside the base. Tellico is conservatively postulated to fail in the OBE and SSE. No hydrologic routing for the OBE and SSE single failure of Tellico in the SSE is made, because a single failure is bounded by its simultaneous failure in combination with other dams.

10. Norris Dam

Figure 2.4.4-207 shows the part of the dam postulated to fail in the SSE and the location and height of the resulting debris. It is evident that flood levels would be considerably lower than the simultaneous failure of Norris, Cherokee, Tellico, and Douglas in the SSE as previously discussed.

11. Cherokee and Douglas Dams Separately

No hydrologic results are given for the single failure of Cherokee or Douglas dams in the OBE because the simultaneous failure of Cherokee, Douglas, and Tellico, as previously discussed, is more critical. The SSE will produce the same postulated failure condition for Cherokee and Douglas as described for the OBE.

12. Hiwassee, Apalachia, Blue Ridge, Ocoee No. 1, and Nottely Dams

These five dams could fail when the OBE is critically located. All five dams were postulated to completely fail in this event. The original analysis determined that this event would produce flood levels several feet below the controlling events discussed above.

13. Chatuge Dam

Chatuge Dam is a homogeneous, impervious rolled-fill dam. With the epicenter of the OBE located at the dam, the maximum ground acceleration is 0.09 g. Ground accelerations of this magnitude would have no detrimental effect on a well-constructed, compacted earthfill embankment. There are no known failures of compacted earth embankment slopes from earthquake motions. Failures to date have been associated with liquefaction of hydraulic fill embankments or with other loose granular foundation materials. The rolled embankment materials in Chatuge are not sensitive to liquefaction.

A field exploration boring program and laboratory testing program of samples obtained was conducted. During the field exploration program, standard penetration test blow counts were obtained on both the embankment and its foundation materials. Both static and dynamic (cyclic) triaxial shear tests were made. The Newmark Method of Analysis utilizing the information obtained from the testing program was used to determine the structural stability of Chatuge Dam. It was concluded that Chatuge Dam can resist the ground acceleration of 0.09 g with no detrimental damage.

14. Fort Loudoun, Tellico, and Fontana Dams

An SSE centered between Fontana and the Fort Loudoun-Tellico complex was postulated to fail these three dams. The three Aluminum Company of America (ALCOA) dams downstream from Fontana and Nantahala, upstream, and Santeetlah on a downstream tributary were also postulated to fail completely in this event. Watts Bar Dam and spillway gates would remain intact, but failure of the roadway bridge was postulated, which would render the spillway gates inoperable. At the time of seismic failure, discharges would be small in the coincident 25-year flood. For conservatism, Watts Bar gates were postulated to be inoperable in the closed position after the SSE event. The resulting flood levels, as shown in earlier studies for TVA's operating plants, would not be controlling.

15. Douglas, Fontana, and Tellico Dams

Douglas, Tellico, and Fontana were postulated to fail simultaneously in the SSE. The location of the SSE required to fail these dams would produce 0.14 g at Douglas, 0.09 g at Fontana, 0.07 g at Cherokee, 0.05 g at Norris, 0.06 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. The postulated failure configurations of Douglas and Fontana would be the same as previously described. Tellico was postulated to fail completely. Fort Loudoun and Watts Bar have previously been judged not to fail in the OBE, as previously discussed. The bridge at Fort Loudoun Dam, however, might fail under 0.06 g forces, falling on gates and on gate hoisting machinery. Fort Loudoun gates were postulated to be inoperable in the closed position following the SSE event. The resulting flood level is judged to be less than the SSE failure of Norris, Cherokee, Douglas, and Tellico.

16. Fontana and Hiwassee River Dams

Fontana and six Hiwassee River dams (Hiwassee, Apalachia, Chatuge, Nottely, Blue Ridge, and Ocoee No. 1) were postulated to fail simultaneously in the SSE. The postulated failure of Fontana would be the same as that previously described. The six Hiwassee dams were postulated to fail completely. Fort Loudoun, Tellico, and Watts Bar are judged not to fail with all gates operable. The resulting flood level is judged not to be controlling based on early studies for TVA's operating plants.

17. Raccoon Mountain Dam

Raccoon Mountain pumped storage dam was not analyzed because of its small capacity (37,800 ac.-ft.) and its considerable upstream distance (53 mi.). Its complete and coincidental failure would not add measurably to the flood level.

There are no safety-related facilities that could be affected by loss of water supply due to dam failure. This is addressed further in Subsection 2.4.11. Additionally, there are no safety-related facilities that could be affected by water supply blockages due to sediment deposition or erosion during dam failure-induced flooding. Landslide potential is addressed in Subsection 2.4.9. There are no on-site water control or storage structures located above site grade that may induce flooding. There are no safety-related structures that could be affected by waterborne objects.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

The unsteady flow models described in Subsection 2.4.3.3 were used to route the outflows from the postulated seismically induced dam failures in conjunction with additional routing techniques described below.

The outflows for postulated failure of Cherokee, Douglas, Norris, Fontana, Hiwassee, and Blue Ridge dams were determined using HEC-HMS and HEC-RAS. Because Apalachia is a very short reservoir with a small storage volume, its failure was included by adding the total volume into the analysis at the time of failure. The base case flood (25-year or one-half PMF events) was routed in HEC-HMS for each reservoir to determine the maximum headwater level and hence, the time of failure depending on the failure combination. The HEC-HMS was then setup to fail the structure instantaneously to the predetermined failure configuration using a post-failure rating curve. To determine if submergence correction should be applied, a HEC-RAS model was set up for the reach below the dam. The outflow from the failed dam was then routed thru the downstream reach to check for any submergence effects. These simulations indicated that no correction for submergence was required. TRBROUTE was used to simulate the same failure event as a verification of the outflow from the project. The Fort Loudoun unsteady flow model, which extends up the Holston and French Broad rivers to Cherokee and Douglas dams, respectively, was used to route the Cherokee and Douglas failure outflows downstream.

The Clinch River unsteady flow model, which extends up to Norris Dam, was used to route the Norris failure outflows downstream.

An outflow hydrograph was determined at Chilhowee (ALCOA project) dam located 27.4 mi. downstream of Fontana. In the reach between these projects there are two other ALCOA projects, Calderwood and Cheoah. The three ALCOA projects were postulated to fail during the OBE at the same time as the Fontana failure. In addition, two other ALCOA projects, Nantahala Dam, located about 47 mi. upstream of Fontana, and Santeetlah Dam, located on a tributary to the Little Tennessee River below Fontana, were postulated to fail at the same time. The combined outflows from all of these failed projects (six) were routed downstream in conjunction with the base case one-half PMF event using a HEC-RAS unsteady flow simulation. The outflow hydrograph at Chilhowee was compared with a non-QA SOCH unsteady flow model result. The Fort Loudoun/Tellico unsteady flow model, which extends up to Chilhowee, was used to route the resulting failure wave downstream.

The failure outflows from Hiwassee and Blue Ridge dams with subsequent failure of Ocoee No. 1 and Ocoee No. 3 dams were approximated at Hiwassee River mile (HRM) 18.9. The inflow at HRM 18.9 was based on examination of the most extreme case of adding the volume of all failed dams and assuming it was placed at HRM 18.9, without lag or attenuation, and on results of routings using an unverified unsteady flow model. Using these data as guidance, an inflow hydrograph was approximated at HRM 18.9. The Hiwassee River unsteady flow model, which extends up to HRM 18.9, was used to route the failure wave downstream.

2.4.4.3 Water Level at the Plant Site

The translation of flow to elevation is discussed in Subsection 2.4.3.3. The maximum flood elevation as a result of seismically induced multiple dam failures would be 618.28 ft. msl. The flood elevation and discharge hydrograph is shown in Figure 2.4.4-212. It would result from the postulated simultaneous failure of Norris, Cherokee, Douglas, and Tellico dams in the SSE coincident with the 25-year flood. Table 2.4.4-202 provides a summary of maximum flood elevations determined for the five failure combinations analyzed. Coincident wind wave activity for the PMF is described in Subsection 2.4.3.6. Wind waves were not computed for the seismic event floods, but superimposed wind wave activity would result in a water surface elevation several feet below the PMF described in Subsection 2.4.3.

2.4.4.4 Flood Wave Travel Time

The time of travel from the postulated failure times to maximum elevation at BLN ranges from 105 to 127 hours. None of the five cases considered exceeded plant grade elevation 628.6 ft msl.

BLN COL 2.4-2

TABLE 2.4.4-201
CUMULATIVE ANNUAL PROBABILITY OF EXCEEDANCE FOR SEISMICALLY INDUCED DAM FAILURE SCENARIOS[(a)]

Case 1 – Safe Shutdown Earthquake (SSE) Combined with 25-Year Flood

SSE PGA Level	SSE Annual Probability of Exceedance[(b)]	25-Year Flood Probability of Exceedance	Annual Exposure Window Probability[(c)]	Cumulative Annual Probability of Exceedance
0.18g	6.00E-05	4.00E-02	5.48E-03	1.32E-08

Case 2 – Operating Basis Earthquake (OBE) Combined with 500-Year Flood or One-Half Probable Maximum Flood (PMF)

OBE PGA Level	OBE Annual Probability of Exceedance[(b)]	500-Year Flood Probability of Exceedance[(d)]	Annual Exposure Window Probability[(c)]	Cumulative Annual Probability of Exceedance
0.09g	2.20E-04	2.0E-03	5.48E-03	2.41E-09

a) These scenarios are taken from ANSI standard 2.8-1992 with the exception that the 2-year wind has not been included; this standard sets the acceptance level of probability for combined events at 1.0 X 10^{-6} or less.
b) The SSE of 0.18g and OBE of 0.09g correspond to the levels for these earthquake conditions in the original Bellefonte analysis which is consistent with the current Watts Bar and Sequoyah seismic design levels. The SSE and OBE probabilities are based on annual probability of exceedance for mean peak ground acceleration (100 Hz spectral value) shown in Bellefonte FSAR Subsection 2.5.2, Figure 2.5-274.
c) Annual Exposure Window Probability is the probability of the peak flood level, 2 days out of 365 days.
d) The return period for a one-half PMF is greater than 500 years; therefore, for comparison purposes the probability of the more likely 500-year flood is conservatively used here.

TABLE 2.4.4-202

BLN COL 2.4-2 FLOODS FROM POSTULATED SEISMIC FAILURE OF UPSTREAM DAMS

OBE Failures with One-Half PMF		Elevation[a] at BLN
1	Fontana, Hiwassee, Apalachia, Blue Ridge	617.17
2	Norris and Tellico	613.69
3	Cherokee, Douglas, Tellico	614.25
SSE Failures with 25-Year Flood		
4	Norris, Cherokee, Douglas, Tellico	618.28
5	Norris, Douglas, Fort Loudoun, Tellico	613.62

a) Elevation in feet above msl (NGVD 1929)

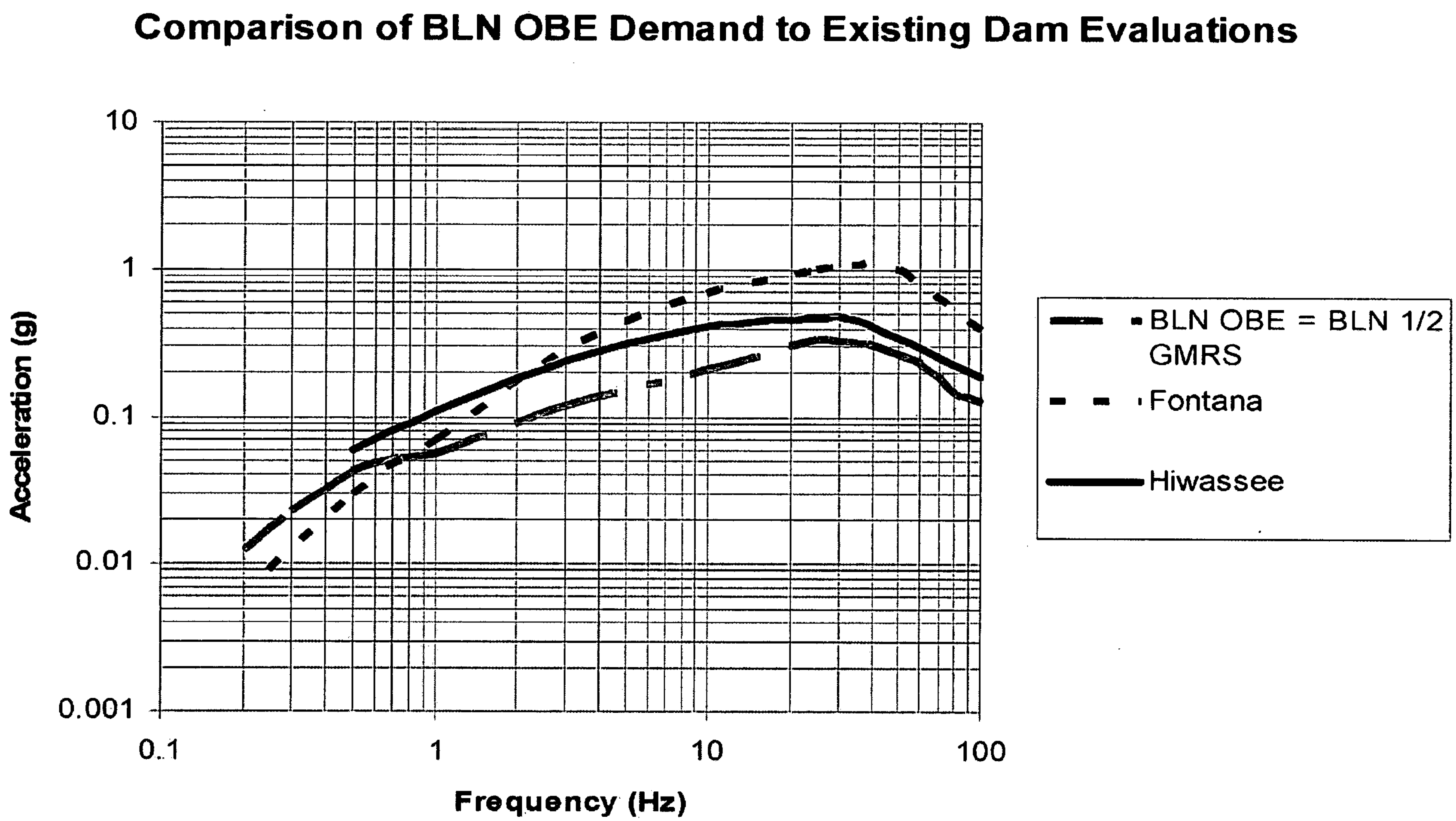


FIGURE 2.4.4-201
Comparison of BLN OBE Demand to Existing Dam Evaluations

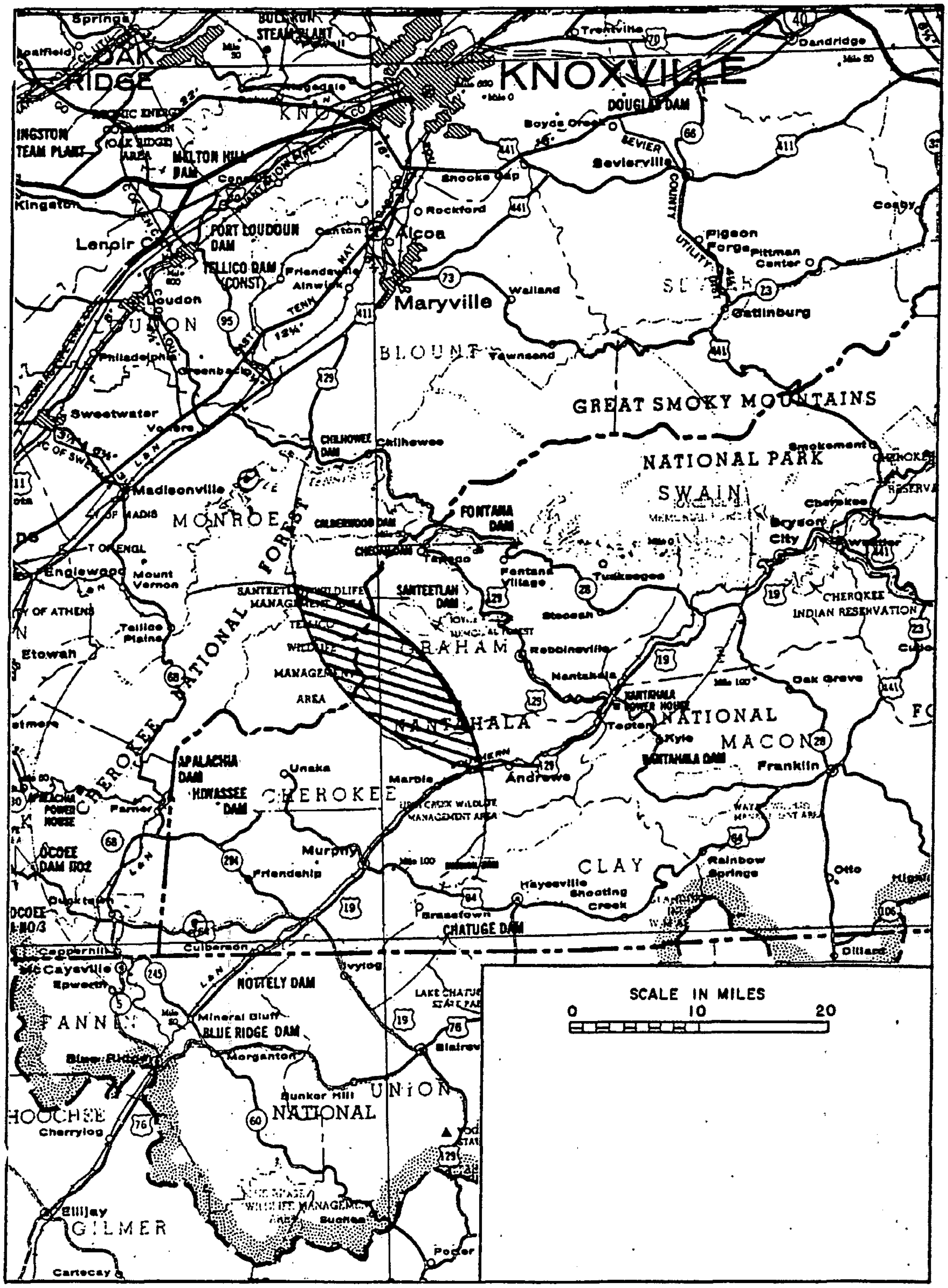


FIGURE 2.4.4-202
OBE with Epicenter Within Area Shown

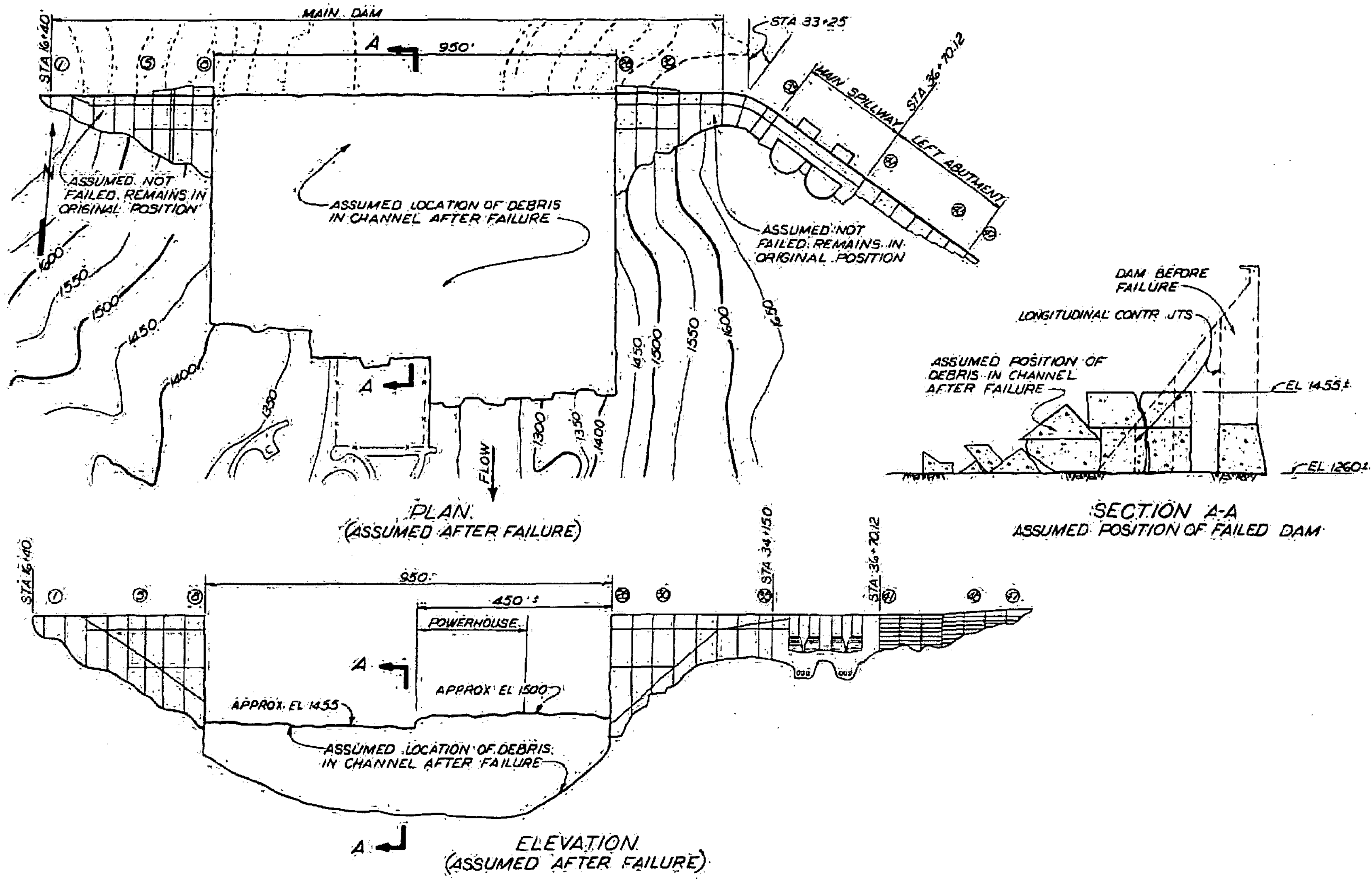


FIGURE 2.4.4-203
Fontana Dam - Postulated Condition of Dam after Failure OBE and One-Half Probable Maximum Flood

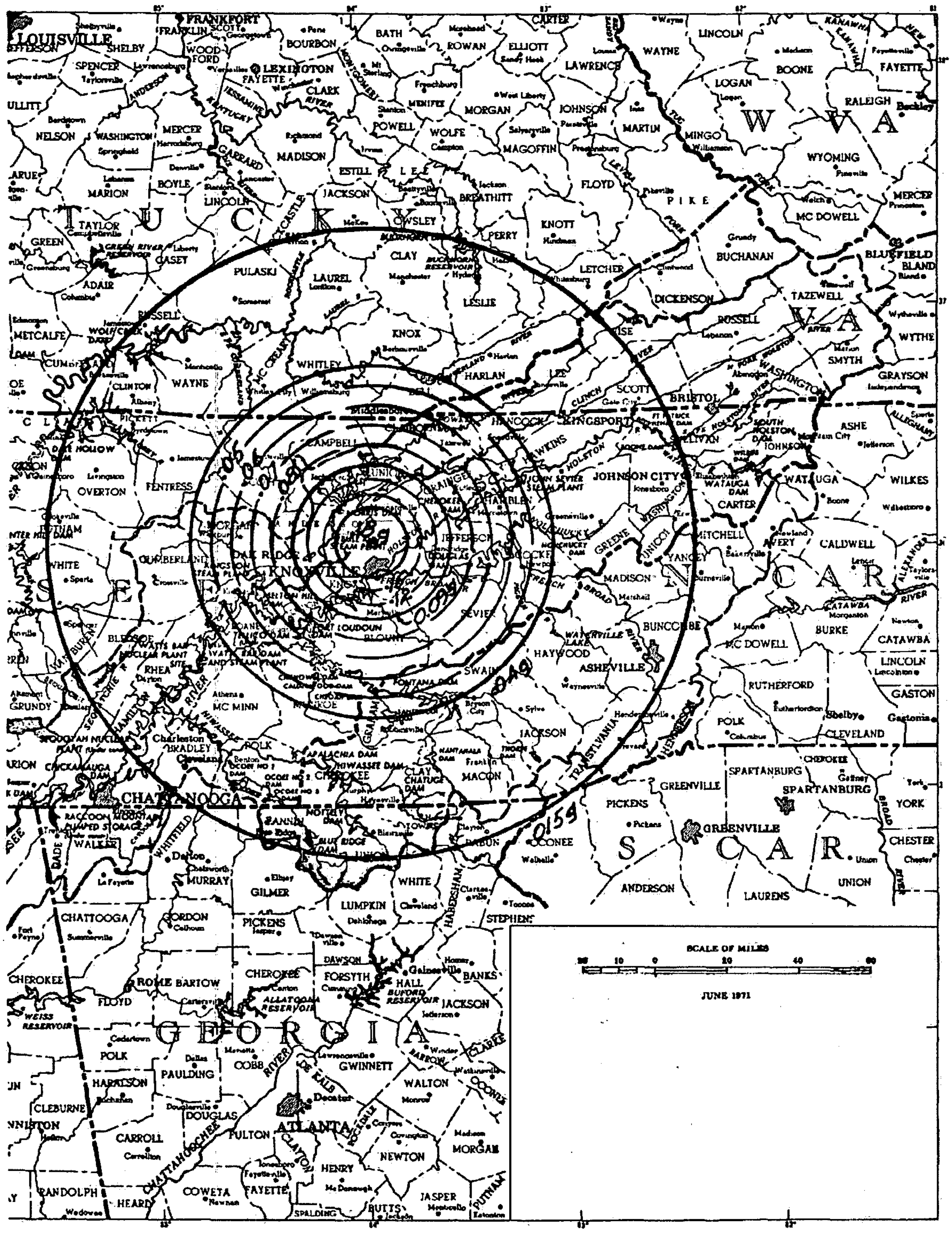


FIGURE 2.4.4-204
SSE with Epicenter in North Knoxville Vicinity

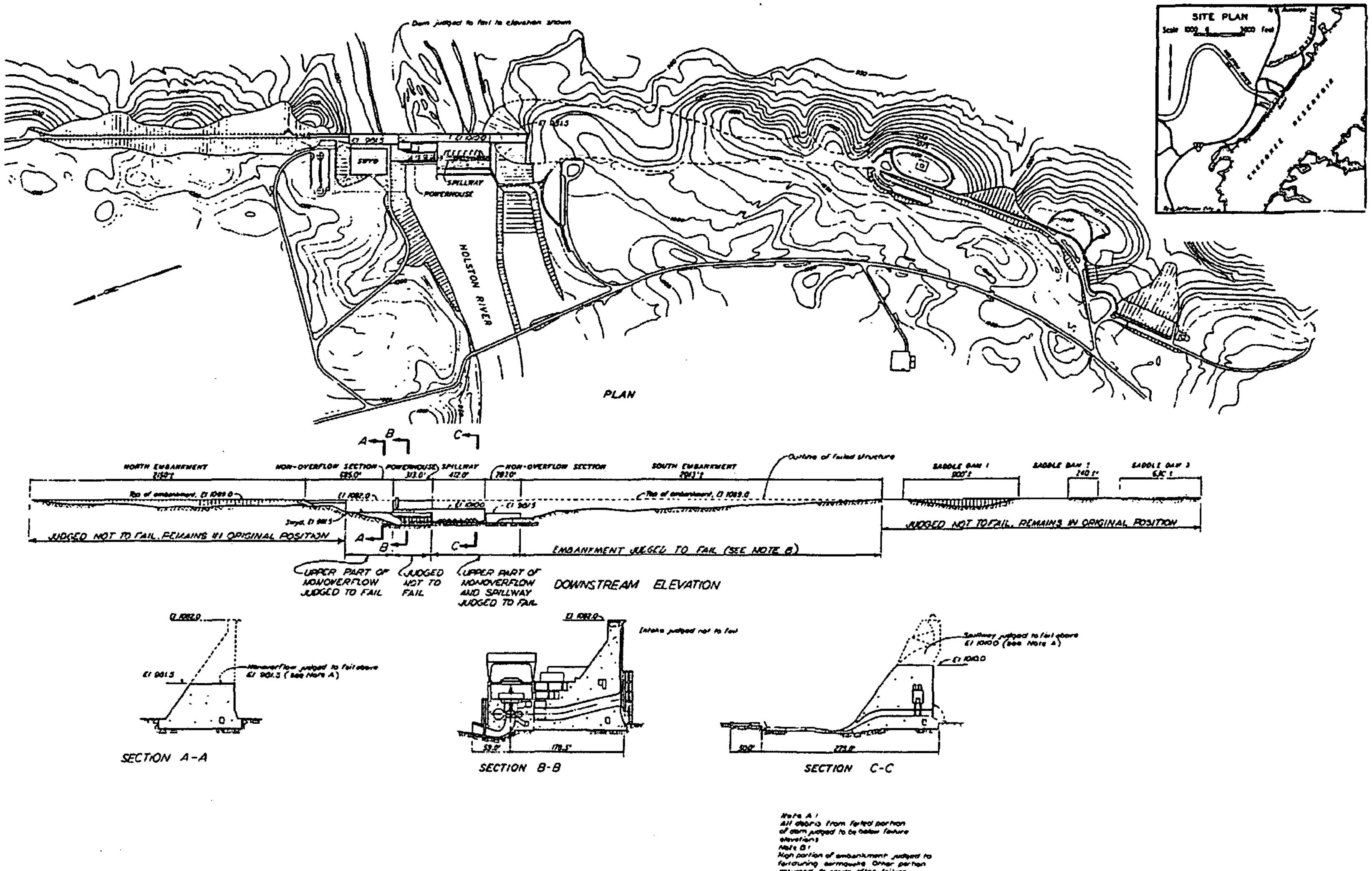
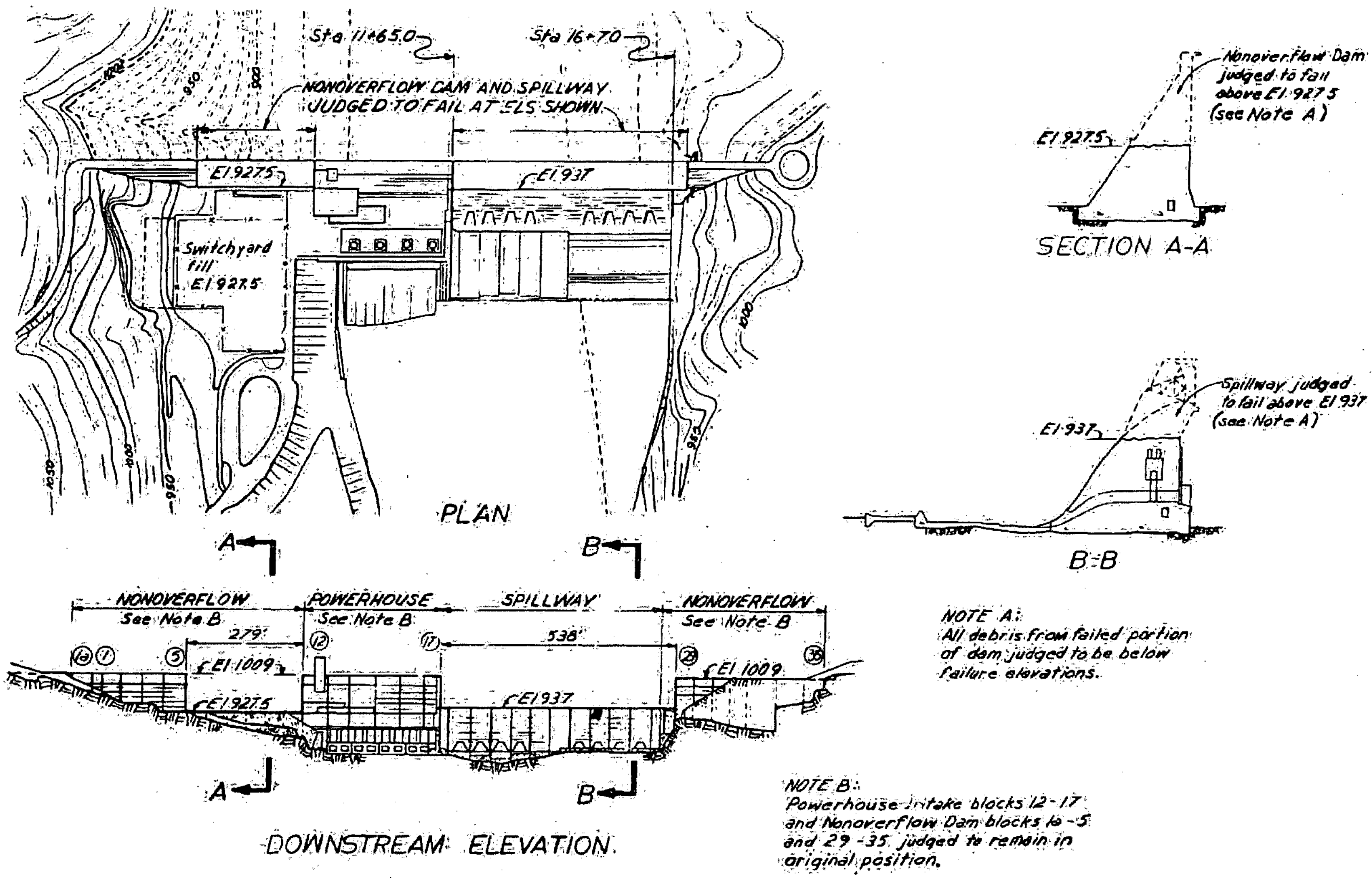


FIGURE 2.4.4-205
Cherokee Dam - Postulated Condition of Dam after Failure OBE and One-Half Probable Maximum Flood



BLN COL 2.4-2

FIGURE 2.4.4-206
Douglas Dam - Postulated Condition of Dam after Failure OBE and One-Half Probable Maximum Flood

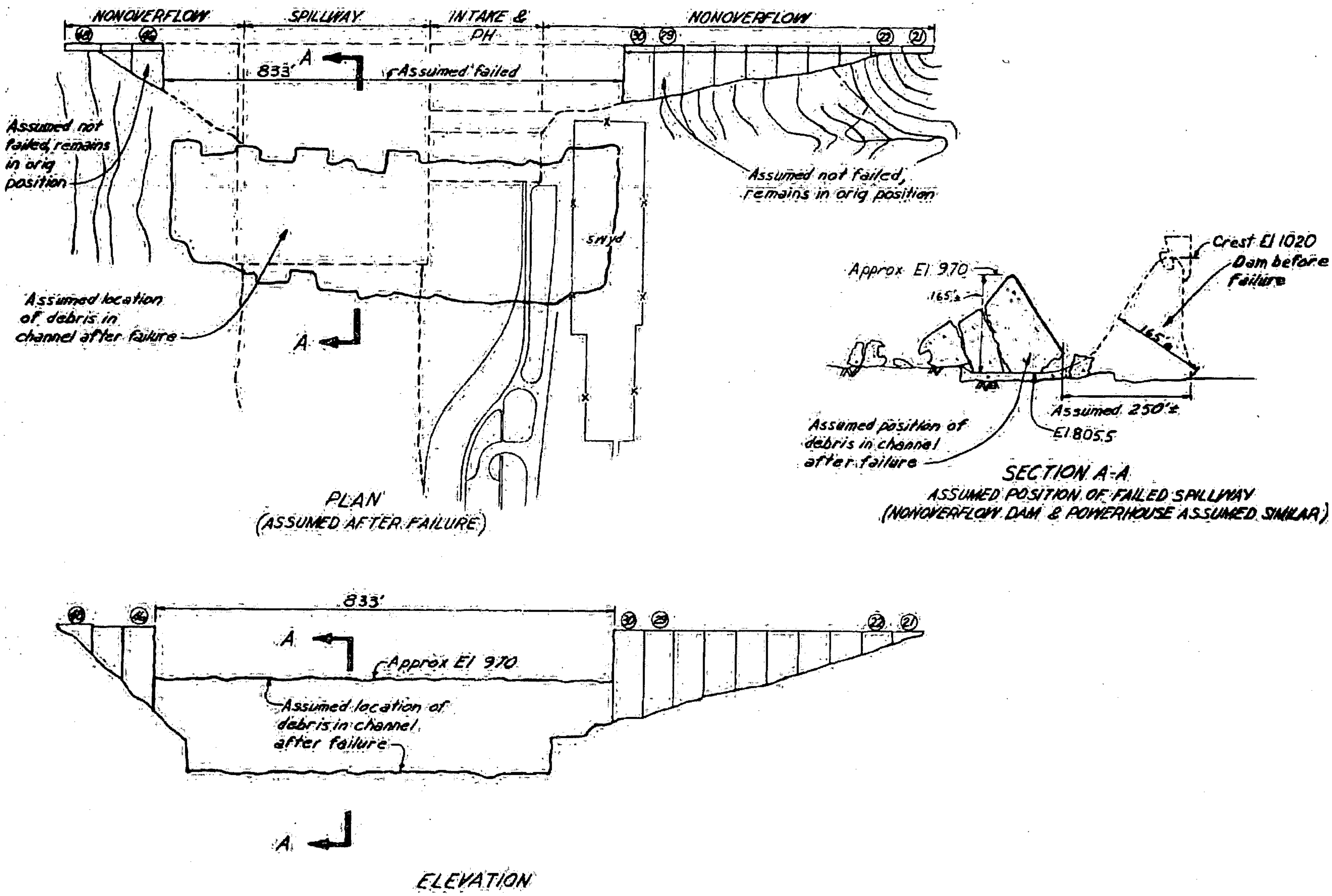


FIGURE 2.4.4-207
Norris Dam - Postulated Condition of Dam after Failure SSE and 25-Year Flood

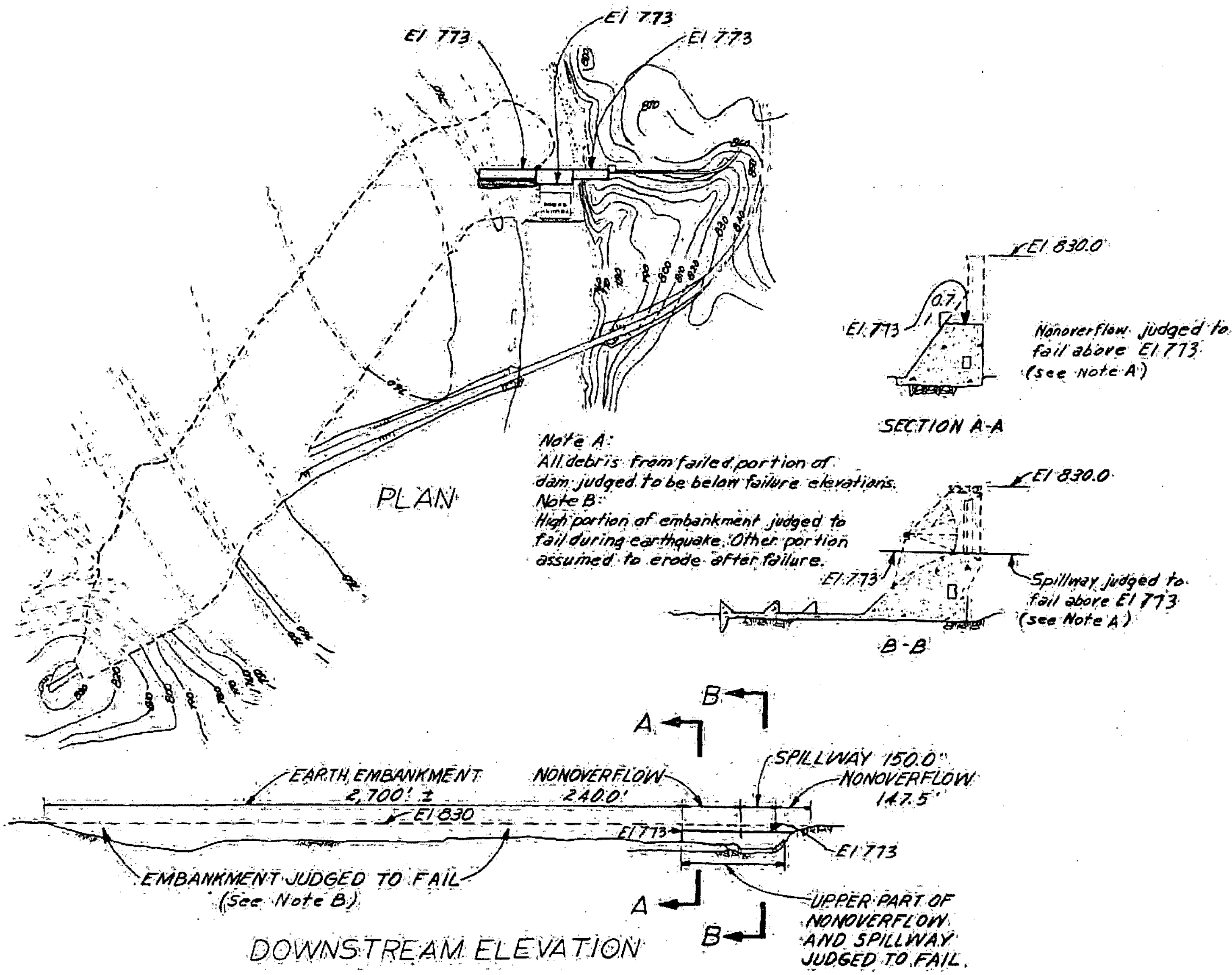


FIGURE 2.4.4-208
Tellico Dam - Postulated Condition of Dam after Failure SSE and 25-Year Flood

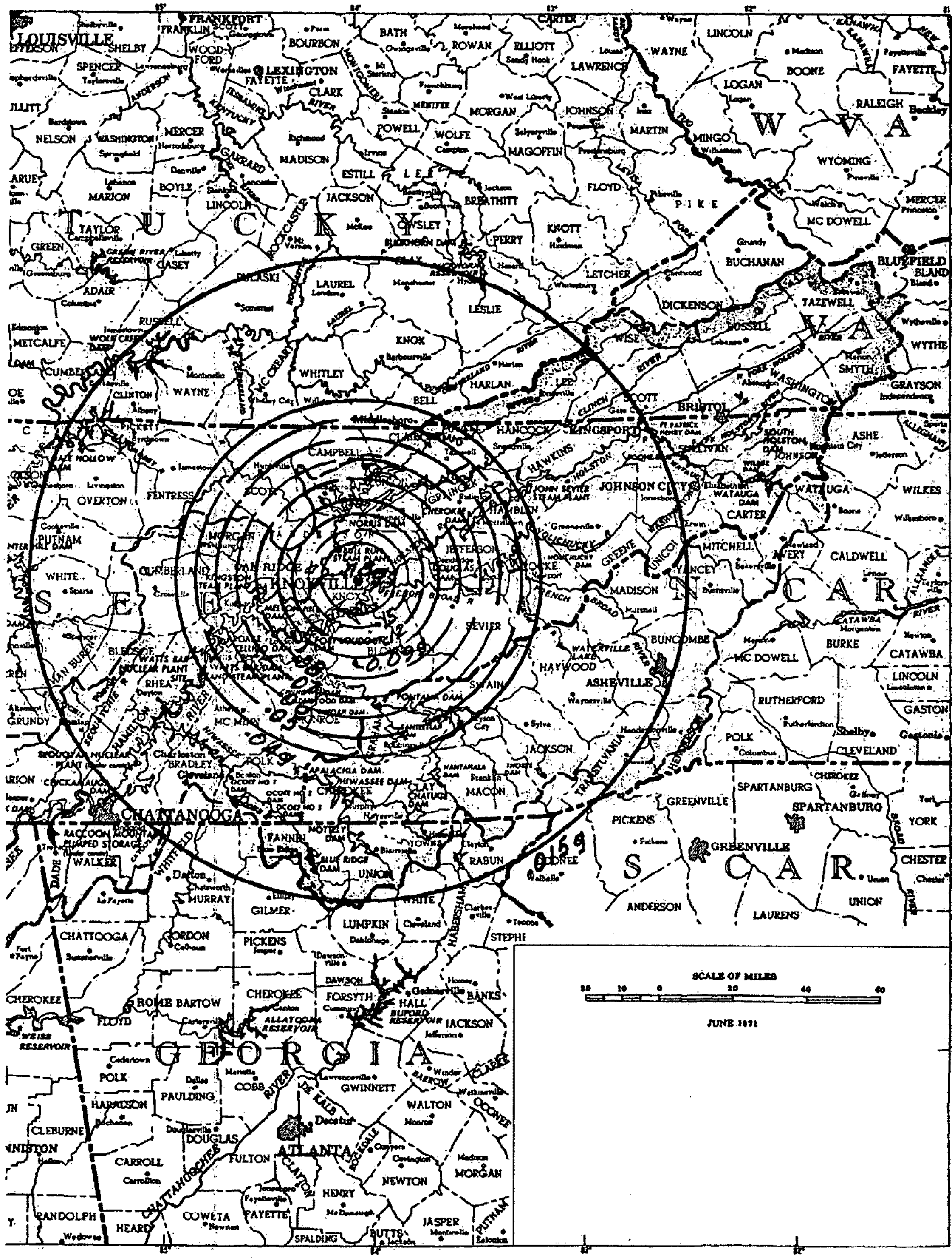


FIGURE 2.4.4-209
SSE with Epicenter in West Knoxville Vicinity

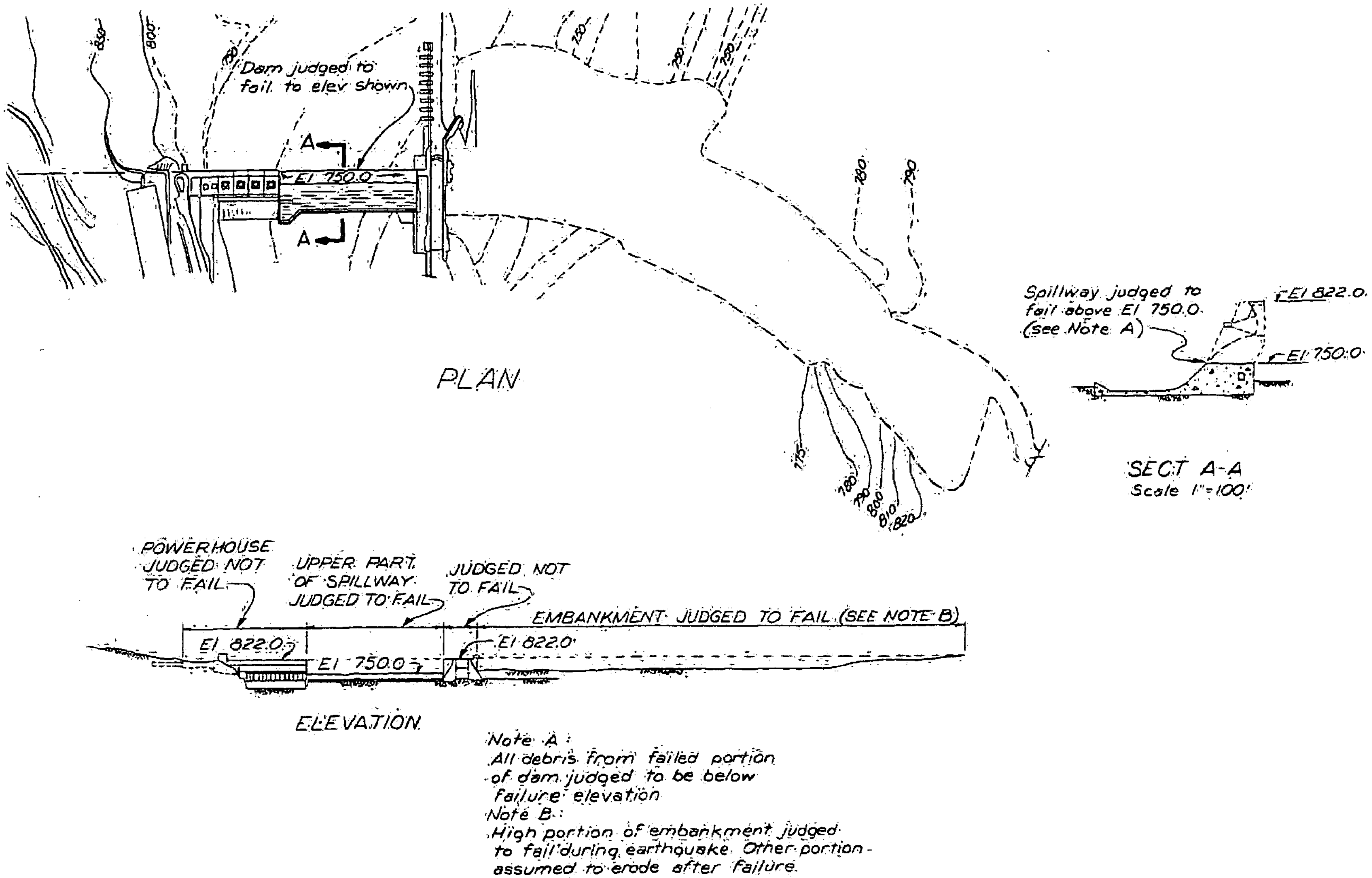


FIGURE 2.4.4-210
Fort Loudoun Dam - Postulated Condition of Dam after Failure SSE and 25-Year Flood

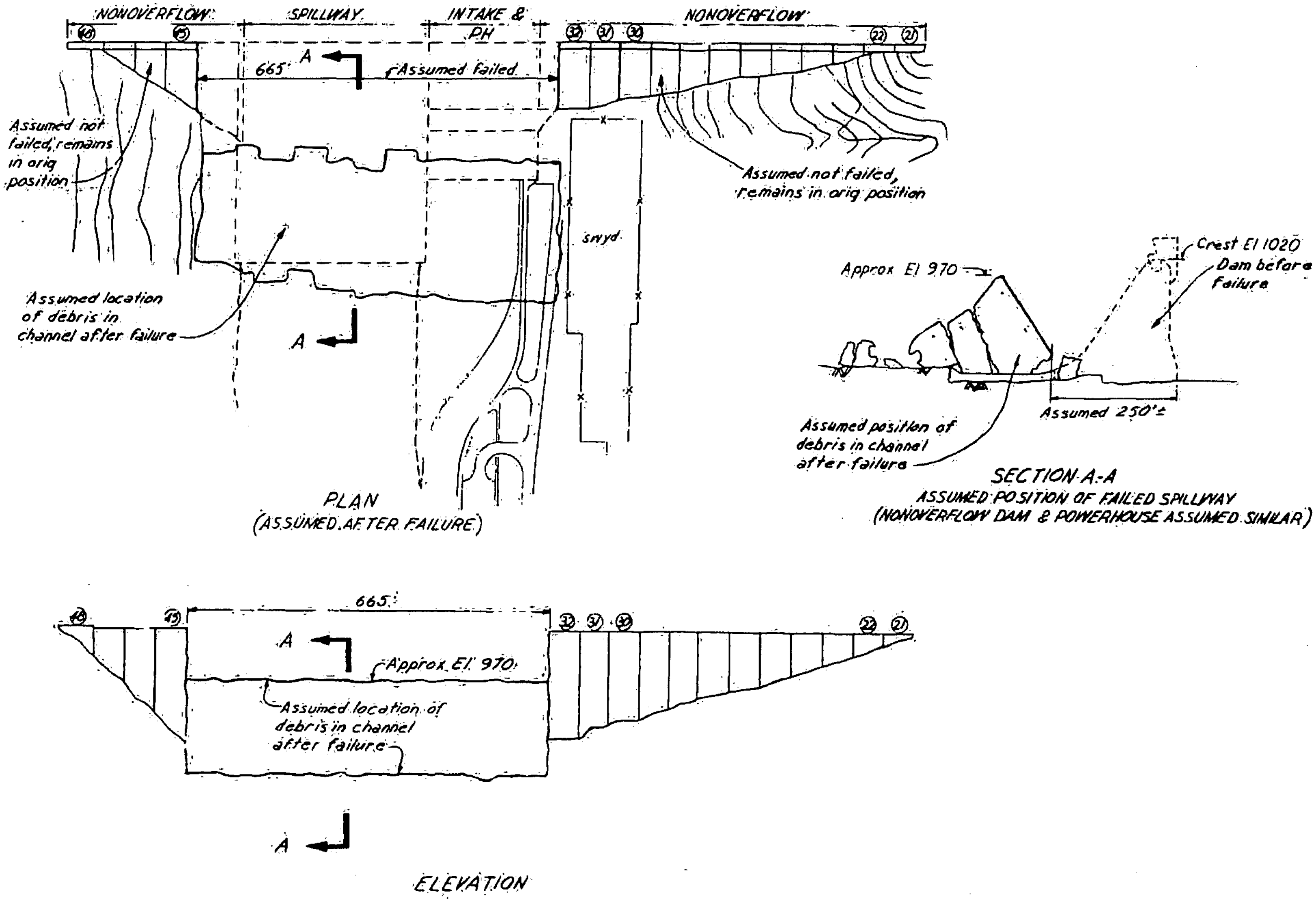
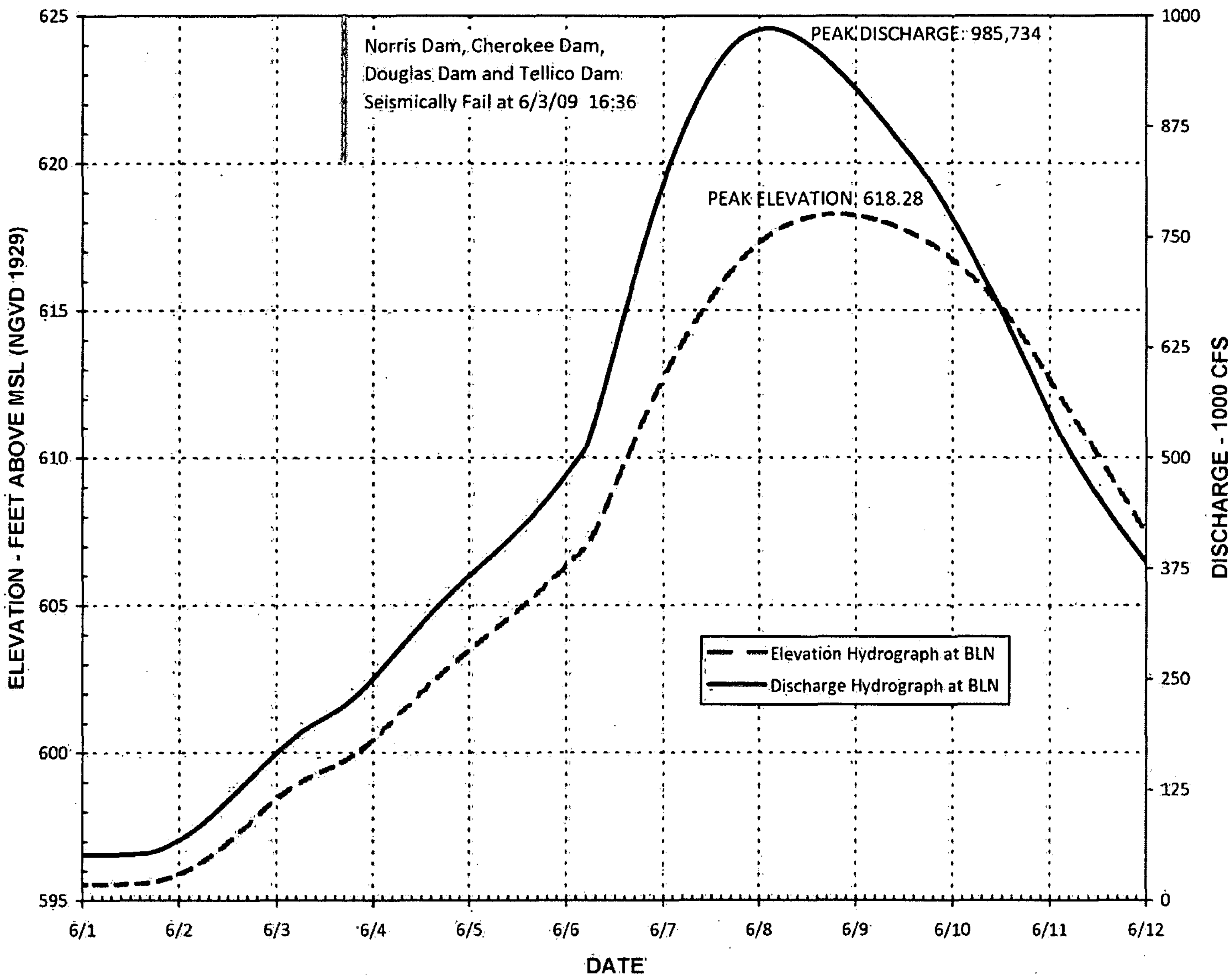


FIGURE 2.4.4-211
Norris Dam - Postulated Condition of Dam after Failure OBE and One-Half Probable Maximum Flood



BLN COL 2.4-2

FIGURE 2.4.4-212
Norris, Cherokee, Douglas, and Tellico Dams in SSE Failure with 25-Year Flood, and Resulting Flood Elevation and Discharge Hydrograph at BLN (TRM 391.5)

Attachment 02.04.04 - 1B

FSAR 2.4.16 Proposed Revision (References)

(6 Pages including Cover Sheet)

2.4.16 References

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Attachment 02.04.04 - 2

Table and Figure Change Roadmap

(3 Pages including Cover Sheet)

Bellefonte Nuclear Plant, Units 3 & 4
COL Application
Part 2, FSAR

FSAR 2.4.4 TABLE RENUMBERING OR CHANGE ROADMAP

Rev 2 Table Number	Title	Renumbering / Change
2.4.4-201	Summary of Floods from Postulated Seismic Failure of Upstream Dams	Removed
2.4.4-202	Cumulative Annual Probability of Exceedance for Seismically-Induced Dam Failure Scenarios	2.4.4-201
2.4.4-203	TVA Dams River Mile Distances	2.4.1-203
2.4.4-204	Facts about Major TVA Dams and Reservoirs	Removed
2.4.4-205	Storage Characteristics of Major TVA Dams and Reservoirs	Removed
2.4.4-206	Facts about Non-TVA Dams and Reservoirs	2.4.1-205

FSAR 2.4.4 FIGURE RENUMBERING OR CHANGE ROADMAP

Rev 2 Figure Number	Title	Renumbering / Change
2.4.4-201	Comparison of BLN OBE Demand to Existing Dam Evaluations	NC
2.4.4-202	Location of Dams	2.4.1-205
2.4.4-203	TVA Water Control System	2.4.1-206

NC – No change