2.4.8 Cooling Water Canals and Reservoirs ~~(HISTORICAL INFORMATION)~~

2.4.8.1 Canals

The intake channel, as shown in Figure 2.1.2-1, referenced in paragraph 2.4.1.1, is designed for a flow of 2,250 cfs. At minimum pool (elevation 675.0 ft), as shown in Figure 2.4.8-1, this flow is maintained at a velocity of 2.7 fps.

The protection of the intake channel slopes from wind-wave activity is afforded by the placement of riprap, shown in Figure 2.4.8-1, in accordance with TVA Design Standards, from elevation 665.0 ft to elevation 690.0 ft. The riprap is designed for a wind velocity of 45 mph.

2.4.8.2 Reservoirs ~~(HISTORICAL INFORMATION)~~

Chickamauga Reservoir provides the cooling water for SQN. This reservoir and the extensive TVA system of upstream reservoirs, which regulate inflows, are described in Table 2.4.1-~~1~~2. The location in an area of ample runoff and the extensive reservoir system assures sufficient cooling waterflow for the plant.

2.4.9 Channel Diversions ~~(HISTORICAL INFORMATION)~~

Channel diversion is not a potential problem for the plant. There are now no channel diversions upstream of SQN that would cause diverting or rerouting of the source of plant cooling water, and none are anticipated in the future. The floodplain is such that large floods do not produce major channel meanders or cutoffs. Carbon 14 dating of material at the high terrace levels shows that the Tennessee River has essentially maintained its present alignment for over 35,000 years. The topography is such that only an unimaginable catastrophic event could result in flow diversion above the plant.

2.4.10 Flooding Protection Requirements

Assurance that safety-related facilities are capable of surviving all possible flood conditions is provided by the discussions given in ~~Paragraph 2.4.2.2~~Section 2.4.14, ~~Section~~ 3.4, ~~Section~~ 3.8.1, 3.8.2, and ~~Appendix 2.4A~~3.8.4.

The plant is designed to be shutdown and remain in a safe shutdown condition for any rainfall flood exceeding plant grade, up to the "design basis flood" discussed in S~~ubs~~ection 2.4.3~~,~~ and for lower, seismic-caused floods discussed in S~~ubs~~ection 2.4.4. Any rainfall flood exceeding plant grade will be predicted at least 27 hours in advance by TVA's Reservoir Operations.

~~-~~Warning of seismic failure of key upstream dams will be available at the plant ~~at least~~approximately 27 hours before a resulting flood surge would reach plant grade. Hence, there is adequate time to prepare the plant for any flood.

See ~~Appendix 2.4A~~Section 2.4.14 for a detailed presentation of the flood protection plan.

2.4.11 Low Water Considerations

Because of its location on Chickamauga Reservoir, maintaining minimum water levels at SQN is not a problem. The high rainfall and runoff of the watershed and the regulation afforded by upstream dams assure minimum flows for plant cooling.

2.4.11.1 Low Flow in Rivers and Streams

The targeted minimum water level at SQN is elevation 675.0 ft~~, which corresponds to the lower bound of the winter operating zone for Chickamauga Reservoir~~ and would occur in the winter flood season as a result of Chickamauga Reservoir operation. On rare occasions, the water level may be slightly lower (.1 or .2 tenths of a foot) for a brief period of time (hours) due to hydropower peaking operations at

Chickamauga and Watts Bar Dams during the winter season. A minimum elevation of 675.0 ft must be maintained in order to provide the prescribed commercial navigation depth in Chickamauga Reservoir.

The "Preferred Alternative" Reservoir Operating Policy was designed to provide increased recreation opportunities while avoiding or reducing adverse impacts on other operating objectives and resource areas. Under the Preferred Alternative, TVA will no longer target specific summer pool elevations at 10 tributary storage reservoirs. Instead, TVA tends to manage the flow of water through the system to meet operating objectives. TVA will use weekly average system flow requirements to limit the drawdown of 10 tributary reservoirs (Blue Ridge, Chatuge, Cherokee, Douglas, Fontana, Nottely, Hiawassee, Norris, South Holston, and Watauga) June 1 through Labor Day to increase recreation opportunities. For four main stem reservoirs (Chickamauga, Guntersville, Wheeler, and Pickwick), summer operating zones will be maintained through Labor Day. For Watts Bar Reservoir, the summer operating zone will be maintained through November 1.

Weekly average system minimum flow requirements from June 1 through Labor Day, measured at Chickamauga Dam, are determined by the total volume of water in storage at the 10 tributary reservoirs compared to the seasonal total tributary system minimum operating guide (SMOG). If the volume of water in storage is above the SMOG, the weekly average system minimum flow requirement will be increased each week from 14,000 cfs (cubic feet per second) the first week of June to 25,000 cfs the last week of July.

Beginning August 1 and continuing through Labor Day, the weekly average flow requirement will be 29,000 cfs. If the volume of water in storage is below the SMOG curve, 13,000 cfs weekly average minimum flows will be released from Chickamauga Dam between June 1 and July 31, and 25,000 cfs weekly average minimum flows will be released from August 1 through Labor Day.

Within these weekly averages, TVA has the flexibility to schedule daily and hourly flows to best meet all operating objectives, including water supply for TVA's thermal power generating plants. Flows may be higher than these stated minimums if additional releases are required at tributary or main river reservoirs to maintain allocated flood storage space or during critical power situations to maintain the integrity and reliability of the TVA power supply system.

In the assumed event of complete dam failure of the north embankment of Chickamauga Dam resulting in a breach width of 400 feet, with the Chickamauga pool at elevation 681.0 ft, the water surface at SQN will begin to drop within one hour and will fall to elevation 641.0 ft about ~~60~~51 hours after failure. TVA will begin providing steady releases of at least 14,000 cfs at Watts Bar within 12 hours of Chickamauga Dam failure to assure that the water level recession at SQN does not drop below elevation 641.0 ft. The estimated minimum river flow requirement for the ERCW system is only 45 cfs.

Reference: Programmatic Environmental Impact Statement, TVA Reservoir Operations Study, Record of Decision, May 2004.

2.4.11.2 Low Water Resulting From Surges, Seiches, or Tsunamis

Because of its inland location on a relatively small, narrow lake, low water levels resulting from surges, seiches, or tsunamis are not a potential problem.

2.4.11.3 Historical Low Water

From the beginning of stream gauge records at Chattanooga in 1874 until the closure of Chickamauga Dam in January 1940, the lowest daily flow in the Tennessee River at SQN was 3,200 cfs on September 7 and 13, 1925. The next lowest daily flow of 4,600 cfs occurred in 1881 and also in 1883.

Since January 1942, low flows at the site have been regulated by TVA reservoirs, particularly by Watts Bar and Chickamauga Dams. Under normal operating conditions, there may be periods of several hours daily when there are no releases from either or both dams, but average daily flows at the site

have been less than 5,000 cfs ~~only 0.65 percent~~about 2.2% of the time and have been less than 10,000 cfs~~, 5.19 percent~~ about 10.4% of the time.

On March 30 and 31, 1968, during special operations for the control of water milfoil, there were no releases from either Watts Bar or Chickamauga Dams during the two-day period. ~~The previous minimum daily flow was 700 cfs on November 1, 1953. TVA no longer conducts special operations for the control of water milfoil on Chickamauga Reservoir~~Over the last 25 years (1986 - 2010) the number of zero flow days at Watts Bar and Chickamauga Dams have been 0 and 2, respectively.

Since January 1940, water levels at the plant have been controlled by Chickamauga~~ Reservoir. Since then,~~ Dam. For the period (1940 - 2010), the minimum level at the dam was elevation 673.3 ft on January 21, 1942. TVA no longer routinely conducts pre-flood drawdowns below elevation 675.0 ft at Chickamauga Reservoir and the minimum elevation in the past 20 years (1987 - 2006) was elevation 674.97 ft at Chickamauga head water.

2.4.11.4 Future Control

Future added controls which could alter low flow conditions at the plant are not anticipated because no sites that would have a significant influence remain to be developed. However, any control that might be considered would be evaluated before implementation.

2.4.11.5 Plant Requirements

2.4.11.5.1 Two-Unit Operation

The safety related water supply systems requiring river water are: the essential raw cooling water (ERCW) (Subsection 9.2.2), and that portion of the high-pressure fire-protection system (HPFP) (Subsection ~~2.4A.4.1~~2.4.14.4.1) supplying emergency feedwater to the steam generators. The fire/flood mode pumps are submersible pumps located in the CCW intake pumping station. The CCW intake pumping station sump is at elevation 648.0 ft. The entrances to the suction pipes for the fire/flood mode pumps are at elevation 651.0 ft~~ feet 0 inches~~ which is 32 feet and 24 feet, respectively, below the maximum normal water elevation of 683.0 ft and the normal minimum elevation of 675.0 ft for the reservoir. Abnormal reservoir level is elevation 670 f~~ee~~t with a technical specification limit of elevation 674 ft. For flow requirements of the HPFP during engineering safety feature operation (Reference 22). The ERCW pump sump in this independent station is at elevation 625.0 ft, which is 58.0~~'~~ ft below maximum normal water elevation, 50.0~~'~~ ft below minimum normal water elevation, and 16~~'~~ ft below the 641~~'~~ ft minimum possible elevation of the river.

Since the ERCW pumping station has direct communication with the river for all water levels and is above probable maximum flood, the ERCW system for two-unit plant operation always operates in an open cooling cycle.

2.4.11.6 Heat Sink Dependability Requirements

The ultimate heat sink, its design bases and its operation, under all normal and credible accident conditions is described in detail in Subsection 9.2.5. As discussed in Subsection 9.2.5, the sink was modified by a new essential raw cooling water (ERCW) pumping station before unit 2 began operation. The design basis and operation of the ERCW system, both with the original ERCW intake station and with the new ERCW intake station, is presented in Subsection 9.2.2. As described in these sections, the new ERCW station is designed to guarantee a continued adequate supply of essential cooling water for all plant design basis conditions. This position is further assured since additional river water may be provided from TVA's upstream multiple-purpose reservoirs, as previously discussed during Low Flow in Rivers and Streams.

2.4.11.6.1 Loss of Downstream Dam

The loss of downstream dam will not result in any adverse effects on the availability of water to the ERCW system or these portions of the original HPFP supplying emergency feedwater to the steam

generator. Loss of downstream dam reduces ERCW flow about 7% to the component cooling and containment spray heat exchangers. ERCW flow does not decrease below that assumed in the analysis (analyzed as 670' to 639') until more than two hours after the peak containment temperature and pressure occurs. (See Section 6.2.1.3.4.)

2.4.11.6.2 Adequacy of Minimum Flow

The cooling requirements for plant safety-related features are provided by the ERCW system. The required ERCW flow rates under the most demanding modes of operation (including loss of downstream dam) as given in Subsection 9.2.2 are contained in TVA calculations and flow diagrams.

Two other safety-related functions may require water from the ultimate heat sink; these are fire protection water (refer to Subparagraph 2.4.11.6.3) and emergency steam generator feedwater (refer to Subsection 10.4.7). These two functions have smaller flow requirements than the ERCW systems. Consequently, the relative abundance of the river flow, even under the worst conditions, assures the availability of an adequate water supply for all safety-related plant cooling water requirements.

River operations methodology for maintaining UHS temperatures are discussed in "Monitoring and Moderating Sequoyah Ultimate Heat Sink," Reference 21.

2.4.11.6.3 Fire-Protection Water

Refer to the Fire Protection Report discussed in Section 9.5.1.

2.4.12 Environmental Acceptance of Effluents

The ability of surface waters near SQN, located on the right bank near Tennessee River Mile (TRM) 484.5, to dilute and disperse radioactive liquid effluents accidentally released from the plant is discussed herein. Routine radioactive liquid releases are discussed in Section 11.2.

The Tennessee River is the sole surface water pathway between SQN and surface water users along the river. Liquid effluent from SQN flows into the river from a diffuser pond through a system of diffuser pipes located at TRM 483.65. An accidental, radioactive liquid effluent release from SQN would enter the Tennessee River after it reached the diffuser pond and entered the diffuser pipes. The contents of the diffuser pond enter the diffuser pipes and mix with the river flow upon discharge. The diffusers are designed to provide rapid mixing of the discharged effluent with the river flow. The flow through the diffusers is driven by the elevation head difference between the diffuser pond and the river [1] (McCold 1979). Descriptions of the diffusers and SQN operating modes are given in Paragraph 10.4.5.2. Flow is discharged into the diffuser pond via the blowdown line, ERCW System (Subsection 9.2.2) and CCW System (Subsection 10.4.5). A layout of SQN is given in Figures 2.1.2-1 and 2.1.2-2. Two pipes comprise the diffuser system and are set alongside each other on the river bottom. They extend from the right bank of the river into the main channel. The main channel begins near the right bank of the river and is approximately 900 feet wide at SQN [1] (McCold, 1979). Each diffuser pipe has a 350-foot section through which flow is discharged into the river. The downstream diffuser leg discharges across a section 0 to 350 feet from the right bank of the main channel. The upstream diffuser leg starts at the end of the downstream diffuser leg and discharges across a section 350 to 700 feet from the right bank of the main channel. The two diffusers therefore provide mixing across nearly the entire main channel width.

The river flow near SQN is governed by hydro power operations of Watts Bar Dam upstream (TRM 529.9) and Chickamauga Dam downstream (TRM 471.0). The backwater of Chickamauga Dam extends to Watts Bar Dam. Peaking hydro power operations of the dams cause short periods of zero (i.e., stagnant) and reverse (i.e., upstream) flow near the plant. Effluent released from the diffusers during these zero and reverse flow periods will not concentrate near the plant or affect any water intake upstream. The maximum flow-reversal during 1978-1981 were not long enough to cause discharge from the diffusers to extend upstream to the SQN intake [2] (El-Ashry, 1983), which is the nearest intake and located at the right bank near TRM 484.7. Moreover, the warm buoyant discharge from the diffusers will tend toward the water surface as it mixes the river flow and away from the

cooler, denser water found near the intake opening below the skimmer wall. The intake opening extends the first 10 feet above the riverbed elevation of about 631 feet mean sea level (MSL). The minimum flow depth at the intake is approximately 45 feet [3] (Ungate and Howerton, 1979). There are no other surface water users between the diffusers and this intake.

Subsection 2.4.13 discusses groundwater movement at SQN. Effluent released through the diffusers will have no impact on SQN groundwater sources along the banks of the river. Paragraph 2.2.3.8 discusses the effect on plant safety features from flammable or toxic materials released in the river near SQN.

The predominant transport and effect of a diffuser release is along the main channel and in the downstream direction. The nearest downstream surface water intake is located along the left bank at TRM 473.0 (Table 2.4.1-~~4~~1).

A mathematical analysis is used to estimate the downstream transport and dilution of a contaminant released in the Tennessee River during an accidental spill at SQN. Only the main channel flow area without the adjacent overbank regions is considered in the analysis. The mathematical analysis of a potential spill scenario can involve: (1) a slug release, which can be modeled as an instantaneous release; (2) a continuous release, which can be modeled as a steady-state release; (3) a bank release, which can be modeled as a vertical line source; and (4) a diffuser release, which can be modeled either as a vertical line or plane source, depending on the width of the diffuser with respect to the channel width.

The following assumptions are used in the mathematical analyses to compute the minimum dilution expected downstream from SQN and, in particular, at the nearest water intake.

1. Mixing calculations are based on unstratified steady flow in the reservoir. River flow, Q, is assumed to be 27,474 cubic feet per second (cfs), which is equalled or exceeded in the reservoir approximately 50 percent of the time (Paragraph 2.4.1.2). Because various combinations of the upstream and downstream hydro power dam operations can create upstream flows past SQN, a minimum flow is not well defined. Larger (smaller) flows will decrease (increase) the travel time to the nearest intake but cause less than an order of magnitude change in the calculated dilution.

2. Because the SQN diffusers and the nearest downstream water intake are on opposite banks of the river, and the diffusers extend across most of the main channel width, an analysis using a diffuser release (rather than a bank release) is selected to yield a lesser (i.e., more conservative) dilution at the intake. Thus, the accidental spill is modeled as a vertical plane source across the width of the main channel.

3. The contaminant concentration profile from a slug release is assumed to be Gaussian (i.e., normal) in the longitudinal direction.

4. The contaminant is conservative, i.e., it does not degrade through radioactive decay, chemical or biological processes, nor is it removed from the reservoir by adsorption to sediments or by volatilization.

5. The transport of the contaminant is described using the motion of the river flow, i.e., the contaminant is neutrally buoyant and does not rise or sink due to gravity.

The main channel and dynamic, flow-dependent processes of the reservoir reach between SQN and the first downstream water intake are modeled as a channel of constant rectangular cross section with the following constant geometric, hydraulic and dispersion characteristics.

Longitudinal distance, x = 10.6 miles

Average water surface elevation = 678.5 feet MSL (Figure 2.4.1-~~3~~4 (1))

Average width, W = 1175 feet

Average depth, H = 50 feet

Average velocity, U (= Q/(W H)) = 0.468 feet per second (fps)

Average travel time (for approximate peak contaminant), t (= x/U) = 1.4 days

Manning coefficient n (surface roughness) = 0.03

Longitudinal dispersion parameter, alpha = 200

where: alpha = E_x / (H u)

E_x = constant longitudinal dispersion coefficient (square feet per second)

u = shear velocity (fps) = $\sqrt{gRS}$

g = acceleration due to gravity = 32.174 ft/s^2

R = hydraulic radius (ft)

S = slope of the energy line (ft/ft)

The average width and depth were estimated from measurements of 9 cross sections in the reach [4] (TVA) [5] (TVA). For wide channels (i.e., large width-to-depth ratio), the hydraulic radius can be approximated as the average depth. The value of alpha = 200 is on the conservative (i.e., low) side [6] (Fischer, et al., 1979). The value of the Manning coefficient n is representative for natural rivers [7] (Chow, 1959).

The equation used to describe the maximum downstream activity (or concentration), C, at a point of interest due to an instantaneous plane source release of volume V is [8] (Guide 1.113):

$$\frac{C}{C_o} = \frac{V}{WH\sqrt{4\pi E_x t}} \qquad (2.4.12\text{-}1)$$

where:

C_o = initial activity (or concentration) in the plant of the released contaminant

π = 3.14156

Any consistent set of units can be used on each side of Equation 2.4.12-1 (e.g., C and C_o in mCi/ml; V in cf; W and H in ft; E_x in ft^2/s; t in s).

The term, C/C_o, is the relative (i.e., dimensionless) activity (or concentration) and its reciprocal is the dimensionsless dilution factor. Equation 2.4.12-1 simplifies to C/C_o = 8.3E-10 * V (V expressed in cubic feet (cf)) when the parameters are substituted and the Manning equation [7] (Chow, 1959) is used in the definition of the shear velocity, u. In the substitution, u = 0.028 ft/s and E_x = 282.1 ft^2/s.

The equation used to describe the maximum downstream concentration at a point of interest due to a continuous plane source release rate, Q_s, where Q_s << Q, is [8] (Guide 1.113):

$$\frac{C}{C_o} = \frac{Q_s}{Q} \qquad (2.4.12\text{-}2)$$

Any consistent set of units can be used on each side of Equation 2.4.12-2 (e.g., C and C_o in mCi/ml; Q_s and Q in cfs).

Equation 2.4.12-2 simplifies to C/C_o = 3.64E-05 * Q_s (Q_s expressed in cfs) for Q = 27,474 cfs.

Examples of quantities and concentrations of potential contaminant releases and the use of Equations 2.4.12-1 and 2.4.12-2 follow. Because C_o is defined as the in-plant activity (or concentration) and not that of the diffuser release, an estimate of the dilution of liquid waste occurring in the diffuser pond and diffuser pipes is not needed. This is because the flow available for dilution in the plant (e.g., CCW and ERCW) is taken from and returned to the river. Only effluent extraneous to the river flow requires consideration in the analyses to calculate the dilution. More information on the possible means which liquid waste from the plant enters the diffuser pond is contained in Subsection 10.4.5.

The largest outdoor tanks whose contents flow into the diffuser pond are the two condensate storage tanks (Paragraph 11.2.3.1), which each have an overflow capacity of 398,000 gallons. Liquid waste that reaches the diffuser pond enters the Tennessee River through the diffuser system. The diffuser pond is approximately 2000 feet long and 500 feet wide with a depth that, although it depends on the Chickamauga Reservoir elevation, averages about 10 feet [9] (McIntosh, et al., 1982). The design flow residence time of the pond is approximately one hour (i.e., diffuser design flow is 2,480 cfs at maximum plant capacity [3] [Ungate and Howerton, 1979]).

For example, assume an instantaneous plane source release into the Tennessee River of the contents of one condensate storage drain tank. Assume the full 398,000 gallon (53,210 cf) volume contains Iodine-131 (I-131) at an activity of 1.5E-06 mCi/gm (Table 10.4.1-1). From Equation 2.4.12-1, the activity, C, at the first downstream water intake would be 6.6E-11 mCi/gm, which is within the acceptable limit [10] (CFR) for soluble I-131.

For a continuous plane source release, assume the contents of the 398,000 gallon (53,210 cf) floor drain tank leak out steadily over a 24-hour period. The effective release rate is 0.6 cfs at an activity of 1.5E-06 mCi/gm. The expected activity at the first downstream water intake would be 3.4E-11 mCi/gm using Equation 2.4.12-2 and is within the acceptable limit [10] (CFR) for soluble I-131.

REFERENCES (for Section 2.4.12 only)

[1] McCold, L. N. (March 1979), "Model Study and Analysis of Sequoyah Nuclear Plant Submerged Multiport Diffuser," TVA, Division of Water Resources, Water System Development Branch, Norris, TN, Report No. WR28-1-45-103.

[2] El-Ashry, Mohammed T., Director of Environmental Quality, TVA, February 1983 letter to Paul Davis, Manager, Permit Section, Tennessee Division of Water Quality Control, SEQUOYAH NUCLEAR PLANT---NPDES PERMIT NO. T0026450.

[3] Ungate, C. D., and Howerton, K. A. (April 1978; revised March 1979), "Effect of Sequoyah Nuclear Plant Discharges on Chickamauga Lake Water Temperatures," TVA, Division of Water Management, Water Systems Development Branch, Norris, TN, Report No. WR28-1-45-101.

[4] TVA, Chickamauga Reservoir Sediment Investigations, Cross Sections, 1940-1961, Division of Water Control Planning, Hydraulic Data Branch.

[5] TVA, Measured Cross Sections of Chickamauga Reservoir, 1972, Flood Protection Branch.

[6] Fischer, H. B., List, E. J., Koh, R.C.Y., Imberger, J., Brooks, N. H. (1979), Mixing in Inland and

Costal Waters, Academic Press, New York.

[7] Chow, V. T. (1959) Open-Channel Hydraulics, McGraw-Hill, New York.

[8] United States Nuclear Regulatory Commission, Office of Standards Development, Regulatory Guide 1.113 (April 1977), "Estimating Aquatic Dispersion of Effluents from Accidental and Routine Reactor Releases for the Purpose of Implementing Appendix I," Revision 1.

[9] McIntosh, D. A., Johnson, B. E. and Speaks, E. B. (October 1982), "A Field Verification of Sequoyah Nuclear Plant Diffuser Performance Model: One-Unit Operation," TVA, Office of Natural Resources, Division of Air and Water Resources, Water Systems Development Branch, Norris, TN, Report No. WR28-1-45-110.

[10] 10 CFR Part 20, Appendix B, Table II, Column 2.

[11] TVA SQN Calculation SQN-SQS2-0242, SQN Site Iodine-131 Release Concentration in Tennessee River.

2.4.13 Groundwater ~~(HISTORICAL INFORMATION)~~

2.4.13.1 Description and Onsite Use

The peninsula on which SQN is located is underlain by the Conasauga Shale, a poor water-bearing formation. About 2,000 feet northwest of the plant site, the trace of the Kingston Fault separates this outcrop area of the Conasauga Shale from a wide belt of Knox Dolomite. The Knox is the major water bearing formation of eastern Tennessee.

Groundwater in the Conasauga Shale occurs in small openings along fractures and bedding planes; these rapidly decrease in size with depth, and few openings exist below a depth of 300 feet. Groundwater in the Knox Dolomite occurs in solutionally enlarged openings formed along fractures and bedding planes and also in locally thick cherty clay overburden.

There is no groundwater use at SQN.

2.4.13.2 Sources

The source of groundwater at SQN is recharged by local, onsite precipitation. Discharge occurs by movement mainly along strike of bedrock, to the northeast and southwest, into Chickamauga Lake. Rises in the level of Chickamauga Lake result in corresponding rises in the water table and recharge along the periphery of the lake, extending inland for short distances. Lateral extent of this effect varies with local slope of the water table, but probably nowhere exceeds 500 feet. Lowering levels of Chickamauga Lake results in corresponding declines in the water table along the lake periphery, and short-term increase in groundwater discharge.

When SQN was initially evaluated in the early 1970s, it was in a rural area, and only a few houses within a two-mile radius of the plant site were supplied by individual wells in the Knox Dolomite (see Table 2.4.13-1, Figure 2.4.13-1). Because the average domestic use probably does not exceed 500 gallons per day per house, groundwater withdrawal within a two-mile radius of the plant site was less than 50,000 gallons per day. Such a small volume withdrawal over the area would have essentially no effect on areal groundwater levels and gradients. Although development of the area has increased, public supplies are available and overall groundwater use is not expected to increase.

Public and industrial groundwater supplies within a 20 mile radius of the site in 1985 are listed in Table 2.4.13-2. The area groundwater gradient is towards Chickamauga Lake, under water table conditions, and at a gradient of less than 120 feet per mile. The water table system is shallow, the surface of which conforms in general to the topography of the land surface. Depth to water ranges from less than 10 feet in topographically low areas to more than 75 feet in higher areas underlain by Knox Dolomite. Figure 2.4.13-2 is a generalized water-table map of SQN, based on water level data from

five onsite observation wells, and in private wells adjacent to the site in April 1973, and also based on surface resistivity measurements of depth to water table made in 1972.

Because permeability across strike in the Conasauga Shale is extremely low, and nearly all water movement is in a southwest-northeast direction, along strike, the Conasauga-Knox Dolomite

Contact is a hydraulic barrier, across which only a very small volume of water could migrate in the event large groundwater withdrawals were made from the adjacent Knox.

Although some water can cross this boundary, the permeability normal to strike of the Conasauga is too low to allow development of an areally extensive cone of depression.

Groundwater recharge occurs to the Conasauga Shale at the plant site. Recharge water moves no more than 3,000 feet before being discharged to Chickamauga Lake.

2.4.13.3 Accident Effects

Design features in SQN further protect groundwater from contamination.

Category I structures in the SQN facility are designed to assure that all system components perform their designed function, including maintenance of integrity during earthquake.

Buildings in which radioactive liquids could be released due to the equipment failure, overflow, or spillage are designed to retain such liquids even if subject to an earthquake equivalent to the safe shutdown earthquake. Outdoor tanks that contain radioactive liquids are designed so that if they overflow, the overflow liquid is redirected to the building where the liquid is collected in the radwaste system. Two outdoor tanks that contain low concentrations of radioactivity at times overflow to yard drains which discharge into the diffuser pond. Overflow liquid is discharged near the discharge diffuser.

The capacity for dispersion and dilution of contaminants by the groundwater system of the Conasauga Shale is low. Dispersion would occur slowly because water movement is limited to small openings along fractures and bedding planes in the shale. Clay minerals of the Conasauga Shale do, however, have a relatively high exchange capacity, and some of the radioactive ions would be absorbed by these minerals. Any ions moving through the groundwater system eventually would be discharged to Chickamauga Lake.

The Conasauga Shale is heterogeneous and anisotropic vertically and horizontally. Water-bearing characteristics change abruptly within short distances. Standard aquifer analyses cannot be applied, and meaningful values for permeability, time of travel, or dilution factors cannot be obtained.

Bedrock porosity is estimated to be less than 3 percent based on examination of results of exploratory core drilling. It is known from experience elsewhere in this region that water movement in the Conasauga Shale occurs almost entirely parallel to strike. Subsurface movement of a liquid radwaste release at the plant site would be about 1,000 feet to the northeast or about 2,000 feet to the southwest before discharge to Chickamauga Lake.

Time of travel can only be estimated as being a few weeks for first arrival, a few months for peak concentration arrival, and perhaps two or more years for total discharge. The computed mean time of travel of groundwater from SQN to Chickamauga Lake is 303 days.

No radwaste discharge would reach a groundwater user. At the nearest point, the reservation boundary lies 2,200 feet northwest of the plant site, across strike. Groundwater movement will not occur from the plant site in this direction across this distance.

During initial licensing, the radionuclide concentrations were determined for both groundwater and surface water movement to the nearest potable water intake (Savannah Valley Utility District, which is no longer in service) and found to be of no concern (see Safety Evaluation Report, March 1979,

Section 2.4.4 Groundwater).

2.4.13.4 Monitoring or Safeguard Requirements

SQN is on a peninsula of low-permeability rock; the groundwater system of the site is essentially hydraulically isolated and potential hazard to groundwater users of the area is minimal. The environmental radiological monitoring program is addressed in Section 11.6.

Monitor wells 1, 2, 3, and 4 were sampled and analyzed for radioactivity during the period from 1976 through 1978. Well 5 was not monitored because of insufficient flow. An additional well (Well 6) was drilled in late 1978 downgradient from the plant and a pump sampler installed.

Wells 1, 2, 4, and 5 are each 150 feet deep, Well 6 is 250 feet deep, and Wells L6 and L7 are 75-80 feet deep. All of the wells are cased in the residuum and open bore in the Conasauga Shale.

2.4.13.5 Conclusions

SQN was designed to provide protection of groundwater resources by preventing the escape of the leaks of radionuclides. Site soils and underlying geology provide further protection in that they retard the movement of water and attenuate any contaminants that would be released. All groundwater movement is toward Chickamauga Lake. The Knox Dolomite is essentially hydraulically separated from the Conasauga Shale; therefore, offsite pumping, including future development, should have little effect upon the groundwater table in the Conasauga Shale at the plant.

Even though the potential for accidental contamination of the groundwater system is extremely low, the radiological monitoring program will provide ample lead times to mitigate any offsite contamination.

As a consequence of the geohydrologic conditions that remain unchanged from evaluations conducted in the 1970s, the information in Chapter 2.4.13 Groundwater is historical and should not be subject to updating revisions.

2.4.14 ~~Technical Requirements and Emergency Operation~~Flooding Protection Requirements

~~Emergency flood protection plans, designed to minimize impact of floods above plant grade on safety-related facilities, are described in Appendix 2.4A. Procedures for predicting rainfall floods, arrangements to warn of upstream dam failure floods, and lead times available and types of action to be taken to meet related safety requirements for both sources of flooding are described therein. The Technical Requirements Manual specify the action to be taken to minimize the consequences of floods.~~The plant grade elevation at SQN can be exceeded by large rainfall and seismically-induced dam failure floods. Assurance that SQN can be safely shut down and maintained in these extreme flood conditions (Section 2.4.2.2 and this Section 2.4.14) is provided by the discussions given in Sections 3.4, 3.8.1, and 3.8.4.

~~2.4A.1~~2.4.14.1 Introduction

This ~~appendix~~subsection describes the methods by which ~~the Sequoyah Nuclear Plant~~SQN will be made capable of tolerating floods above plant grade without jeopardizing public safety. Since flooding of this magnitude, as explained in ~~section 2.4,~~Sections 2.4.2 and 2.4.4, is most unlikely, extreme steps are considered acceptable including actions that create or allow extensive economic damage to the plant. The actions described herein will be implemented for floods ranging from slightly below plant grade, to allow for wave runup~~,~~ to the Design Basis Flood (DBF).

~~2.4A.1.1~~2.4.14.1.1 Design Basis Flood

The DBF is the calculated upper limit flood that includes the probable maximum flood (PMF) plus the wave runup caused by a 45-mile-per-hour overwater wind; this is discussed in subsection 2.4.3.6. The table below gives representative levels of the DBF at different plant locations.

Design Bases Flood (DBF) Levels

Probable maximum flood (still reservoir)	~~719.6~~722.0 ft
DBF runup on Diesel Generator Building	723.2 ft
DBF runup on vertical external, unprotected walls	~~723.8~~726.2 ft
DBF surge level within flooded structures	~~720.1~~722.5 ft

The lower flood elevations listed above are actual DBF elevations and are not normally used for the purpose of design but are typically used in plant procedures including procedures which direct plant actions in response to postulated DBF. For purposes of designing the flood protection for systems, structures, and components, the following higher elevations should be used thus ensuring additional margin has been included in the development of design analysis.

Design Analysis Flood Levels

Maximum still reservoir	723.5 ft
Runup on vertical external, unprotected walls	729.5 ft
Surge level within flooded structures	724.0 ft

See ~~FSAR~~ References ~~2.4A.10-1~~[27] and ~~2.4A.10-2~~[28].

In addition to level considerations, plant flood preparations will cope with the "fastest rising" flood which is the calculated flood that can exceed plant grade with the shortest prediction notice. Reservoir levels for large floods in the Tennessee Valley can be predicted well in advance.

A minimum of 27 hours, divided into two stages, is provided for safe plant shutdown by use of this prediction capability. Stage I, a minimum of 10 hours long, will commence upon a prediction that flood-producing conditions might develop. Stage II, a minimum of 17 hours long, will commence on a confirmed estimate that conditions will provide a flood above plant grade. This two-stage scheme is designed to prevent excessive economic loss in case a potential flood does not fully develop. Refer to Section 2.4.14.4.

~~2.4A.1.2~~2.4.14.1.2 Combinations of Events

Because floods above plant grade, earthquakes, tornadoes, or design basis accidents, including a loss-of-coolant accident (LOCA), are individually very unlikely, a combination of a flood plus any of these events or the occurrence of one of these during the flood recovery time or of the flood during the recovery time after one of these events is considered incredible.

~~Surges from seismic failure of upstream dams, however, can exceed plant grade, but to lower DBF levels, when imposed coincident with wind and certain floods. A minimum 27 hours of warning is assured so that ample time is available to prepare the plant for flooding.~~ However, as an exception, certain reduced levels of floods are considered together with seismic events. Refer to Section 2.4.14.10 and 2.4.4.

~~2.4A.1.3~~2.4.14.1.3 Post Flood Period

Because of the improbability of a flood above plant grade, no detailed procedures will be established for return of the plant to normal operation unless and until a flood actually occurs. If flood mode operation (~~subsection 2.4A.2~~Section 2.4.14.2) should ever become necessary, it will be possible to maintain this mode of operation for a sufficient period of time (100 days) so that appropriate recovery steps can be formulated and taken. The actual flood waters are expected to recede below plant grade within 1 to 6 days.

~~2.4A.1.4~~2.4.14.1.4 Localized Floods

Localized plant site flooding due to the probable maximum storm (~~subsection 2.4.3~~Section 2.4.2.3) will not enter vital structures or endanger the plant. Plant shutdown will be forced by water ponding on the switchyard and around buildings, but this shutdown will not differ from a loss of offsite power situation as described in Chapter 15. The other steps described in this ~~appendix~~subsection are not applicable to this case. Refer to Section 2.4.2.3.

~~2.4A.2~~2.4.14.2 Plant Operation During Floods Above Grade

"Flood mode" operation is defined as the set of conditions described below by means of which the plant will be safely maintained during the time when flood waters exceed plant grade (elevation 705.0 ft) and during the subsequent period until recovery (~~subsection 2.4A.7~~ Section 2.4.14.7) is accomplished.

~~2.4A.2.1~~2.4.14.2.1 Flooding of Structures

~~Only the~~The Reactor Building, the Diesel Generator Building (DGB), and the Essential Raw Cooling Water Intake Station will be maintained dry during the flood mode. Walls and penetrations are designed to withstand all static and dynamic forces imposed by the DBF.

The lowest floor of the DGB is at elevation 722.0 ft with its doors on the uphill side facing away from the main body of flood water. ~~This elevation is lower than the previous DBF elevation of 722.6. The 1998 reanalysis determined the still water~~ With the PMF elevation ~~to be 719.6~~of 722.0 ft, ~~with~~ wind wave runup at the DGB ~~to~~is elevation ~~721.8~~723.2 ft. Therefore, flood levels ~~do not~~ exceed floor elevation of 722.0 ft. The entrances into safety-related areas and all mechanical and electrical penetrations into safety-related areas are sealed either prior to or during flood mode to prevent major leakage into the building for water up to the PMF, including wave runup. ~~Due to the 1998 reanalysis this only applies to below grade features.~~ Redundant sump pumps are provided within the building to remove minor leakage.

The Essential Raw Cooling Water (ERCW) intake station is designed to remain fully functional for floods up to the PMF, including wind-wave runup. The deck elevation (elevation 720.0 ft) is below the PMF plus wind wave runup, but it is protected from flooding by the outside walls. The traveling screen wells extend above the deck elevation up to the design basis surge level. The wall penetration for water drainage from the deck in nonflood conditions is below the DBF elevation, but it is designed for sealing in event of a flood. All other exterior penetrations of the station below the PMF are permanently sealed. Redundant sump pumps are provided on the deck and in the interior rooms to remove rainfall on the deck and water seepage.

All other structures, including the service, turbine, auxiliary, and control buildings, will be allowed to flood as the water exceeds their grade level entrances. All equipment, including power cables, that is located in these structures and required for operation in the flood mode is either above the DBF or designed for submerged operation.

~~2.4A.2.2~~2.4.14.2.2 Fuel Cooling

Spent Fuel Pit

Fuel in the spent fuel pit will be cooled by the normal Spent Fuel Pit Cooling (SFPC) System. The pumps are located on a platform at elevation 721.0 ft which is ~~above~~below the surge level of ~~720.1~~elevation 722.5 ft. However, the pumps are located in an enclosure that provides flooding protection up to elevation 724.5 ft. During the flood mode of operation, heat will be removed from the heat exchangers by ERCW instead of component cooling water.

As a backup to spent fuel cooling, water from the Fire Protection (FP) System can be dumped into the spent fuel pool, and steam removed by the area ventilation system.

Reactors

Residual core heat will be removed from the fuel in the reactors by natural circulation in the Reactor Coolant (RC) system. Heat removal from the steam generators will be accomplished by adding river water from the FP System (subsection 9.5.1) and relieving steam to the atmosphere through the power relief valves. Primary system pressure will be maintained at less than 500 lb/in^{2}g by operation of the pressurizer relief valves and heaters. This low pressure will lessen leakage from the system. Secondary side pressure will be maintained at or below 90 psig by operation of the steam line relief valves.

An analysis has been performed to ensure that the limiting atmospheric relief capacity would be sufficient to remove steam generated by decay heat. At times beyond approximately 10 hours following shutdown of the plant two relief valves have sufficient capacity to remove the steam generated by decay heat. Since a minimum of 27 hours flood warning is available it is concluded that the plant could be safely shutdown and decay heat removed by operation of only two relief valves. Reference ~~FSAR 2.4A.10-1~~[27].

The main steam power operated relief valves will be adjusted to maintain the steam pressure at or below 90 psig. If this control system malfunctions, then the controls in the main control room can be utilized to operate the valves in an open-closed manner. Also, a manual loading station and the relief valve handwheel provide additional backup control for each relief valve. The secondary side steam pressure can be maintained for an indefinite time by the means outlined above.

The cooling water flow paths conform to the single failure criteria as defined in FSAR Section 3.1.1. In particular, all active components of the secondary side feedwater supply and ERCW supply are redundant and can therefore tolerate a single failure in the short or long term. A passive failure, consistent with the 50 gpm loss rate specified in FSAR Section 3.1.1, can be tolerated for an indefinite period without interrupting the required performance in either supply.

If one or both reactors are open to the containment atmosphere as during the refueling operations, then the decay heat of any fuel in the open unit(s) and spent fuel pit will be removed in the following manner. The refueling cavity will be filled with borated water (approximately 2000 ppm boron concentration) from the refueling water storage tank. The SFPC System pump will take suction from the spent fuel pit and will discharge to the SFPC System heat exchangers. The SFPC System heat exchanger output flow will be directed by a piping connection to the Residual Heat Removal (RHR) System heat exchanger bypass line. The tie-in locations in the SFPC System and the RHR System are shown in Figures 9.1.3-1 and 5.5.7-1, respectively. This connection will be made using prefabricated, in- position piping which is normally disconnected. During flood mode preparations, the piping will be connected using prefabricated spool pieces.

Prior to flooding, valve number 78-513 (refer to Figure 9.1.3-1) and valves FCV 74-33, and 74-35 (refer to Figure 5.5.7-1) will be closed; valves HCV 74-36, 74-37, FCV 74-16, 74-28, 63-93, and 63-94 (refer to Figure 5.5.7-1 and 6.3.1-1) will be opened or verified open. This arrangement will permit flow through the RHR heat exchangers and the four normal cold leg injection paths to the reactor vessel. The water will then flow downward through the annulus, upward through the core (thus cooling the fuel), then exit the vessel directly into the refueling cavity. This results in a water level differential between the spent fuel pit and the refueling cavity with sufficient water head to assure the required return flow through the 20-inch diameter fuel transfer tube thereby completing the path to the spent fuel pit.

Except for a portion of the RHR System piping, the only RHR System components utilized below flood elevation are the RHR System heat exchangers. Inundation of these passive components will not degrade their performance for flood mode operation. After alignment, all valves in this cooling circuit located below the maximum flood elevation will be disconnected from their power source to assure that they remain in a safe position.

The modified cooling circuit for open reactor cooling will be assured of two operable SFPC System

pumps (a third pump is available as a backup) as well as two SFPC System heat exchangers. Also, the large RHR System heat exchangers are supplied with essential raw cooling water during the open reactor mode of fuel cooling; these heat exchangers provide an additional heat sink not available for normal spent fuel cooling.

Fuel coolant temperature calculations, assuming conservative heat loads and the most limiting, single active failure in the SFPC System, indicate that the coolant temperatures are acceptable.

The temperatures can be maintained at a value appreciably less than the fuel pit temperature calculated for the nonflood spent fuel cooling case when assuming the loss of one equipment train.

As further assurance, the open reactor cooling circuit was aligned and tested, during pre-operational testing, to confirm flow adequacy. Normal operation of the RHR System and SFPC System heat exchangers will confirm the heat removal capabilities of the heat exchangers.

High spent fuel pit temperature will cause an annunciation in the MCR, thus indicating equipment malfunction. Additionally, that portion of the cooling system above flood water will be frequently inspected to confirm continued proper operation.

For either mode of reactor cooling, leakage from the Reactor Coolant System will be collected, to the extent possible, in the reactor coolant drain tank; nonrecoverable leakage will be made up from supplies of clean water stored in the four cold leg accumulators, the pressurizer relief tank, the cask decontamination tank, and the demineralized water tank. If these sources prove insufficient, the FP System can be connected to the Auxiliary Charging System (subsection 9.3.5) as a backup. Whatever the source, makeup water will be filtered, demineralized, tested, and borated, as necessary, to the normal refueling concentration, and pumped by the Auxiliary Charging System into the reactor (see Figures ~~2.4A-2~~2.4.14-1 and ~~2.4A-3~~2.4.14-2).

~~Power~~

~~Electric power will be supplied by the onsite diesel generators starting at the beginning of Stage II or when offsite power is lost, whichever occurs first (subsection 2.4A.5.3).~~

2.4.14.2.3 Cooling of Plant Loads

Plant cooling requirements, with the exception of the FP System which must supply feedwater to the steam generators, will be met by the ERCW System (refer to ~~subsection~~Section 9.2.2).

2.4.14.2.4 Power

Electric power will be supplied by the onsite diesel generators starting at the beginning of Stage II or when offsite power is lost, whichever occurs first (Section 2.4.14.5.3).

2.4.14.2.5 Plant Water Supply

The plant water supply is thoroughly discussed in ~~subsection~~Section 9.2.2. The following is a summary description of the water supply provided for use during flooded plant conditions. The ERCW station is designed to remain fully functional for all floods up to and including the DBF. The CCW intake forebay will provide a water supply for the fire/flood mode pumps. If the flood approaches DBF proportions, there is a remote possibility that Chickamauga Dam will fail. Such an event would leave the Sequoyah Plant CCW intake forebay isolated from the river as flood water recedes below EL 665. Should this event occur, the CCW forebay has the capacity of retained water to supply two steam generators in each unit and provide spent fuel pit with evaporation makeup flow until CCW forebay inventory makeup is established. The ERCW station is designed to be operable for all plant conditions and includes provisions for makeup to the forebay. Reference ~~FSAR 2.4A.10-1~~ [27].

~~2.4A.3~~2.4.14.3 Warning ~~Plan~~Scheme

See Section 2.4.14.8 (Warning Plan).

~~Plant grade elevation 705 can be exceeded by both rainfall floods and seismic-caused dam failure floods. A warning plan is needed to assure plant safety from these floods.~~

~~2.4A.3.1 Rainfall Floods~~

~~Protection of the Sequoyah Plant from the low probability rainfall floods that might exceed plant grade depends on a flood warning issued by TVA's River Operations as described in Section 2.4A.8. With TVA's extensive climate monitoring and flood predicting systems and flood control facilities, floods in the Sequoyah area can be reliably predicted well in advance. The Sequoyah Nuclear Plant flood warning plan will provide a minimum preparation time of 27 hours including a 3 hour margin for operation in the flood mode. Four additional, preceding hours will provide time to gather data and produce the warning. The warning plan will be divided into two stages--the first a minimum of 10 hours long and the second of 17 hours--so that unnecessary economic penalty can be avoided while adequate time is ensured for preparing for operation in the flood mode.~~

~~The first stage, Stage I, of shutdown will begin when there is sufficient rainfall on the ground in the upstream watershed to yield a projected plant site water level of 697 in the winter months (October 1 through April 15) and 703 in the summer (April 16 through September 30). This assures that the additional time required is available when shutdown is initiated. The water level of 703 (two feet below plant grade) will allow margin so that waves due to high winds cannot disrupt the flood mode preparation. Stage I will allow preparation steps causing some damage to be sustained but will withhold major economic damage until the Stage II warning assures a forthcoming flood above grade.~~

~~The plant preparation status will be held at Stage I until either Stage II begins or TVA's River Operations determines that flood waters will not exceed elevation 703 at the plant. The Stage II warning will be issued only when enough rain has fallen to predict that elevation 703 is likely to be exceeded.~~

~~2.4A.3.2 Seismic Dam Failure Floods~~

~~Protection of the Sequoyah plant from flood waves generated by seismically caused dam failures which exceed plant grade depends on TVA's River Operation organization to identify when a critical combination of dam failures and floods exist. There are nine upstream dams whose failure, in combination coincident with certain storm conditions, would cause a flood to exceed plant grade. These dams are Norris, Cherokee, Douglas, Fort Loudoun, Fontana, Hiwassee, Apalachia, Blue Ridge, and Tellico.~~

~~2.4A.4~~2.4.14.4 Preparation for Flood Mode

An abnormal operating instruction is available to support operation of the plant.

At the time the initial flood warning is issued, the plant may be operating in any normal mode. This means that either or both units may be at power or either unit may be in any stage of refueling.

~~2.4A.4.1~~2.4.14.4.1 Reactors Initially Operating at Power

If both reactors are operating at power, Stage I and then, if necessary, Stage II procedures will be initiated. Stage I procedures will consist of a controlled reactor shutdown and other easily ~~revokable~~revocable steps such as moving supplies necessary to the flood protection plan above the DBF level and making temporary connections and load adjustments on the onsite power supply. Stage II procedures will be the less easily ~~revokable~~revocable and more damaging steps necessary to have the plant in the flood mode when the flood exceeds plant grade. The fire/flood mode pumps may supply auxiliary feedwater for reactor cooling ~~(Reference 3)~~[29]. Other essential plant cooling loads will be transferred from the component cooling water to the ERCW System (subsection 9.2.2). The Radioactive Waste (Chapter 11) System will be secured by filling tanks below DBF level with enough water to prevent flotation; one exception is the waste gas decay tanks, which are sealed and anchored

against flotation. The CVCS hold up tank will also be filled and sealed to prevent flotation. Some power and communication lines running beneath the DBF and not designed for submerged operation will require disconnection. Batteries beneath the DBF will be disconnected.

~~2.4A.4.2~~2.4.14.4.2 Reactor Initially Refueling

If time permits, fuel ~~will be~~is removed from the unit(s) undergoing refueling and placed in the spent fuel pit; otherwise fuel cooling will be accomplished as described in ~~subsection 2.4A.2.2~~Section 2.4.14.2.2. If the refueling canal is not already flooded, the mode of cooling described in ~~subsection 2.4A.2.2~~Section 2.4.14.2.2 requires that the canal be flooded with borated water from the refueling water storage tank. If the flood warning occurs after the reactor vessel head has been removed or at a time when it could be removed before the flood exceeds plant grade, the flood mode reactor cooling water will flow directly from the vessel into the refueling cavity. If the warning time available does not permit this, then the upper head injection piping will be disconnected above the vessel head to allow the discharge of water through the four upper head injection standpipes. Additionally, it is required that the prefabricated piping be installed to connect the RHR and SFPC Systems, and that ERCW be directed to the secondary side of the RHR System and SFPC System heat exchangers.

~~2.4A.4.3~~2.4.14.4.3 Plant Preparation Time

All steps needed to prepare the plant for flood mode operation can be accomplished within 24 hours of receipt of the initial warning that a flood above plant grade is possible. An additional 3 hours are available for contingency margin before wave runup from the rising flood might enter the buildings. ~~Site grading and building design prevent any flooding before the end of the 27 hour preflood period.~~

~~2.4A.5~~2.4.14.5 Equipment

Both normal plant components and specialized flood-oriented supplements will be utilized in coping with floods. All such equipment required in the flood mode is either located above the DBF or is within a nonflooded structure or is designed for submerged operation. Systems and components needed only in the preflood period are protected only during that period.

~~2.4A.5.1~~2.4.14.5.1 Equipment Qualification

To ensure capable performance in this highly unlikely but rigorous, limiting design case, only high quality components will be utilized. Active components are redundant or their functions diversely supplied. Since no rapidly changing events are associated with the flood, repairability offers reinforcement for both active and passive components during the long period of flood mode operation. Equipment potentially requiring maintenance will be accessible throughout its use, including components in the Diesel Generator Building.

~~2.4A.5.2~~2.4.14.5.2 Temporary Modification and Setup

Normal plant components used in flood mode operation and in preparation for flood mode operation may require modification from their normal plant operating configuration. Such modification, since it is for a limiting design condition and since extensive economic damage is acceptable, will be permitted to damage existing facilities for their normal plant functions. However, most alterations will be only temporary and nondestructive in nature. For example, the switchover of plant cooling loads from the component cooling water to the ERCW System will be done through valves and a prefabricated spool piece, causing little system disturbance or damage.

Equipment especially provided for the flood design case includes both permanently installed components and more portable apparatus that will be emplaced and connected into other systems during the preflood period.

Detailed procedures to be used under flood mode operation have been developed and are incorporated in the plant's Abnormal Operating Instructions.

~~2.4A.5.3~~2.4.14.5.3 Electric Power

Because there is a possibility that high winds may destroy powerlines and disconnect the plant from offsite power at any time during the preflood transition period, only onsite power will be used once Stage II of the preparation period begins. While most equipment requiring alternating current electric power is a part of the permanent emergency onsite power system, other components will be temporarily connected, when the time comes, by prefabricated jumper cables.

All loads that are normally supplied by onsite power but are not required for the flood will be switched out of the system during the preflood period. Those loads used during the preflood period but not during flood mode operation will be disconnected when they are no longer needed. During the preparation period, all power cables running beneath the DBF level, except those especially designed for submerged operation, will be disconnected from the onsite power system. Similarly, direct current electric power will be disconnected from unused loads and potentially flooded lines. Charging will be maintained for each battery by the onsite alternating current power system as long as it is required. Batteries that are beneath the DBF will be disconnected during the preflood period when they are no longer needed.

~~2.4A.5.4~~2.4.14.5.4 Instrument Control, Communication and Ventilation Systems

All instrument, control, and communication lines that will be required for operation in the flood mode are either above the DBF or within a nonflooded structure or are designed for submerged operation. Unneeded cables that run below the DBF will be disconnected to prevent short circuits.

Redundant means of communications are provided between the central control area (the main and auxiliary control rooms) and all other vital areas that might require operator attention, such as the Diesel Generator Building.

Instrumentation is provided to monitor all vital plant parameters such as the reactor coolant temperature and pressure and steam generator pressure and level. Control of the pressurizer heaters and relief valves and steam generator feedwater flow and atmospheric relief valves will ensure continued natural circulation core cooling during the flood mode. All other important plant functions will be either monitored and controlled from the main control area or, in some cases where time margins permit, from other points in the plant that are in close communication with the main control area. Ventilation, when necessary, and limited heating or air-conditioning will be maintained for all points throughout the plant where operators might be required to go or where required by equipment heat loads.

~~2.4A.6~~2.4.14.6 Supplies

All equipment and most supplies required for the flood are on hand in the plant at all times. Some supplies will require replenishment before the end of the period in which the plant is in the flood mode. In such cases supplies on hand will be sufficient to last through the short time (~~subsection 2.4A.1.3~~Section 2.4.14.1.3) that flood waters will be above plant grade and until replenishment can be supplied. For instance, there is sufficient diesel generator fuel available at the plant to last for 3 or 4 weeks; this will allow sufficient margin for the flood to recede and for transportation routes to be reestablished.

~~2.4A.7~~2.4.14.7 Plant Recovery

The plant is designed to continue safely in the flood mode for 100 days even though the water is not expected to remain above plant grade for more than 1 to 6 days. After recession of the flood, damage will be assessed and detailed recovery plans developed. Arrangements will then be made for reestablishment of offsite power and removal of spent fuel.

The 100-day period provides more than adequate time for the development of procedures for any maintenance, inspection, or installation of replacements for the recovery of the plant or for a continuation of flood mode operations in excess of 100 days. A decision based on economics will be

made on whether or not to regain the plant for power production. In either case, detailed plans will be formulated after the flood, when damage can be accurately assessed.

~~2.4A.8~~2.4.14.8 ~~Basis For Flood Protection Plan In Rainfall Floods~~Warning Plan

Plant grade elevation 705.0 ft can be exceeded by both rainfall floods and seismic-caused dam failure floods. A warning plan is needed to assure plant safety from these floods.

The warning plan is divided into two stages: Stage I, a minimum of 10 hours long and Stage II, a minimum of 17 hours so that unnecessary economic consequences can be avoided, while adequate time is allowed for preparing for operation in the flood mode. Stage I allows preparation steps causing minimal economic consequences to be sustained but will postpone major economic damage until the Stage II warning forecasts a likely forthcoming flood above elevation 703.0 ft.

2.4.14.8.1 Rainfall Floods

Protection of the Sequoyah Plant from the low probability rainfall floods that might exceed plant grade depends on a flood warning issued by TVA's River Operations (RO). With TVA's extensive climate monitoring and flood forecasting systems and flood control facilities, floods in the Sequoyah area can be reliably predicted well in advance. The SQN flood warning plan will provide a minimum preparation time of 27 hours including a 3 hour margin to prepare for operation in the flood mode. Four additional, preceding hours will provide time to gather data and produce the warning.

The first stage, Stage I, of shutdown will begin when there is sufficient rainfall on the ground in the upstream watershed to yield a forecasted plant site water level of 694.5 ft in the winter months and 699.0 ft in the summer. This assures that the additional time required is available when shutdown is initiated. The water level of 703.0 ft (two feet below plant grade) will allow margin so that waves due to high winds cannot disrupt the flood mode preparation. Stage I will allow preparation steps causing some damage to be sustained but will withhold major economic damage until the Stage II warning assures a forthcoming flood above grade.

The plant preparation status will be held at Stage I until either Stage II begins or TVA's RO determines that flood waters will not exceed elevation 703.0 ft at the plant. The Stage II warning will be issued only when enough rain has fallen to predict that elevation 703.0 ft (winter or summer) is likely to be exceeded.

2.4.14.8.2 Seismically-Induced Dam Failure Floods

Four postulated combinations of seismically induced dam failures and coincident storm conditions were shown to result in floods which could exceed elevation 703.0 ft at the plant. SQN's notification of these floods utilizes TVA's RO forecast system to identify when a critical combination exists. Stage I shutdown is initiated upon notification that a critical dam failure combination has occurred or loss of communication prevents determining a critical case has not occurred. Stage I shutdown continues until it has been determined positively that critical combinations do not exist. If communications do not document this certainty, shutdown procedures continue into Stage II activity. Stage II shutdown continues to completion or until lack of critical combinations is verified.

~~Summary~~2.4.14.9 Basis For Flood Protection Plan In Rainfall Floods

2.4.14.9.1 Overview

Large Tennessee River floods can exceed plant grade elevation 705.0 ft at ~~Sequoyah Nuclear Plant~~SQN. Plant safety in such an event requires shutdown procedures which may take 24 hours to implement. TVA flood forecast procedures will provide at least 27 hours of warning before river levels reach elevation 703.0 ft. Use of elevation 703.0 ft, 2 f~~ee~~t below plant grade, provides enough freeboard to prevent waves from 45-mile-per-hour, overwater winds from endangering plant safety during the final hours of shutdown activity. For conservatism the fetches calculated for the PMF (Figures 2.4.3-~~15~~24 and 2.4.3-~~16~~25) were used to calculate maximum wind wave additive to the

reservoir surface at elevation 703.0 ft feet ~~msl~~. The maximum wind additive to the reservoir surface would be ~~2.8~~4.2 f~~ee~~t and would not endanger plant safety during the final hours of shutdown. This is due to the long shallow approach and the waves breaking at the perimeter road (elevation 705.0 ft ~~feet msl~~). After the waves break there is not sufficient depth or distance between the perimeter road and the safety-related facilities for new waves to be generated. Forecast will be based upon rainfall already reported to be on the ground.

Different target river level criteria are needed for winter use and for summer use to allow for seasonally varied reservoir levels and rainfall potential.

To be certain of 27 hours for preflood preparation, warnings of floods with the prospect of reaching elevation 703.0 ft must be issued early; consequently, some of the warnings may later prove to have been unnecessary. For this reason preflood preparations are divided into two stages. Stage I steps, requiring 10 hours, would be easily ~~revokable~~revocable and cause minimum damage. The estimated probability is ~~less than 0.0026~~small that a Stage I warning will be issued during the ~~40-year~~ life of the plant.

Additional rain and stream-flow information obtained during Stage I activity will determine if the more damaging steps of Stage II need to be taken with the assurance that at least 17 hours will be available before elevation 703.0 ft is reached. The ~~estimated~~ probability of a Stage II warning during the life of the plant is ~~less than 0.0010 that shutdown will need to continue into Stage II during plant life~~very small.

Flood forecasting and warnings, to assure adequate warning time for safe plant shutdown during floods, will be conducted by ~~River Operations of River System Operations~~TVA's RO.

2.4.14.9.2 TVA Forecast System ~~(HISTORICAL INFORMATION)~~

TVA has in constant use an extensive, effective system to forecast flow and elevation as needed in the Tennessee River Basin. This permits efficient operation of the reservoir system and provides warning of when water levels will exceed critical elevations at selected, sensitive locations which includes SQN.

Elements of the present (~~2001~~2012) forecast system above ~~Sequoyah Nuclear Plant~~SQN include the following:

1. ~~One hundred sixty (160)~~More than 100 rain gages measure rainfall, with an average density of ~~165~~about 200 square miles per rain gage. ~~Of these gages 112 are owned by TVA, 35 are owned by the National Weather Service (NWS), 7 are owned by the United States Geological Service (USGS), 2 are owned by the United States Corps of Engineers (USACE), and 4 are owned by Alcoa. Most of these gages are tipping buckets collector type and the transmission of the data is either by satellite or telephone. At some of the gages located at hydroplants, the data is manually read.~~All are Geostationary Operational Environmental Satellites (GOES) Data Collection Platform (DCP) satellite telemetered gages.

   ~~Information normally is received daily from the gages at 6 a.m. and at least every 6 hours during flood periods. Close interval rainfall reports can be obtained from a majority of the gages if needed.~~ All of the rainfall gages transmit hourly rainfall data.

2. Streamflow data are received for ~~35~~23 gages ~~from 16 TVA gages amd 19 USGS gages. These gages trasmit their data either by satellite or telephone or both. Discharge data are received from 26 hydroplants. Of these plants, 25 also transmit headwater elevation data, and 13 transmit tailwater elevation data. Therefore, steamflow data are available from 61 locations. Streamflow data are received daily at 8 a.m. and at least every 2 hours if needed during flood operations.~~in the system. All are GOES Data Collection Platform satellite telemetered gages. The satellite gages transmit 15-minute stage data every hour during normal operations.

3. Real-time headwater elevation, tailwater elevation, and discharge data are received from 21 TVA hydro projects (Watts Bar, Melton Hill, Fort Loudoun, Tellico, Norris, Douglas, Cherokee, Fort Patrick Henry, Boone, Watauga, Wilbur, South Holston, Chickamauga, Ocoee No. 1, Ocoee No. 2, Ocoee No. 3, Blue Ridge, Apalachia, Hiwassee, Chatuge and Nottely) and hourly data are received from non-TVA hydro plants (Chilhowee, Cheoah, Calderwood and Santeetlah).

~~3~~4. Weather forecasts including quantitative precipitation forecasts are received ~~four times~~at least twice daily and at other times when changes are expected.

~~4~~5. Computer programs which translate rainfall into streamflow based on current runoff conditions and which permit a forecast of flows and elevations based upon both observed and predicted rainfall. ~~Two separate~~A network of UNIX servers and personal computers are utilized and are designed to provide backup for each other. One computer is used primarily for data collection, with the other used for executing forecasting programs for reservoir operations. The time interval between receiving input data and producing a forecast is less than 4 hours. Forecasts normally cover at least a ~~8~~three-day period.

As effective as the forecast system already is, it is constantly being improved as new technology provides better methods to interrogate the watershed during floods and as the watershed mathematical model and computer system are improved. Also, in the future, improved quantitative precipitation forecasts may provide a more reliable early alert of impending major storm conditions and thus provide greater flood warning time.

~~The TVA forecast center is manned 24 hours a day. Normal operation produces two forecasts daily, one by 12 noon based on data collected at 6 a.m. Central time, and the second by 4 a.m. based on data collected at midnight Central Time. When serious flood situations demand, forecasts are produced every 4 hours.~~

2.4.14.9.3 Basic Analysis

~~To develop a~~The forecast procedure to assure safe shutdown of ~~Sequoyah Nuclear Plant~~SQN for flooding~~, 17~~ is based upon an analysis of nine hypothetical PMP storms~~, including their antecedent storms, were analyzed. They~~ up to PMP magnitude. The storms enveloped potentially critical areal and seasonal variations and time distributions of rainfall. To be certain that fastest rising flood conditions were included, the effects of varied time distribution of rainfall were tested by alternatively placing the maximum daily PMP ~~on~~in the ~~first, the~~ middle~~,~~ and the last day of the ~~3~~three-day main storm. ~~In each day the maximum 6-hour depth was placed during the second interval except when the maximum daily rain was placed on the last day. Then the maximum 6-hour amount was placed in the last 6 hours.~~Earlier analysis of 17 hypothetical storms demonstrated that the shortest warning times resulted from storms in which the heavy rainfall occurred on the last day and that warning times were significantly longer when heavy rainfall occurred on the first day. Therefore, heavy rainfall on the first day was not reevaluated. The warning system is based on those storm situations which resulted in the shortest time interval between watershed rainfall and elevation 703.0 ft at SQN, thus assuring that this elevation could be predicted at least 27 hours in advance.

The procedures used to compute flood flows and elevations are described in ~~subsections 2.4.3.1, 2.4.3.2, and 2.4.3.3~~ Section 2.4.3. ~~Some flood events were analyzed using earlier versions of the watershed model described in subsection 2.4.3.3. Those events which established important elements of the warning system or those where the present model might produce significant differences in warning times have been reevaluated. Events reevaluated have been noted either in tables or figures where appropriate.~~

~~The warning system is based on those storm situations which resulted in the shortest time interval between watershed rainfall and elevation 703, thus assuring that this elevation could be predicted at least 27 hours in advance.~~

2.4.14.9.4 Hydrologic Basis for Warning System

A minimum of 27 hours has been allowed for preparation of the plant for operation in the flood mode, three hours more than the 24 hours needed. An additional 4 hours for communication and forecasting computations are provided to allow TVA's RO to translate rain on the ground to river elevations at the plant. Hence, the warning plan must provide 31 hours from arrival of rain on the ground until ~~critical~~ elevation 703.0 ft could be reached. The 27 hours allowed for shutdown at the plant are utilized for a minimum of 10 hours of Stage I preparation and an additional 17 hours for Stage II preparation that is not concurrent with the Stage I activity. This 27 hour allocation includes a 3-hour margin.

Although river elevation 703.0 ft, 2 f~~ee~~t below plant grade to allow for wind waves, is critical during final stages of plant shutdown for flooding, lower forecast target levels are used in most situations to assure that the 27 hours preflood transition interval will always be available. The target river levels differ with season.

During the ~~October 1 through April 15~~ "winter" season, Stage I shutdown procedures will be started as soon as target river elevation ~~697~~694.5 ft has been forecast. ~~Shutdown~~Stage II shutdown will be initiated and carried to completion if and when target river elevation 703.0 ft at SQN has been forecast. Corresponding target river elevations for the ~~April 16 through September 30~~ "summer" season at SQN ~~is 703~~ are elevation 699.0 ft and elevation 703.0 ft. ~~The one target river elevation in the summer season permits waiting to initiate shutdown procedures until enough rain is on the ground to forecast reaching critical elevation 703; shutdown would then be initiated and carried to completion.~~

Inasmuch as the hydrologic procedures and target river elevations have been designed to provide adequate shutdown time in the fastest rising flood, longer times will be available in other floods. In such cases there ~~will~~may be a waiting period after the Stage I, 10-hour shutdown activity during which activities shall be in abeyance until ~~it is predicted from recorded rainfall that Stage II shutdown should be implemented or it is determined from weather conditions that plant operation can be resumed~~ weather conditions determine if plant operation can be resumed, or if Stage II shutdown should be implemented.

Resumption of plant operation following Stage I shutdown activities will be allowable only after flood levels and weather conditions, as determined by TVA's RO, have returned to a condition in which 27 hours of warning will again be available.

~~River Scheduling of River Operations prepares at least an 8-day water level forecast seven days per week for Tennessee River locations. During prospective flooding conditions forecasts can be prepared 4 times a day so that warnings for Sequoyah will assure that 27 hours always will be available to shut down the plant and prepare it for flooding.~~

2.4.14.9.5 Hydrologic Basis for Target Stages

~~Figure 2.4A-4, in four parts, shows how target forecast flood elevations at the Sequoyah plant have been determined to assure adequate warning times. The floods shown are the fastest rising floods at the site which are produced by the 21,400-square-mile PMP with downstream centering described in subsection 2.4.3.1. The storms are the main PMP amounts and have been preceded 3 days earlier by a 3-day storm having 40 percent of the main storm rainfall. This has caused soil moisture to be high and reservoirs to be well above seasonal levels when the main storm begins.~~Figure 2.4.14-3 (Sheet 1) and Figure 2.4.14-3 (Sheet 2) for winter and summer respectively, show target forecast flood warning time and elevation at SQN which assure adequate warning times. The fastest rising probable maximum flood for the winter at the site is shown in Figure 2.4.14-3 (Sheet 1A). Figure 2.4.14-3 (Sheets 1B and 1C) show the adopted rainfall distribution for the 21,400 square mile storm and the 7,980 square mile storm, respectively. An intermediate flood with average basin rainfall of 10 inches (rainfall heavy at the end) is shown in Figure 2.4.14-3 (Sheet 1D). Figure 2.4.14-3 (Sheet 2A) shows the 7,980 square mile fastest rising probable maximum flood for the summer with heavy rainfall at the end. The 7,980 square mile adopted rainfall distribution is shown in Figure 2.4.14-3 (Sheet 2B). An intermediate flood with average basin rainfall of 10 inches heavy at the end is shown in Figure 2.4.14-3 (Sheet 2C). All of these storms have been preceded three days earlier by a three-day storm having 40% of PMP storm rainfall.

~~Figure 2.4A.-4 (A, B, and C) shows the winter PMP which could produce the fastest rising flood which would cross plant grade and variations caused by changed time distribution.~~ The fastest rising flood occurs during a PMP when the ~~6~~six-hour increments increase throughout the storm with the maximum ~~6 hours~~six-hour increment increase occurring in the last period. Figure ~~2.4A-4~~ 2.4.14-3 (~~B~~Sheet 1A) shows the essential elements of this storm which provides the basis for the warning ~~scheme~~plan. In this flood ~~9.2~~7.35 inches of rain would have fallen 31 hours (27 + 4) prior to the flood crossing elevation 703.0 ft and would produce elevation ~~697~~694.5 ft at the plant. Hence, any time rain on the ground results in a predicted plant stage of ~~697~~694.5 ft a Stage I shutdown warning will be issued. Examination of Figure ~~2.4A.-4 (A and C)~~ 2.4.14-3 (Sheets 1B and 1C) shows that following this procedure in these ~~noncritical~~ floods would result in ~~a lapsed time of~~ longer times to reach elevation 703.0 ft after Stage I warning was issued. These times would be ~~42~~33.6 and ~~44~~43.6 hours ~~between when 9.2 inches had fallen and the flood would cross critical elevation 703~~ (includes 4 hours for forecasting and communication) for Figure 2.4.14-3 (Sheet 1B) and (Sheet 1C), respectively. This compares to the 31 hours for the fastest rising flood as shown in Figure 2.4.14-3 (Sheet 1A). Stage I warning would be issued for the storm shown in Figure 2.4.14-3 (Sheet 1D) and 63 hours would pass before elevation 703.0 ft would be reached.

~~An~~A Stage II warning would be issued if an additional ~~2.2~~2.44 inches of rain ~~must fall~~fell promptly for a total of ~~11.4~~9.79 inches of rain ~~to cause the flood to cross critical elevation 703~~. In the fastest rising flood, Figure ~~2.4A.-4 (B)~~2.4.14-3 (Sheet 1A), this rain would have fallen in the next ~~5~~6.9 hours. Thus, 6.9 hours after issuance of a Stage I warning, enough rain would have fallen to require a Stage II warning. A Stage II warning would be issued within the next 4 hours and the flood wood exceed elevation 703.0 ft in 24.1 hours. ~~Thus, the Stage II warning would be issued 5 hours after issuance of a Stage I warning and 22 hours before the flood would cross critical flood elevation 703.~~ In the slower rising floods, Figure ~~2.4A.-4 (A and C)~~ 2.4.14-3 (Sheets 1B and 1C), the time between issuance of a Stage I warning and when the ~~11.4~~9.79 inches of rain required to put the flood to elevation 703.0 ft would have occurred is ~~6~~3.6 and ~~10~~3.3 hours, respectively. This would result in issuance of a Stage II warning ~~not less than 4 hours later or 32 and 30~~30 or 40.3 hours, respectively, before the flood would reach elevation 703.0 ft.

The summer flood shown by Figure ~~2.4A.-4 (D)~~ 2.4.14-3 (Sheet 2A), with the maximum ~~1~~one-day rain on the last day provides controlling conditions when reservoirs are at summer levels. At a time 31 hours (27 + 4) before the flood reaches elevation 703.0 ft, ~~11~~8.18 inches of rain would have fallen. This ~~11~~8.18 inches of rain, under these runoff conditions, would produce ~~critical~~ elevation ~~703~~699.1 ft, ~~so this level becomes both~~ the Stage I ~~and Stage II~~ target. An additional 1.3 inches of rain must fall promptly for a total of 9.48 inches of rain to cause the flood to exceed elevation 703.0 ft.

The above criteria all relate to forecasts which use rain on the ground. In actual practice quantitative rain forecasts, which are already a part of daily operations, would be used to provide advance alerts that need for shutdown may be imminent. Only rain on the ground, however, is included in the procedure for firm warning use.

Because the above analyses have used fastest possible rising floods at the plant, all other floods will allow longer warning times than required for all physical plant shutdown activity.

In summary, the ~~predicted target levels~~forecast elevations which will assure adequate shutdown times are:

	Forecast Flood Elevations at Sequoyah	
Season	For Stage I Shutdown	For Stage II Shutdown
Winter ~~(October 1-April 15)~~	~~697~~694.5 ft	703.0 ft
Summer ~~(April 16-September 30)~~	~~703~~699.0 ft	703.0 ft

2.4.14.9.6 Communications Reliability ~~(HISTORICAL INFORMATION)~~

Communication between projects in the TVA power system is via (a) TVA owned microwave network, (b) Fiber-Optic System, and (c) by commercial telephone. In emergencies, additional communication links are provided by Transmission Power Supply radio network. The four networks provide a high level of dependability against emergencies. Additionally, RO have available satellite telephone communications with the TVA hydro projects upstream of Chattanooga (listed in Section 2.4.14.9.2).

~~The hydrologic network for the watershed above Sequoyah that would be available in flood emergencies if commecial telephone communications is lost include 138 rainfall gages (24 at power installations and 114 satellite and file transfer gages) and 47 streamflow gages (26 at hydroplants, 20 satellite gages, and 1 file transfer gage). River Scheduling~~ RO is linked to the TVA power system by all ~~four~~five communication networks. The data from the satellite gages are received via a data collection platform-satellite computer system located in the ~~River Scheduling's~~RO office. ~~These are so distributed over the watershed that reasonable flood forecasting can be done from this data while the balance of data is being secured from the remaining hydrologic network stations.~~

~~The preferred, complete coverage of the watershed, employ 160 rainfall and 61 streamflow locations above the Sequoyah plant. Involved in the communications link to these locations are routine radio, radio satellite, and commercial telephone system networks. In an emergency, available radio communications would be called upon to assist.~~

~~The various networks proved to be capable in the large floods of 1957, 1963, 1973, 1984, 1994, and 1998 of providing the rain and streamflow data needed for reliable forecasts.~~

~~2.4A.9~~2.4.14.10 Basis for Flood Protection Plan in Seismic-Caused Dam Failures

~~Floods resulting from combined seismic and flood events can exceed plant grade, thus requiring emergency measures. The 1998 reanalysis showed that only two combinations of seismic dam failures coincident with a flood would result in floods above plant grade: (1) failure of Fontana, Hiwassee, Apalachia, and Blue Ridge Dams in the one-half SSE concurrent with a 1/2 PMF, (2) SSE failure of Norris, Cherokee, and Douglas concurrent with a 25-year flood. As shown in Table 2.4.4-1 all other potentially critical candidates would create flood levels below plant grade elevation 705.~~ Plant grade would be exceeded by four of the five candidate seismic failure combinations evaluated, thus requiring emergency measures. Table 2.4.4-1, shows the maximum elevations at SQN for the candidate combinations. The combination producing the shortest time interval between seismic event and plant grade crossing is a OBE located so as to fail Fontana, Tellico, Hiwassee, Apalachia, and Blue Ridge Dams during the one-half PMF. The time between the seismic event and the resulting flood wave crossing plant grade elevation 705.0 ft is 40 hours. The time to elevation 703.0 ft, which allows a margin for wind wave considerations, is 32 hours. The event producing the next shortest time interval to elevation 703.0 ft involves the OBE failure of Tellico and Norris during the one-half PMF resulting in a time interval of 34 hours. These times are adequate to permit safe plant shutdown in readiness for flooding.

Dam failure during non-flood periods ~~would not present a problem at the plant. The reanalysis showed that failure in a non-flood period and at summer flood guide levels in the most critical dam failure combination (SSE failure of Norris, Cherokee and Douglas) would produce a maximum elevation of 703.6 at the plant, 1.4 feet below plant grade. All other combinations in non-flood periods would produce elevations much lower.~~ was not evaluated, but would be bounded by the four critical failure combinations.

~~The time from seismic occurrence to arrival of failure surge at the plant is adequate to permit safe plant shutdown in readiness for flooding. Table 2.4A-2 lists the time between the postulated seismic event and when the flood wave would exceed plant grade elevation 705 and elevation 703. Use of elevation 703 provides a margin for possible wind wave effects.~~

The warning plan for safe plant shutdown is based on the fact that a combination of critically centered large earthquake ~~and rain produced flood~~ conditions must coincide before the flood wave from seismically caused dam failures will ~~cross~~approach plant grade. In flood situations, an extreme earthquake must be precisely located to fail ~~three~~two or more major dams before a flood threat to the

site would exist.

~~The combination producing the shortest time interval between seismic event and plant grade crossing is a one-half SSE located so as to fail Fontana, Hiwassee, Apalachia, and Blue Ridge Dams during the one-half PMF. The time between the seismic event and the resulting flood wave crossing plant grade elevation 705 is 40 hours. The time to elevation 703, which allows a margin for wind wave considerations, is 35 hours. The event producing the next shortest time interval to elevation 703 involves the SSE failure of Norris, Cherokee, and Douglas during the 25-year flood resulting in a time interval of 63 hours.~~

The warning system utilizes TVA's flood forecast system to identify when flood conditions will be such that seismic failure of critical dams could cause a flood wave to exceed elevation 703.0 ft at the plant site. In addition to the critical combinations, failure of a single major upstream dam will lead to an early warning. A Stage I warning is declared once failure of (1) Norris, Cherokee, Douglas, and Tellico Dams or (2) Norris and Tellico Dams, or (3) Fontana, Tellico, Hiwassee, Appalachia, and Blue Ridge Dams, or (4) Cherokee, Douglas and Tellico Dams has been confirmed.

~~Two levels of warning will be provided: (1) an early warning will be issued to SQN whenever a dam failure has occurred or is imminent for any single critical dam; or it appears from rain and flood forecasts that a critical situation may develop and (2) a flood warning or alert to begin preparation for plant shutdown when a critical situation exists that will result in the flood level to exceeding plant grade. A Stage I flood warning is declared once failure of critical dams has been confirmed and flood conditions are such that the flood surge will exceed plant grade. It shall be issued at least 27 hours before the flood level exceeds elevation 703 at the site. A Stage II flood warning will be issued at least 17 hours before the flood level exceeds elevation 703 at the site. Communication will be established and maintained during these two levels of warning to assure the 27 hour flood preparation period. Any prolonged interruption of communication or failure to confirm that a critical case has not occurred will result in the initiation of flood preparation at the plant site. The flood preparation shall continue until completion, unless communication is re-established and the site is notified that a critical case has not occurred.~~If loss of or damage to an upstream dam is suspected based on monitoring by TVA's RO, efforts will be made by TVA to determine whether dam failure has occurred. If the critical case has occurred or it cannot be determined that it has not occurred, Stage I shutdown will be initiated. Once initiated, the flood preparation procedures will be carried to completion unless it is determined that the critical case has not occurred.

Communications between ~~the plant~~SQN, dams, power system control center, and ~~River Operations at Knoxville, Tennessee~~TVA RO, are ~~provided~~accomplished by TVA-owned microwave networks, fiber-optics network, radio networks, and commercial and satellite telephone service.

2.4.14.11 Special Condition Allowance

The flood protection plan is based upon the minimum time available for the worst case. This worst case provides adequate preparation time including contingency margin for normal and anticipated plant conditions including anticipated maintenance operations. It is conceivable, however, that a plant condition might develop for which maintenance operations would make a longer warning time desirable. In such a situation the Plant Manager determines the desirable warning time. He contacts TVA's RO to determine if the desired warning time is available. If weather and reservoir conditions are such that the desired time can be provided, special warning procedures will be developed, if necessary, to ensure the time is available. This special case continues until the Plant Manager notifies TVA's RO that maintenance has been completed. If threatening storm conditions are forecast which might shorten the available time for special maintenance, the Plant Manager is notified by RO and steps taken to assure that the plant is placed in a safe shutdown mode.

~~2.4A.10 References~~

~~1. SQN-DC-V-1.1, Design of Reinforced Concrete Structures Design Criteria~~

~~2. SQN-DC-V-12.1, Flood Protection Provisions Design Criteria~~

~~3. SQN-DC-V-43.0, High Pressure Fire Protection Water Supply System~~

2.4.15 References

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2. U.S. Weather Bureau, "Probable Maximum and TVA Precipitation for Tennessee River Basins Up To 3,000 Square Miles in Area and Duration to 72 Hours," Hydrometeorological Report No. 45, 1969.

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4. PSAR, Phipps Bend Nuclear Plant, Docket Nos. 50-553, 50-554.

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15. Reference removed per Amendment 6.

16. Cristofano, E. A., "Method of Computing Erosion Rate for Failure of Earthfill Dams," Engineering and Research Center, Bureau of Reclamation, Denver 1966.

17. "The Breaching of the Oros Earth Dam in the State of Ceara, North-East Brazil," Water and Water Engineering, August 1960.

18. NRC letter to TVA dated December 8, 1989, "Chickamauga Reservoir Sediment Deposition and Erosion - Sequoyah Nuclear Plant, Units 1 and 2."

19. Programmatic Environmental Impact Statement, TVA Reservoir Operations Study, Record of Decision, May 2004.

20. Updated Predictions of Chickamauga Reservoir Recession Resulting from Postulated Failure of the South Embankment at Chickamauga Dam; TVA River System Operations and Environment, Revised June 2004 (B85 070509 001).

21. Monitoring and Moderating Sequoyah Ultimate Heat Sink, June 2004, River System Operations and Environment, River Operations, River Scheduling (B85 070509 001).

22. SQN Calculation MDQ0026970001A, "High Pressure Fire Protection Supply to the Steam Generators for Flood Mode Operation."

23. Newton, Donald W., and Vineyard, J. W., "Computer-Determined Unit Hydrographs From Floods," Journal of the Hydraulics Division, ASCE, Volume 93, No. HY5, September 1967.

24. U.S. Army Corps of Engineers, Hydrologic Engineering Center, River Analysis System, HEC-RAS computer software, version 3.1.3.

25. Federal Emergency Management Agency (FEMA), "Federal Guidelines for Dam Safety: Earthquake Analysis and Design of Dams," FEMA 65, May 2005.

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27. SQN-DC-V-1.1, Design of Reinforced Concrete Structures Design Criteria.

28. SQN-DC-V-12.1, Flood Protection Provisions Design Criteria.

29. SQN-DC-V-43.0, High Pressure Fire Protection Water Supply System.

ENCLOSURE 1

EVALUATION OF PROPOSED CHANGES

ATTACHMENT 2

Proposed SQN Units 1 and 2 UFSAR Tables

Table 2.4.1-1

Public and Industrial Surface Water Supplies Withdrawn from the 98.6 Mile Reach of the Tennessee River between Dayton Tennessee and Meade Corp. Stevenson Ala.

Plant Name	Use (MGD)	Location	Approximate Distance From Site (River Miles)	Type Supply
City of Dayton	1.780	TRM 503.8 R	19.1 (Upstream)	Municipal
Cleveland Utilities Board	5.030	TRM 499.4 L Hiwassee RM 22.9	37.6 (Upstream)	Municipal
Bowaters Southern Paper	80.000	TRM 499.4 L Hiwassee RM 22.7	37.4 (Upstream)	Industrial & Potable
Hiwassee Utilities	3.000	TRM 499.4 L Hiwassee RM 22.5	37.2 (Upstream)	Municipal
Olin Corporation	5.000	TRM 499.4 L Hiwassee RM 22.3	37.0 (Upstream)	Industrial & Potable
Soddy-Daisy Falling Water U.D.	0.927	TRM 487.2 R Soddy Cr. 4.6 Plus 2 Wells	7.1 (Upstream)	Municipal
Sequoyah Nuclear Plant	1615.680	TRM 484.7 R	0.0	Industrial
East Side Utility	5.000	TRM 473.0 L	11.7 (Downstream)	Municipal
Chickamauga Dam	#	TRM 471.0	13.7 (Downstream)	Industrial
DuPont Company	7.200	TRM 469.9 R	14.8 (Downstream)	Industrial
Tennessee-American Water	40.930	TRM 465.3 L	19.4 (Downstream)	Municipal
Rock-Tennessee Mill	0.510	TRM 463.5 R	21.2 (Downstream)	Industrial
Dixie Sand and Gravel	0.035	TRM 463.2 R	21.5 (Downstream)	Industrial
Chattanooga Missouri Portland Cement	0.100	TRM 456.1 R	28.6 (Downstream)	Industrial
Signal Mountain Cement	2.800	TRM 454.2 R	30.5 (Downstream)	Industrial
Racoon Mount. Pump Stor.	0.561	TRM 444.7 L	40.0 (Downstream)	Industrial
Signal Mountain Cement	0.200	TRM 433.3 R	51.4 (Downstream)	Industrial
Nickajack Dam	#	TRM 424.7	60.0 (Downstream)	Industrial
South Pittsburg	0.900	TRM 418.0 R	66.7 (Downstream)	Municipal
Penn Dixie Cement	0.00001	TRM 417.1 R	67.6 (Downstream)	Industrial
Bridgeport	0.600	TRM 413.6 R	71.1 (Downstream)	Municipal
Widows Creek Stream Plant	397.440	TRM 407.7 R	77.0 (Downstream)	Industrial
Mead Corporation	4.400	TRM 405.2 R	79.5 (Downstream)	Industrial

Water usage is not metered Flow Rate fluctuates as needed and is directed by power control center in Chattanooga.

Table 2.4.1-2 Facts About TVA Dams and Reservoirs
(Page 1 of 2)

Main River Projects	Dam Locations: River	Dam Locations: State	Drainage Area Above Dam (Square Miles)	Cost[b] (Millions)	Construction Began	Dam Closure	First Unit in Service (Actual or Scheduled)	Last Unit in Service (Actual or Scheduled)	Winter Net Dependable Capacity[c] (Megawatts)	Number of Generating Units	Location of Dam Above Mouth of River (Miles)	Height of Dam (Feet)	Length of Dam (Feet)	Type of Dam[d]	Lock Chamber Size: Width x Length x Maximum Lift (Feet)	Length of Reservoir[e] (Miles)	Miles of Shoreline[e]	Reservoir Surface Area[e] (Acres)	Area of Original River Bed (Acres)	Reservoir Elevation (Feet Above Mean Sea Level): Jan. 1 Flood Guide Elevation	Reservoir Elevation: Top of Gates	Reservoir Elevation: June 1 Flood Guide Elevation	Reservoir Volume (Acre Feet): At Jan. 1 Flood Guide Elevation	Reservoir Volume: At Top of Gates	Reservoir Volume: At June 1 Flood Guide Elevation	Jan. 1 Controlled Storage (Acre Feet)[f]	Project	Number of Dams in Project
Kentucky[g]	Tennessee	KY	40,200	128.8	7/1/1938	8/30/1944	9/14/1944	1/16/1948	184	5	22.4	206	8,422	C G E	110x600x75[v]	184.3	2064.3	160,300	25,200	354.0	375.00	359.0	2,121,000	6,129,000	2,839,000	4,008,000	TN River	9
Pickwick Landing	Tennessee	TN	32,820	120.9	12/30/1934	2/8/1938	6/29/1938	12/31/1952	229	6	206.7	113	7,715	C G E	110x1000x63[i] 110x600x63 60x232x48	52.7	490.6	42,700	9,580	408.0	418.00	414.0	839,300	1,332,000	1,119,000	492,700	TN River	9
Wilson[h]	Tennessee	AL	30,750	133.5	4/14/1918	4/14/1924	9/12/1925	4/12/1962	663	21	259.4	137	4,541	C G	110x600x100[i] 60x300x52 60x292x48	15.5	166.2	15,600	9,108	504.7	507.88	507.7	589,700	640,200	637,200	50,500	TN River	9
Wheeler	Tennessee	AL	29,590	69.0	11/21/1933	10/3/1936	11/9/1936	12/18/1963	361	11	274.9	72	6,342	C G	60x400x52 110x600x52[i]	74.1	1027.2	67,070	17,600	550.5	556.28	556.0	742,600	1,069,000	1,050,000	326,500	TN River	9
Guntersville	Tennessee	AL	24,450	74.2	12/4/1935	1/16/1939	8/1/1939	3/24/1952	124	4	349.0	96.5	3,979	C G E	60x360x45 110x600x45[i]	75.7	889.1	66,000	12,065	593.0	595.44	595.0	886,600	1,048,700	1,018,000	162,100	TN River	9
Nickajack	Tennessee	TN	21,870	56.1	4/1/1964	12/14/1967	2/20/1968	4/30/1968	105	4	424.7	86	3,767	C G E	110x800x41[j] 110x600x41	46.3	178.7	10,200	4,200	632.5-634.5	635.00	632.5-634.5	N/A	251,600	N/A	N/A	TN River	9
Chickamauga	Tennessee	TN	20,790	74.4	1/13/1936	1/15/1940	3/4/1940	3/7/1952	119	4	471.0	129	5,800	C G E	60x360x53[v]	58.9	783.7	36,050	9,500	675.0	685.44	682.5	392,000	737,300	622,500	345,300	TN River	9
Watts Bar	Tennessee	TN	17,310	66.4	7/1/1939	1/1/1942	2/11/1942	4/24/1944	182	5	529.9	125[w]	2,960	C G E	60x360x70	95.5[r]	721.7	37,500	10,343	735.0	745.00	741.0	796,000	1,175,000	1,010,000	379,000	TN River	9
Fort Loudoun	Tennessee	TN	9,550	45.3	7/8/1940	8/2/1943	11/9/1943	1/27/1949	162	4	602.3	129[w]	4,190	C G E	60x360x80	60.8[s]	378.2	14,000	4,420	807.0	815.00	813.0	282,000	393,000	363,000	111,000	TN River	9
Pumped Storage Project																												
Raccoon Mountain	Tennessee	TN	1	237.8	7/1/1970	7/11/1978	12/31/1978	8/31/1979	1653	4[k]		230	8,500	E R	N/A			528		1530.0-1672.0			N/A		N/A	N/A	Raccoon Mtn.	1
Tributary Power Projects																												
Tims Ford	Elk	TN	529	43.8	3/28/1966	12/1/1970	3/1/1972	3/1/1972	36	1	133.3	175	1,580	E R	N/A	34.2	308.7	10,500	565	873.0	895.00	888.0	388,400	608,000	530,000	219,600	Elk River	1
Apalachia	Hiwassee	NC	1,018	29.4	7/17/1941	2/14/1943	9/22/1943	11/17/1943	82	2	66.0	150	1,308	C G	N/A	9.8	31.5	1,100	80	1272.0-1280.0	1280.00	1272.0-1280.0	N/A	57,800	N/A	N/A	Hiwassee	4
Hiwassee	Hiwassee	NC	968	42.5	7/15/1936	2/8/1940	5/21/1940	5/24/1956	141	2[l]	75.8	307	1,376	C G	N/A	22.2	164.8	5,870	1,000	1485.0	1526.50	1521.0	228,400	434,000	399,000	205,600	Hiwassee	4
Chatuge	Hiwassee	NC	189	9.5	7/17/1941	2/12/1942	12/9/1954	12/9/1954	13	1	121.0	150	2,850	E	N/A	13.0	128.0	6,700	107	1918.0	1928.00	1926.0	177,900	240,500	226,600	62,600	Hiwassee	4
Ocoee 1[h][m]	Ocoee	TN	595	11.8	8/00/1910	12/15/1911	1/28/1912	0/0/1914	24	5	11.9	135	840	C G	N/A	7.5	47.0	1,620	170	820.0	830.76	829.0	64,300	83,300	79,900	19,000	Ocoee	3
Ocoee 2[h]	Ocoee	TN	512	28.8	5/00/1912	10/00/1913	10/0/1913	10/00/1913	23	2	24.2	30	450	O	N/A	N/A	N/A	N/A	N/A	N/A	1115.20	N/A	N/A	N/A	N/A	N/A	Ocoee	3
Ocoee 3	Ocoee	TN	492	4.9	7/17/1941	8/15/1942	4/30/1943	4/30/1943	29	1	29.2	110	612.1	C G	N/A	7.0	24.0	600	260	1428.0-1435.0	1435.00	1428.0-1435.0	N/A	4,200	N/A	N/A	Ocoee	3
Blue Ridge[h][m]	Toccoa	GA	232	20.4	11/00/1925[n]	12/6/1930	7/0/1931	7/0/1931	13	1	53.0	175	1,000	E	N/A	11.0	68.1	3,220	182	1668.0	1691.00	1687.0	127,400	195,900	182,800	68,500	Toccoa/ Ocoee	1
Nottely	Nottely	GA	214	17.2	7/17/1941	1/24/1942	1/10/1956	1/10/1956	18	1	21.0	197	2,300	R E	N/A	20.2	102.1	3,970	170	1762.0	1780.00	1777.0	112,700	174,300	162,000	61,600	Hiwassee	4
Melton Hill	Clinch	TN	3,343	21.5	9/6/1960	5/1/1963	7/3/1964	11/11/1964	79	2	23.1	103	1,020	C G	75x400x60	44	193.4	5,690	1,645	792.0-795.0	796.00	792.0-795.0	N/A	126,000	N/A	N/A	Clinch	2
Norris	Clinch	TN	2,912	46.1	10/1/1933	3/4/1936	7/28/1936	9/30/1936	110	2	79.6	265	1,860	C G E	N/A	129.0[u]	809.2	34,000	2,930	1000.0	1034.00	1,020.0	1,439,000	2,552,000	2,040,000	1,113,000	Clinch	2
Tellico	Little TN	TN	2,627	117.0	3/7/1967	11/29/1979	(o)	(o)	(o)	(o)	0.3	133[w]	3,238	C G E	(o)	33.2	357.0	15,600	2,133	807.0	815.00	813.0	304,000	424,000	392,000	120,000	Little TN	2
Fontana	Little TN	TN	1,571	69.1	1/1/1942	11/7/1944	1/20/1945	2/4/1954	304	3	61.0	480	2,365	C G	N/A	29.0	237.8	10,290	1,650	1653.0	1710.00	1703.0	929,000	1,443,000	1,370,000	514,000	Little TN	2
Douglas	French Broad	TN	4,541	83.0	2/2/1942	2/19/1943	3/21/1943	8/3/1954	111	4	32.3	215.5	1,705	C G	N/A	43.1	512.5	28,070	3,170	954.0	1002.00	994.0	379,000	1,461,000	1,223,500	1,082,000	French Broad	1
Cherokee	Holston	TN	3,428	29.3	8/1/1940	12/5/1941	4/16/1942	10/7/1953	148	4	52.3	178[w]	6,760	C G E R	N/A	54.0	394.5	29,560	2,426	1045.0	1075.00	1071.0	791,600	1,541,000	1,422,900	749,400	Holston	4
Fort Patrick Henry	South Fork Holston	TN	1,903	18.9	5/14/1951	10/27/1953	12/5/1953	2/22/1954	41	2	8.2	95	737	C G	N/A	10.4	31.0	840	339	1258.0-1,263.0	1263.00	1258.0-1263.0	N/A	26,900	N/A	N/A	Holston	4
Boone	South Fork Holston	TN	1,840	15.5	8/29/1950	12/16/1952	3/16/1953	9/3/1953	89	3	18.6	168	1,532	E C G	N/A	32.7[t]	126.6	4,130	719	1364.0	1385.00	1382.0	117,600	193,400	180,500	75,800	Holston	4
South Holston	South Fork Holston	TN	703	23.1	8/04/1947[p]	11/20/1950	2/13/1951	2/13/1951	44	1	49.8	285	1,600	E R	N/A	23.7	181.9	7,600	710	1708.0	1742.00	1729.0	511,300	764,000	658,000	252,800	Holston	4
Watauga	Watauga	TN	468	22.1	7/22/1946[p]	12/1/1948	8/30/1949	9/29/1949	66	2	36.7	332	900	E R	N/A	16.3	104.9	6,440	313	1952.0	1975.00	1959.0	524,200	677,000	568,500	152,800	Watauga	2
Wilbur[h]	Watauga	TN	471	1.6	00/00/1909	00/00/1912	0/0/1912	7/19/1950	11	4	34.0	76.33	375.5	C G	N/A	1.8	4.8	70		1641.0-1648.0	1650.00	1641.0-1648.0	N/A	714	N/A	N/A	Watauga	2
Great Falls[a][h]	Caney Fork	TN	1,675	21.4	12/7/1915	12/8/1916	0/0/1916	0/0/1925	36	2	91.1	92	800	C G	N/A	22.0	120.0	1,830	1,490	785.0	805.30	800.0	19,700	50,200	40,600	30,500	Caney Fork	1
Nolichucky (retired)[h][m]	Nolichucky	TN	1,183	0.1		00/00/1913			(q)	(q)	46.0	94	482	C G			26.0	380			1240.90						Nolichucky	1

Table 2.4.1-2 Facts About TVA Dams and Reservoirs
(Page 2 of 2)

a) All in the Tennessee Valley, except for Great Falls which is in the Cumberland Valley

b) Cost of plant including the inception balance of the plant and all additions and retirements from the plant. Transmission assets are not included.

c) Winter net dependable capacity as of October 2009. Winter net dependable capacity is the amount of power a plant can produce on an average winter day, minus the electricity used by the plant itself.

d) E: Earth; R: Rock fill; G: Gravity; C: Concrete; O: Other (Codes for each dam are listed in order of importance.)

e) At June 1 flood guide elevation.

f) Volume between the January 1 elevation and top of gates.

g) Connected to Barkley Reservoir by 1-1/2 mile canal, which opened July 14, 1966

h) Acquired: Wilson by transfer from the U.S. Army Corps of Engineers in 1933; Ocoee 1, Ocoee 2, Blue Ridge, and Great Falls by purchase from Tennessee Electric Power Company in 1939; Wilbur and Nolichucky (retired) by purchase from East Tennessee Power and Light Company in 1945. Subsequent to acquisition, TVA installed additional units at Wilson and Wilbur. Reconstructed flume at Ocoee 2 was placed in service in November 1983.

i) Main locks placed in operation in 1959 at Wilson, 1963 at Wheeler, 1965 at Guntersville, and 1984 at Pickwick Landing.

j) Construction of main lock at Nickajack limited to underwater construction.

k) Generating units at Raccoon Mountain are reversible Francis type pump-turbine units, each with 428,400 kW generator rating and 612,000 hp pump motor rating.

l) Unit 2 at Hiwassee is a reversible Francis type pump-turbine unit with 95,000 kW generator rating and 121,530 hp pump motor rating at 200 ft. net head.

m) Ocoee 1 creates Parksville Reservoir, Nolichucky (retired) creates Davy Crockett Reservoir, and Blue Ridge creates Toccoa Reservoir.

n) Construction of Blue Ridge discontinued early in 1926, resumed in March 1929.

o) Tellico project has no lock or powerhouse. Streamflow through navigable canal to Fort Loudoun Reservoir permits navigation and increases average annual energy output at Fort Loudoun.

p) Initial construction of South Holston and Watauga started February 16, 1942; temporarily discontinued to conserve critical materials during WWII.

q) Generating units at Nolichucky were removed from system generating capacity in August 1972. The dam was renovated and modified to convert the reservoir for use as a wildlife preserve.

r) Includes 72.4 miles up the Tennessee River to Fort Loudoun Dam and 23.1 miles up Clinch River to Melton Hill Dam.

s) Includes 6.5 miles up the French Broad River and 4.4 miles up the Holston River.

t) Includes 17.4 miles up the South Fork Holston River and 15.3 miles up the Watauga River.

u) Includes 73 miles up the Clinch River and 56 miles up the Powell River.

v) The U.S. Army Corps of Engineers is increasing the size of lock structures at Kentucky and Chickamauga.

w) The structural height of the dam is the vertical distance from the lowest point of the excavated foundation to the top of the dam. Top of dam refers to the highest poing of the water barrier on an embankment (or top of parapet wall) and deck elevation (or top of parapet wall) for concrete structures

x) As an interim measure to prevent overtopping, these four dams were raised by HESCO Concertainer floodline units.

Watts Bar - 3 feet: embankment at elevation 767 raised to elevation 770.
Fort Loudoun - 3.75 feet: embankment at elevation 830 was raised 7 feet to elevation 837 (3.75 feet above top of concrete wall at elevation 833 25).
Tellico - 4 feet: embankment at elevation 830 raised to elevation 834.
Cherokee - 3 feet: embankment at elevation 1089 raised to elevation 1092.

Table 2.4.1-3 TVA Dams - River Mile Distances to SQN
(Page 1 of 2)

River	Structure/River Mouth	River Mile[(a)]	Distance from SQN (mi.)
Tennessee River			
	Chickamauga Dam	471	13.7
	SQN	484.7	
	Hiwassee River	499.5	14.8
	Watts Bar Dam	530	45.3
	Clinch River	568	83.3
	Little Tennessee River	601	116.3
	Fort Loudoun Dam	602	117.3
	Holston River	652	167.3
	French Broad River	652	167.3
Hiwassee River		0	14.8
	Ocoee River	34.5	49.3
	Apalachia Dam	66	80.8
	Hiwassee Dam	76	90.8
	Nottely River	92	106.8
	Chatuge Dam	121	135.8
Ocoee River		0	49.3
	Ocoee #1 Dam	12	61.3
	Ocoee #2 Dam	24	73.3
	Ocoee #3 Dam	29	78.3
	Toccoa River	38(b)	87.3
Toccoa River		0	87.3
	Blue Ridge Dam	15(b)	102.3
Nottely River		0	106.8
	Nottely Dam	21	127.8
Clinch River		0	83.3
	Melton Hill Dam	23	106.3
	Norris Dam	80	163.3
Little Tennessee River		0	116.3
	Tellico Dam	0.5	116.8

Table 2.4.1-3 TVA Dams - River Mile Distances to SQN
(Page 2 of 2)

River	Structure/River Mouth	River Mile[a]	Distance from SQN (mi.)
	Chilhowee Dam	33.5	149.8
	Calderwood Dam	43.5	159.8
	Cheoah Dam	51.5	167.8
	Fontana Dam	61	177.3
Holston River		0	167.3
	Cherokee Dam	52	219.3
French Broad River		0	167.3
	Douglas Dam	32	199.3

a) Approximated to the one-half river mile based on U.S. Geological Survey Quadrangles river mile designations.
b) Estimated river mile. River miles not provided for Toccoa River on U.S. Geological Survey Quadrangles.

Table 2.4.1-4 Facts about TVA Dams Above Chickamauga

Project	Outlet Works			
	Spillway Type	Spillway Crest Elevation	Top of Gate Elevation	Capacity, cfs at Gate Top
Apalachia	Ogee, radial gates	1257	1280	135,900
Blue Ridge	Ogee, tainter gates	1675	1691	39,000
Boone	Ogee, radial gates	1350	1385	141,700
Chatuge	Concrete chute, curved weir, vertical-lift gates	1923	1928	11,700
Cherokee	Ogee, radial gates	1043	1075	255,900
Chickamauga	Concrete gravity, vertical-lift fixed roller gates	645	685.44	436,300
Douglas	Ogee, radial gates	970	1002	312,700
Fontana	Ogee, radial gates	1675	1710	107,300
Fort Loudoun	Ogee, radial gates	783	815	392,200
Fort Patrick Henry	Ogee, radial gates	1228	1263	141,700
Hiwassee	Ogee, radial gates	1503.5	1526.5	88,300
Melton Hill	Ogee, radial gates	754	796	115,600
Norris	Ogee, drum gates	1020	1034	55,000
Nottely	Concrete chute, curved weir vertical-lift gates	1775	1780	11,500
South Holston	Uncontrolled morning-glory with concrete-lined shaft and discharge tunnel	1742	N/A	41,200[a]
Tellico	Ogee, radial gates	773	815	117,900
Watauga	Uncontrolled morning-glory with concrete-lined shaft and discharge tunnel	1975	N/A	41,200[b]
Watts Bar	Ogee, radial gates	713	745	560,300

a) At elevation 1752.
b) At elevation 1985.

Table 2.4.1-5 Facts About Non-TVA Dams and Reservoirs

Projects	River	Drainage Area (sq. mi.)	Distance from Mouth (mi.)	Maximum Height, (ft.)	Length(ft.)	Area of Lake (ac.)	Length of Lake (mi.)	Total[1] Storage, (ac.-ft.)	Construction Started
Major Dams									
Calderwood	Little Tennessee	1,856	43.7	232	916	536	8	41,160	1928
Cheoah	Little Tennessee	1,608	51.4	225	750	595	10	35,030	1916
Chilhowee	Little Tennessee	1,976	33.6	91	1,373	1,690	8.9	49,250	1955
Nantahala	Nantahala	108	22.8	250	1,042	1,605	4.6	138,730	1930
Santeetlah	Cheoah	176	9.3	212	1,054	2,863	7.5	158,250	1926
Thorpe (Glenville)	West Fork Tuckasegee	36.7	9.7	150	900	1,462	4.5	70,810	1940
Minor Dams									
Bear Creek	East Fork Tuckasegee	75.3	4.8	215	740	476	4.6	34,711	1952
Cedar Cliff	East Fork Tuckasegee	80.7	2.4	165	600	121	2.4	6,315	1950
Mission (Andrews)	Hiwassee	292	106.1	50	390	61	1.46	283	1924
Queens Creek	Queens Creek	3.58	1.5	78	382	37	0.5	817	1947
Wolf Creek	Wolf Creek	15.2	1.7	180	810	176	2.2	10,056	1952
East Fork	East Fork Tuckasegee	24.9	10.9	140	385	39	1.4	1,797	1952
Tuckasegee	West Fork Tuckasegee	54.7	3.1	61	254	9	0.5	183	1949
Walters (Carolina P&L)	Pigeon	455	38.0	200	870	340	5.5	25,390	1927

(1) Volume at top of gates.

Table 2.4.1-6 Flood Detention Capacity - TVA Projects Above Sequoyah Nuclear Plant

Project	Flood Storage January 1 (ac-ft)	Flood Storage March 15 (ac-ft)	Flood Storage Summer (ac-ft)
Tributary			
Boone	75,800	60,000	12,900
Chatuge	62,600	62,600	13,900
Cherokee	749,400	749,400	118,100
Douglas	1,082,000	1,020,000	237,500
Fontana	514,000	514,000	73,000
Hiwassee	205,600	205,600	35,000
Norris	1,113,000	1,113,000	512,000
Nottely	61,600	61,600	12,300
South Holston	252,800	220,000	106,000
Tellico	120,000	120,000	32,000
Watauga	152,800	152,800	108,500
Blue Ridge	68,500	49,500	13,100
Main River			
Fort Loudoun	111,000	111,000	30,000
Watts Bar	379,000	379,000	165,000
Total	4,948,100	4,818,500	1,469,300

Table 2.4.2-1 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 1 of 5)

Water Year[a]	Date	Discharge (cfs)
1867	3/11/1867	459,000
1874	5/01/1874	195,000
1875	3/01/1875	410,000
1876	12/31/1875	227,000
1877	4/11/1877	190,000
1878	2/25/1878	125,000
1879	1/15/1879	252,000
1880	3/18/1880	254,000
1881	12/03/1880	174,000
1882	1/19/1882	275,000
1883	1/23/1883	261,000
1884	3/10/1884	285,000
1885	1/18/1885	174,000
1886	4/03/1886	391,000
1887	2/28/1887	181,000
1888	3/31/1888	178,000
1889	2/18/1889	198,000
1890	3/02/1890	283,000
1891	3/11/1891	259,000
1892	1/17/1892	252,000
1893	2/20/1893	221,000
1894	2/06/1894	167,000
1895	1/12/1895	212,000
1896	4/05/1896	269,000
1897	3/14/1897	257,000
1898	9/05/1898	167,000
1899	3/22/1899	273,000
1900	2/15/1900	159,000
1901	5/25/1901	221,000
1902	1/02/1902	271,000
1903	4/11/1903	210,000

Table 2.4.2-1 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 2 of 5)

Water Year[(a)]	Date	Discharge (cfs)
1904	3/25/1904	144,000
1905	2/11/1905	146,000
1906	1/26/1906	140,000
1907	11/22/1906	222,000
1908	2/17/1908	163,000
1909	6/06/1909	163,000
1910	2/19/1910	86,600
1911	4/08/1911	198,000
1912	3/31/1912	190,000
1913	3/30/1913	222,000
1914	4/03/1914	105,000
1915	12/28/1914	185,000
1916	12/20/1915	197,000
1917	3/07/1917	341,000
1918	2/02/1918	270,000
1919	1/05/1919	189,000
1920	4/05/1920	275,000
1921	2/13/1921	213,000
1922	1/23/1922	229,000
1923	2/07/1923	188,000
1924	1/05/1924	143,000
1925	12/11/1924	138,000
1926	4/16/1926	92,900
1927	12/29/1926	249,000
1928	7/02/1928	184,000
1929	3/26/1929	248,000
1930	11/19/1929	180,000
1931	4/08/1931	125,000
1932	2/01/1932	192,000
1933	1/01/1933	241,000
1934	3/06/1934	215,000

Table 2.4.2-1 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 3 of 5)

Water Year[a]	Date	Discharge (cfs)
1935	3/15/1935	175,000
1936	3/29/1936	234,000
1937	1/04/1937	204,000
1938	4/10/1938	136,000
1939	2/17/1939	193,000
1940	9/02/1940	89,400
1941	7/18/1941	58,200
1942	3/22/1942	72,300
1943	12/30/1942	235,000
1944	3/30/1944	201,000
1945	2/18/1945	115,000
1946	1/09/1946	225,000
1947	1/20/1947	186,000
1948	2/14/1948	225,000
1949	1/06/1949	179,000
1950	2/02/1950	192,000
1951	3/30/1951	140,000
1952	(b)	(b)
1953	2/22/1953	107,000
1954	1/22/1954	185,000
1955	3/23/1955	118,000
1956	2/04/1956	187,000
1957	2/02/1957	208,000
1958	11/19/1957	189,000
1959	1/23/1959	110,000
1960	12/20/1959	108,000
1961	3/09/1961	178,000
1962	12/18/1961	190,000
1963	3/13/1963	219,000
1964	3/16/1964	122,000
1965	3/26/1965	180,000

Table 2.4.2-1 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 4 of 5)

Water Year[a]	Date	Discharge (cfs)
1966	2/16/1966	104,000
1967	7/08/1967	120,000
1968	12/23/1967	148,000
1969	2/03/1969	121,000
1970	12/31/1969	186,000
1971	2/07/1971	90,700
1972	1/11/1972	116,000
1973	3/18/1973	267,000
1974	1/11/1974	181,000
1975	3/14/1975	148,000
1976	1/28/1976	67,200
1977	4/05/1977	191,000
1978	1/28/1978	115,000
1979	3/05/1979	145,000
1980	3/21/1980	168,000
1981	2/12/1981	50,800
1982	1/04/1982	133,000
1983	5/21/1983	116,000
1984	5/9/1984	239,000
1985	2/02/1985	81,000
1986	2/18/1986	66,200
1987	2/27/1987	109,000
1988	1/21/1988	74,100
1989	6/21/1989	173,000
1990	2/19/1990	169,000
1991	12/23/1990	185,000
1992	12/04/1991	146,000
1993	3/24/1993	113,000
1994	3/28/1994	202,000
1995	2/18/1995	99,900
1996	1/28/1996	145,000

Table 2.4.2-1 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 5 of 5)

Water Year[(a)]	Date	Discharge (cfs)
1997	3/04/1997	138,000
1998	4/19/1998	207,000
1999	1/24/1999	91,400
2000	4/05/2000	137,000
2001	2/18/2001	86,100
2002	1/24/2002	184,100
2003	5/8/2003	241,000
2004	9/18/2004	160,000
2005	12/13/2004	153,000
2006	1/23/2006	63,800
2007	1/09/2007	66,300

(a) Water Year runs from October 1 of prior year to September 30 of year identified.

(b) Not reported.

[36]

Table 2.4.3-1 Seasonal Variations of Rainfall (PMP)

	Antecedent (in.)				3-Day PMP (in.)	
Month	Ratio to Main Storm (Percent)	7,980 Sq.-Mi. Basin	21,400 Sq.-Mi. Basin	Dry Interval Before PMP (Days)	7,980 Sq.-Mi. Basin	21,400 Sq.-Mi. Basin
March	40	8.14	6.71	3	20.36	16.78
April	40	8.08	6.44	3	20.20	16.11
May	40	7.96	6.10	3	19.92	15.27
June	40	7.81	5.63	3	19.53	14.09
July	30	5.72	3.87	2½	19.07	12.92
August	30	5.72	3.87	2½	19.07	13.09
September	30	6.09	4.47	2½	20.30	14.92

Source: HMR Report 41

Table 2.4.3-2 **Probable Maximum Storm Precipitation and Precipitation Excess**
(Page 1 of 2)

		Antecedent Storm		Main Storm	
Index No.	**Unit Area[a] Name**	**Rain, (Inches)**	**Excess[b] (Inches)**	**Rain, (Inches)**	**Excess[c] (Inches)**
1	Asheville	6.18	2.91	18.12	15.44
2	Newport, French Broad	6.18	3.67	18.42	16.43
3	Newport, Pigeon	6.18	2.91	19.26	16.58
4	Embreeville	6.18	3.67	15.30	13.31
5	Nolichucky Local	6.18	3.67	15.42	13.43
6	Douglas Local	6.18	4.43	17.16	15.94
7	Little Pigeon River	6.18	3.81	21.12	19.13
8	French Broad Local	6.18	3.81	19.38	17.39
9	South Holston	6.18	4.60	12.12	10.90
10	Watauga	6.18	3.67	12.96	10.97
11	Boone Local	6.18	3.81	13.86	11.87
12	Fort Patrick Henry	6.18	4.60	14.34	13.12
13	Gate City	6.18	4.60	12.30	11.08
14 & 15	Total Cherokee Local	6.18	4.60	15.42	14.20
16	Holston River Local	6.18	4.60	16.74	15.52
17	Little River	6.18	3.81	20.82	18.83
18	Fort Loudoun Local	6.18	3.81	17.28	15.29
19	Needmore	6.18	2.73	20.22	17.54
20	Nantahala	6.18	2.73	20.94	18.26
21	Bryson City	6.18	2.91	20.04	17.36
22	Fontana Local	6.18	2.91	19.56	16.88
23	Little Tennessee Local - Fontana to Chilhowee Dam	6.18	2.91	22.50	19.82
24	Little Tennessee Local - Chilhowee to Tellico Dam	6.18	2.91	19.26	16.58
25	Watts Bar Local above Clinch River	6.18	3.81	15.84	13.85
26	Norris Dam	6.18	4.60	13.56	12.34
27	Melton Hill Local	6.18	4.27	15.42	14.01
33	Local above mile 16	6.18	4.43	15.42	14.01
34	Poplar Creek	6.18	4.43	14.88	13.47
35	Emory River	6.18	4.43	12.78	11.37
36	Local Area at Mouth	6.18	4.43	14.94	13.53

Table 2.4.3-2 Probable Maximum Storm Precipitation and Precipitation Excess (Continued)
(Page 2 of 2)

		Antecedent Storm		Main Storm	
Index No.	**Unit Area[a] Name**	**Rain, (Inches)**	**Excess[b] (Inches)**	**Rain, (Inches)**	**Excess[c] (Inches)**
37	Watts Bar Local below Clinch River	6.18	4.43	14.28	12.87
38	Chatuge	6.18	2.91	21.12	18.44
39	Nottely	6.18	2.91	18.66	15.98
40	Hiwassee Local	6.18	2.73	18.18	15.50
41	Apalachia	6.18	3.81	18.18	16.19
42	Blue Ridge	6.18	2.91	22.14	19.46
43	Ocoee No. 1, Blue Ridge to Ocoee No. 1	6.18	2.91	18.42	15.74
44A	Hiwassee River Local at Charleston	6.18	3.81	15.48	13.49
44B	Hiwassee River Local mouth to Charleston	6.18	4.27	14.52	13.11
45	Chickamauga Local	6.18	4.27	13.56	12.15
	Average above Chickamauga Dam	6.18	3.85	16.25	14.39

a. Unit area corresponds to Figure 2.4.3-5 numbered areas.

b. Adopted antecedent precipitation index prior to antecedent storm varies by unit area, ranging from 0.78-1.29 inches.

c. Computed antecedent precipitation index prior to main storm, 3.65 inches.

Table 2.4.3-3 Historical Flood Events

Unit Area	Basin	Flood	Rain (in.)	Runoff (in.)
1	French Broad at Asheville	4/05/1957	5.53	2.30
		5/03/2003	5.66	1.44
2	French Broad Newport Local	3/13/1963	5.31	2.47
		3/17/1973	4.68	2.20
		3/28/1994	5.60	2.33
3	Pigeon at Newport	3/28/1994	6.19	2.92
		5/06/2003	7.18	2.68
7	Little Pigeon at Sevierville	3/17/2002	4.61	3.46
		5/06/2003	6.19	3.85
9	South Holston Dam	3/12/1963	3.12	1.55
		3/16/1973	3.33	1.29
		3/18/2002	4.41	1.55
10	Watauga Dam	3/12/1963	3.64	2.16
		3/17/1973	3.61	1.84
		1/14/1995	6.97	3.75
17	Little River at Mouth	3/17/1973	6.26	3.82
18	Fort Loudoun Local	3/17/1973	6.81	3.14
23	Chilhowee Local	3/16/1973	6.97	3.24
		5/06/2003	6.19	3.13
24	Tellico Local	3/17/1973	7.34	3.56
		5/06/2003	7.84	3.72
26	Norris Dam	3/17/2002	5.00	2.90
27	Melton Hill Local	3/16/1973	6.66	4.85
42	Blue Ridge Dam	3/29/1951	5.70	1.61
44A	Hiwassee at Charleston (RM 18.9)	3/27/1965	6.04	3.52
		3/16/1973	7.36	5.84

Table 2.4.3-4 Unit Hydrograph Data
(Page 1 of 2)

Unit Area Number	Unit Area Name	GIS Drainage Area (sq. mi.)	Duration (hrs.)	Q_p	C_p	T_p	W_{50}	W_{75}	T_B
1	Asheville	944.4	6	14,000	0.21	12	39	15	168
2	Newport,French Broad	913.1	6	43,114	0.66	12	10	4	48
3	Newport, Pigeon	667.1	6	30,910	0.65	12	8	4	90
4	Embreeville	804.8	4	33,275	0.65	12	10	7	80
5	Nolichucky Local	378.7	6	11,740	0.44	12	14	6	90
6	Douglas Local	835	6	47,207	0.27	6	8	5	60
7	Little Pigeon River	352.1	4	17,000	0.75	12	10	6	66
8	French Broad Local	206.5	6	8,600	0.20	6	13	6	60
9	South Holston	703.2	6	15,958	0.53	18	25	17	96
10	Watauga	468.2	4	37,002	0.74	8	6	3	32
11	Boone Local	667.7	6	22,812	0.16	6	13	7	90
12	Fort Patrick Henry	62.8	6	2,550	0.19	6	12	7	66
13	Gate City	668.9	6	11,363	0.56	24	34	26	108
14&15	Total Cherokee Local	854.6	6	25,387	0.42	12	20	10	54
16	Holston River Local	289.6	6	8,400	0.27	9	18	12	96
17	Little River	378.6	4	11,726	0.68	16	15	7	96
18	Fort Loudoun Local	323.4	6	20,000	0.29	6	10	5	36
19	Needmore	436.5	6	9,130	0.49	18	22	12	126
20	Nantahala	90.9	2	3,130	0.38	8	16	11	54
21	Bryson City	653.8	6	26,000	0.43	10	13	7	60
22	Fontana Local	389.8	4	17,931	0.14	4	14	7	28
23	Little Tennessee Local-Fontana to Chilhowee Dam	404.7	6	16,613	0.58	12	10	4	84
24	Little Tennessee Local-Chilhowee to Tellico Dam	650.2	6	22,600	0.49	12	15	8	54
25	Watts Bar Local above Clinch River	295.3	6	11,063	0.18	6	10	4	90
26	Norris Dam	2912.8	6	43,773	0.07	6	18	6	102
27	Melton Hill Local	431.9	6	12,530	0.14	6	19	10	90

Table 2.4.3-4 Unit Hydrograph Data (Continued)
(Page 2 of 2)

Unit Area Number	Unit Area Name	GIS Drainage Area (sq. mi.)	Duration (hrs.)	Q_p	C_p	T_p	W_{50}	W_{75}	T_B
33	Local above mile 16	37.2	2	4,490	0.94	6	3	2	48
34	Poplar Creek	135.2	2	2,800	0.61	20	26	13	90
35	Emory River	868.8	4	36,090	0.39	8	11	6	84
36	Local area at Mouth	29.3	2	3,703	0.99	6	3	2	48
37	Watts Bar Local below Clinch River	408.4	6	16,125	0.19	6	10	4	90
38	Chatuge	189.1	1	19,062	0.24	2	3	2	37
39	Nottely	214.3	1	44,477	0.16	1	1	1	12
40	Hiwassee Local	565.1	6	23,349	0.58	12	11	6	96
41	Applachia	49.8	1	5,563	0.26	2	4	1	23
42	Blue Ridge	231.6	2	11,902	0.40	6	10	7	60
43	Ocoee No. 1 Local	362.6	6	17,517	0.23	6	12	8	36
44A	Hiwassee at Charletson	686.6	6	9,600	0.59	30	39	23	108
44B	Hiwassee at Mouth	396.0	6	16,870	1.00	18	11	6	78
45	Chickamauga Local	792.1	6	32,000	0.38	9	14	7	36

Definition of Symbols
Q_p = Peak discharge in cfs
C_p =Snyder coefficient
T_p = Time in hours from beginning of precipitation excess to peak of unit hydrograph
W_{50} = Width in hours at 50% of peak discharge
W_{75} = Width in hours at 75% of peak discharge
T_B = Base length in hours of unit hydrograph

Table 2.4.4-1

Floods from Postulated Seismic Failures of Upstream Dams
Plant Grade is Elevation 705.0 ft

OBE Failures With One-Half Probable Maximum Flood		Sequoyah Nuclear Plant Elevation (ft)
1.	Tellico - Norris	706.7
2.	Partial Fontana – Tellico[a]	702.2
3.	Partial Fontana. – Tellico – Hiwassee – Apalachia – Blue Ridge[a]	706.3
4.	Cherokee – Douglas - Tellico	708.6
SSE Failures With 25-Year Flood		
5.	Norris – Cherokee – Douglas – Tellico[b]	706.0

a. Includes failure of four ALCOA dams and one Duke Energy dam – Nantahala (Duke Energy, formerly ALCOA), upstream; Santeetlah, on a downstream tributary; and Cheoah, Calderwood, and Chilhowee, downstream. Fort Loudoun gates are inoperable in open position.

b. Gate opening at Fort Loudoun prevented by bridge failure.

Table 2.4.13-1 (Sheet 1)

Well and Spring Inventory
Within 2-Mile Radius of Sequoyah Nuclear Plant Site

(1972 Survey Only)

Map Ident. No.	Location Latitude	Longitude	Well Depth, Feet	Estimated Elevation, Feet Ground	Water Surface	Well Dia., Feet	Remarks
1	35°13'34"	85°06'09"	--	725	--	.5	Serves 2 families; submersible
2	35°13'23"	85°06'12"	75	720	685	.5	Submersible pump
3	35°13'30"	85°06'47"	116	745	--	.5	Submersible pump
4	35°13'58"	85°05'45"	42	700	696	3.0	
5	35°14'15"	85°06'25"	--	680	--	.5	1/4-hp pump
6	35°14'34"	85°06'46"	85	720	--	15	Submersible pump
7	35°14'35"	85°06'52"	65	720	670	2.5	3/4-hp pump
8	35°14'36"	85°06'57"	73	735	687	.5	1/3-hp pump
9	35°15'06"	85°06'32"	27	780	761	5.0	Bucket
10	35°14'46"	85°06'16"	110	720	--	.5	Submersible
11	35°14'55"	85°06'15"	--	725	--	-	
12	35°14'53"	85°06'13"	77	800	--	.5	
13	35°14'52"	85°06'13"	--	800	--	-	Summer home
14	35°14'50"	85°06'12"	--	800	--	-	Summer home
15	35°14'45"	85°06'14"	50	720	680	.5	
16	35°14'44"	85°06'18"	275	795	525	.5	1-hp submersible pump
17	35°14'45"	85°06'22"	--	740	--	.5	1-hp pump
18	35°14'21"	85°05'30"	--	695	--	-	
19	35°14'26"	85°05'27"	200	695	--	.5	1-hp pump
20	35°14'34"	85°05'29"	150	695	--	.5	1/2-hp pump
21	35°14'31"	85°05'29"	--	695	--	.5	
22	35°14'29"	85°05'29"	110	690	--	.5	1-hp pump
23	35°14'23"	85°05'32"	85	700	--	.75	1-hp jet pump
24	35°14'22"	85°05'40"	--	695	--	.5	Serves 2 familes; 1-hp pump
25	35°14'24"	85°05'46"	52	710	680	.5	3/4-hp pump
26	35°14'28"	85°05'45"	130	740	620	.5	
27	35°14'26"	85°05'41"	90	740	710	.5	
28	35°14'32"	85°05'44"	141	740	650	.5	
29	35°14'34"	85°05'44"	--	735	--	-	Summer home
30	35°14'38"	85°05'41"	58	700	670	.5	1/3-hp pump
31	35°14'41"	85°05'41"	--	720	--	.5	
32	35°14'45"	85°05'46"	--	715	--	-	
33	35°14'43"	85°05'47"	--	720	--	-	
34	35°14'41"	85°05'48"	--	695	--	-	Summer home
35	35°14'39"	85°05'50"	48	695	650	.5	1-hp pump
36	35°14'39"	85°05'53"	60	700	--	.5	Submersible pump
37	35°14'40"	85°05'58"	--	695	653	.5	1-hp pump
38	35°14'41"	85°05'56"	50	695	655	.5	3/4-hp pump
39	35°14'35"	85°05'54"	--	700	--	-	Summer home
40	35°14'36"	85°05'57"	--	700	--	-	
41	35°14'37"	85°06'01"	--	715	--	-	Summer home
42	35°14'33"	85°05'02"	223	720	530	.5	

NOTE: The information in this table is historic and not subject to updating revisions.

Table 2.4.13-1 (Sheet 2)

Well and Spring Inventory
Within 2-Mile Radius of Sequoyah Nuclear Plant Site

(1972 Survey Only)

Map Ident. No.	Location Latitude	Longitude	Well Depth, Feet	Estimated Elevation, Feet Ground	Water Surface	Well Dia., Feet	Remarks
43	35°14'46"	85°05'54"	65	695	655	.5	3/4-hp pump
44	35°14'47"	85°05'54"	95	705	655	.5	
45	35°14'48"	85°05'53"	--	700	--	-	Summer home
46	35°14'50"	85°05'53"	257	695	665	.5	1-hp submersible pump
47	35°14'52"	85°05'48"	--	710	--	-	Summer home
48	35°15'04"	85°05'56"	--	725	--	-	Summer home
49	35°15'06"	85°06'02"	--	720	--	-	Summer home
50	35°15'06"	85°06'05"	90	705	625	.5	Submersible pump
51	35°14'58"	85°06'06"	--	695	--	-	Summer home
52	35°15'01"	85°06'02"	65	720	680	.5	3/4-hp pump
53	35°14'47"	85°05'57"	46	700	670	.5	2 familes; 1-hp pump
54	35°14'42"	85°06'01"	48	695	675	.5	1/2-hp pump
55	35°14'41"	85°06'02"	--	695	--	-	Summer home
56	35°14'40"	85°06'03"	--	695	--	-	Summer home
57	35°14'37"	85°06'08"	155	690	670	.5	1-hp pump
58	35°14'34"	85°06'09"	--	695	--	-	
59	35°14'23"	85°05'53"	--	760	--	.5	Submersible pump
60	35°14'49"	85°05'58"	--	705	--	-	
61	35°13'01"	85°04'41"	--	720	--	-	Summer home
62	35°13'18"	85°04'24"	--	845	--	.5	1-hp pump
63	35°13'19"	85°04'23"	206	845	645	.5	1/2-hp pump
64	35°13'33"	85°04'19"	50	720	680	.5	1-hp pump
65	35°13'49"	85°04'14"	100	720	640	.5	Servies clubhouse, 15 houses
66	35°13'57"	85°03'55"	175	741	--	.6	1-hp pump
67	35°13'53"	85°03'49"	100	738	690	.5	1-hp submersible pump
68	35°13'50"	85°03'52"	133	720	675	.5	1/2-hp pump
69	35°13'48"	85°03'43"	85	736	--	.5	1-hp pump
70	35°13'43"	85°03'38"	80	780	--	.5	1-hp pump
71	35°13'37"	85°03'36"	130	800	715	.5	1-hp pump
72	35°13'38"	85°03'43"	--	800	--	-	Well not used
73	35°13'16"	85°03'30"	227	880	680	.5	Submersible pump
74	35°13'09"	85°03'41"	397	900	820	.5	2-hp pump
75	35°12'47"	85°03'58"	190	860	800	.5	Serves 2 families; submersible
76	35°13'03"	85°04'17"	--	720	--	-	Summer home
77	35°13'05"	85°04'10"	90	740	670	.5	1/2-hp pump
78	35°12'50"	85°04'13"	85	760	--	.5	1-hp pump
79	35°12'45"	85°03'59"	190	880	--	.5	Serves 2 families; 1-hp pump
80	35°12'26"	85°04'07"	290	860	--	.5	Serves 5 families; submersible

NOTE: The information in this table is historic and not subject to updating revisions.

Table 2.4.13-1 (Sheet 3)

Well and Spring Inventory
Within 2-Mile Radius of Sequoyah Nuclear Plant Site

(1972 Survey Only)

Map Ident. No.	Location Latitude	Longitude	Well Depth, Feet	Estimated Elevation, Feet Ground	Water Surface	Well Dia., Feet	Remarks
81	35°12'20"	85°04'33"	265	940	--	.5	Submersible pump
82	35°12'15"	85°04'34"	250	965	735	.5	1-hp submersible pump
83	35°12'24"	85°04'35"	305	965	665	.5	Submersible pump
84	35°12'22"	85°05'05"	135	740	690	.5	1-hp pump
85	35°12'21"	85°05'08"	120	740	--	.5	Serves 2 families; 3/4-hp jet pump
86	35°12'17"	85°05'06"	190	800	--	.5	3/4-hp submersible pump
87	35°12'23"	85°05'09"	--	740	--	.5	1-hp pump
88	35°12'16"	85°05'12"	55	740	720	2.5	Bucket
89	35°12'07"	85°05'09"	251	775	700	.5	Serves 2 families; 3/4-hp pump
90	35°11'54"	85°04'56"	170	980	--	.5	1/2-hp pump
91	35°12'19"	85°05'20"	125	740	705	.5	Submersible pump
92	35°12'22"	85°05'33"	--	725	--	-	Summer home
93	35°12'22"	85°05'35"	--	700	--	-	1-hp pump
94	35°12'22"	85°05'36"	--	705	--	-	Summer home
95	35°12'20"	85°05'44"	--	700	--	-	Summer home
96	35°12'04"	85°05'56"	160	700	--	.5	Serves 5 families; 1-hp pump
97	35°12'04"	85°05'59	65	700	--	.5	House and cottage; 1-hp pump

NOTE: The information in this table is historic and not subject to updating revisions.

Table 2.4.13-2 (Sheet 1)

Ground Water Supplies Within 20-Mile
Radius of the Plant Site

(1972 Survey Only)

	Location	Owner	Average Daily Use mgd	Source	Approximate Distance From Site[a] (Miles)
1.	Chattanooga	Kay's Ice Cream Company	0.0400	Well	20.4
2.	Chattanooga	Selox, Inc.	0.0250	Well	21.0
3.	Chattanooga	Stainless Metal Products	0.0100	Well	16.4
4.	Chattanooga	American Cyanamid	0.0727	Well	21.0
5.	Chattanooga	Dixie Yarns, Inc.	0.5350	Wells (2) and Tennessee-American Water Company	13.3
6.	Chattanooga	Scholze Tannery	0.1560	Wells (2) and Tennessee-American Water Company	24.0
7.	Chattanooga	Southern Cellulose Products, Inc.	4.0000 0.1000	Well (1) and Tennessee-American Water Company	24.2
8.	Chattanooga	Alco Chemical Corporation	0.2300	Well (1) and Tennessee-American Water Company	--
9.	Chattanooga	Chattem Drug and Chemical	0.8500 0.2380	Wells (3) and Tennessee-American Water Company	24.0
10.	Chattanooga	Cumberland Corporation	0.2380 0.0150	Well (1) and Tennessee-American Water Company	17.4
11.	Chattanooga	Bacon Trailer Park		Well	--
12.	Dunlap	Bethel Church of Christ		Well	20.0
13.	Dayton	Blue Water Trail and Campground		Well	19.0
14.	Cleveland	Cohulla Baptist Church		Well	9.5
15.	Dayton	Crystal Springs Recreation Area		Spring	19.0
16.	Georgetown	Eastview School		Well	9.5
17.	Dayton	Fort Bluff Youth Camp		Well	19.0
18.	Dayton	Frazier Elementary School		Well	19.0
19.	Birchwood	Grasshopper Church of God		Well	11.3

NOTE: The information in this table is historic and not subject to updating revisions.

Table 2.4.13-2 (Sheet 2)

Ground Water Supplies Within 20-Mile Radius of the Plant Site

(1972 Survey Only)

	Location	Owner	Average Daily Use mgd	Source	Approximate Distance From Site[a] (Miles)
20.	Dayton	Hastings Mobile Home Park		Spring	19.0
21.	Ooltewah	High Point Baptist Church		Well	10.0
22.	Dayton	Lake Richland Apartments		Well	19.0
23.	Dayton	Laurelbrook Sanitarium School	.017	Wells (7)	19.0
24.	Cleveland	Labanon Baptist Church		Well	13.5
25.	Cleveland	Mt. Carmel Baptist Church		Well	13.5
26.	Sale Creek	Mt. Vernon Baptist Church		Well	11.0
27.	Dayton	Mt. Vista Mobile Home Park		Wells (2)	19.0
28.	Dayton	New Bethel Methodist Church		Well	19.0
29.	Cleveland	New Friendship Baptist Church		Well	13.5
30.	Dayton	Ogden Baptist Church		Well	19.0
31.	Dunlap	Old Union Water System		Spring	20.0
32.	Dunlap	P.A.W., Inc. #2		Well	20.0
33.	Cleveland	Red Clay State Historic Area		Well	13.5
34.	Chattanooga	Riverside Catfish House		Well	25.0
35.	Cleveland	Robert Allen		Well	13.5
36.	Dayton	Salem Baptist Church		Well	19.0
37.	Dunlap	Sequatchie-Bledsoe VO-Training		Well	20.0
38.	Dayton	Seventh Day Adventist Church		Well	19.0
39.	Chattanooga	Shamrock Motel		Well	20.1
40.	Dayton	Sinclair Packing House		Well	19.0
41.	Dunlap	Stonecave Institute Water System	0.0064	Spring	20.0
42.	Dunlap	Old Union Water System		Spring	20.0
43.	Sale Creek	Sale Creek Marina Multiboating		Well	11.0

NOTE: The information in this table is historic and not subject to updating revisions.

Table 2.4.13-2 (Sheet 3)

Ground Water Supplies Within 20-Mile Radius of the Plant Site

(1972 Survey Only)

	Location	Owner	Average Daily Use mgd	Source	Approximate Distance From Site[a] (Miles)
44.	Sale Creek	Sale Creek P.U.A. - TVA		Well	11.0
45.	Sale Creek	Sale Creek Utility District	0.204	Wells (2)	10.8
46.	Graysville	Graysville Water Supply	0.220	Wells (2)	15.0
47.	Graysville	Graysville Nursing Home		Well	15.0
48.	Dayton	Dayton Golf & CC % Mokas		Well	19.0
49.	Birchwood	Birchwood School		Well	11.3
50.	Cleveland	Cassons Grocery Water System	0.0170	Well	19.7
51.	Cleveland	Black Fox School		Well	13.5
52.	Cleveland	Blue Springs Baptist Church		Well	13.5
53.	Cleveland	Blue Springs School		Well	13.5
54.	Cleveland	Bradley Limestone, Div. of Dalton Rock Product Co.	0.2400	Well	13.5
55.	Cleveland	Hardwick Stone Company	0.1130	Well	13.5
56.	Cleveland	Cleveland-Tenn. Enamel	0.2240	Well	13.5
57.	Cleveland	Magic Chef, Inc.	0.4200	Spring	13.5
58.	Hamilton County	Savannah Valley U.D.	0.720	Wells (2)	5.0
59.	Hamilton County	Eastside Utility District	3.0130 0.0920	Wells (3) and Tennessee American Water Company	7.9
60.	Hamilton County	Hixson Utility District	4.0000 0.3330	Cave Springs (3) and Tennessee American Water Company	12.9
61.	Soddy	Union Fork Bakewell, U.D.	0.192 0.0010	Wells (3) and Sale Creek Utility District	9.8
62.	Hamilton County	Walden's Ridge, U.D.	0.471	Wells (2)	17.4
63.	Hamilton County	Container Corporation of America	1.9200	Well	22.0
64.	Hamilton County	Dave L. Brown Company	0.0200	Well	--

NOTE: The information in this table is historic and not subject to updating revisions.

Table 2.4.13-2 (Sheet 4)

Ground Water Supplies Within 20-Mile
Radius of the Plant Site

(1972 Survey Only)

	Location	Owner	Average Daily Use mgd	Source	Approximate Distance From Site[a] (Miles)
65.	Hamilton County	De Sota, Inc.	0.0750	Well	--
66.	Hamilton County	Hamilton Concrete Products	0.0050	Spring	24
67.	Cleveland	Thompson Spring Baptist Church		Well	13.5
68.	Dayton	Vaughn Trailer Park		Well	19.0
69.	Dayton Church	Walden's Ridge Baptist		Well	19.0
70.	Dayton	Walden's Ridge Elementary School		Well	19.0
71.	Cleveland	White Oak Baptist Church		Well	13.5
72.	Bradley County	Bockman Childrens Home		Well	10.2
73.	Catoosa County	Catoosa County U.D.		Well	19.0

[a] River mile distance from differences (TRM 483.6) for supplies taken from the Tennessee River channel; radial distance to other supplies.

NOTE: The information in this table is historic and not subject to updating revisions.

Table 2.4.14-1

Time between Floods from Postulated Seismic Failures of Upstream Dams
and Sequoyah Nuclear Plant Elevation 703.0 ft

OBE Failures With One-Half Probable Maximum Flood	Flood Wave Travel Time (hr)[c]
1. Tellico - Norris	34
2. Partial Fontana – Tellico[a]	N/A
3. Partial Fontana. – Tellico – Hiwassee – Apalachia – Blue Ridge[a]	32
4. Cherokee – Douglas - Tellico	46
SSE Failures With 25-Year Flood	
5. Norris – Cherokee – Douglas – Tellico[b]	53

a. Includes failure of four ALCOA dams and one Duke Energy dam – Nantahala (Duke Energy, formerly ALCOA), upstream; Santeetlah, on a downstream tributary; and Cheoah, Calderwood, and Chilhowee, downstream. Fort Loudoun gates are inoperable in open position.

b. Gate opening at Fort Loudoun prevented by bridge failure.

c. Time from seismic dam failure to arrival of failure wave at SQN elevation 703.0 ft (two ft below plant grade).

(1) Elevation 705.0 ft not reached
(2) Elevation 703.0 ft not reached

ENCLOSURE 1

EVALUATION OF PROPOSED CHANGES

ATTACHMENT 3

Proposed SQN Units 1 and 2 UFSAR Figures (Public)

Figure 2.4.1-1 Topographic Map, Plant Vicinity

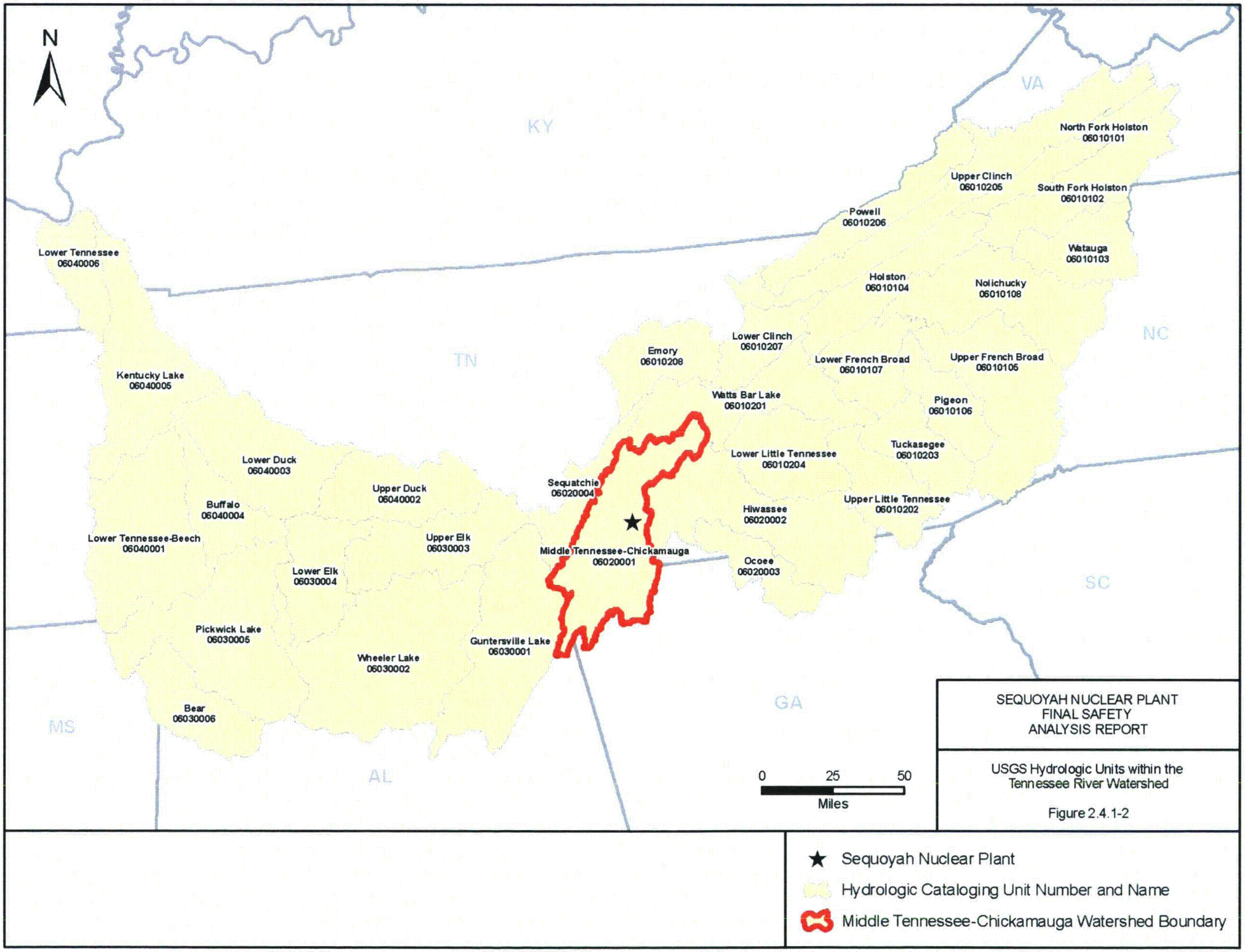
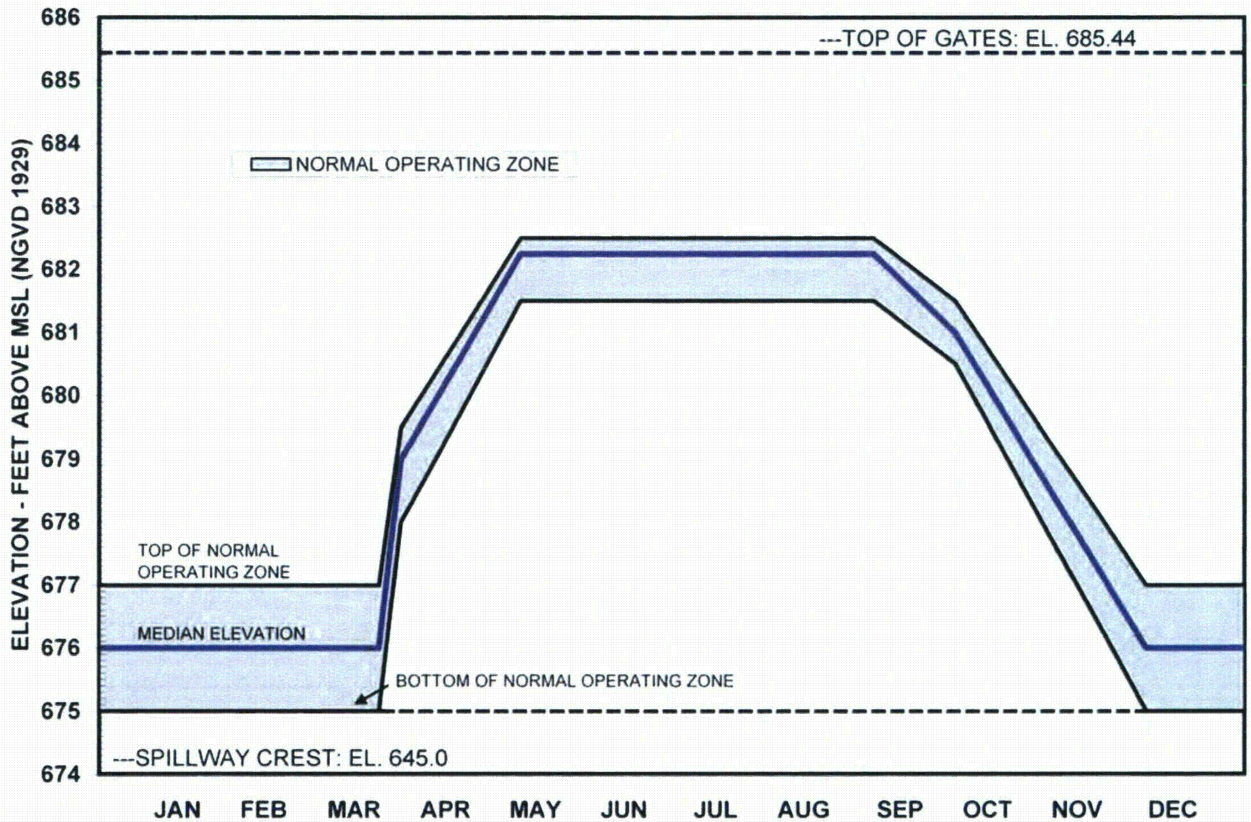


Figure 2.4.1-2 USGS Hydrologic Units within the Tennessee River Watershed

Figure 2.4.1-3 TVA Water Control System

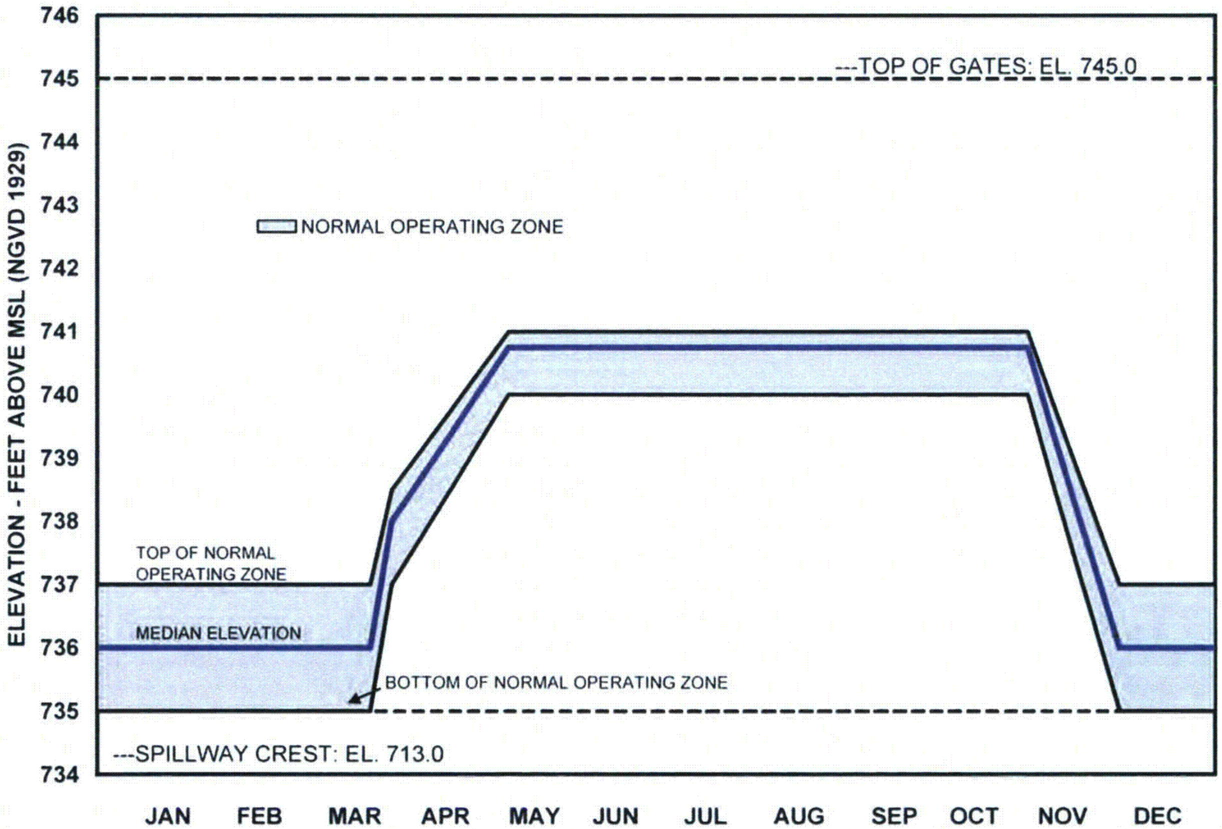


SEQUOYAH NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Seasonal Operating Curve, Chickamauga

Figure 2.4.1-4 (Sheet 1 of 16)

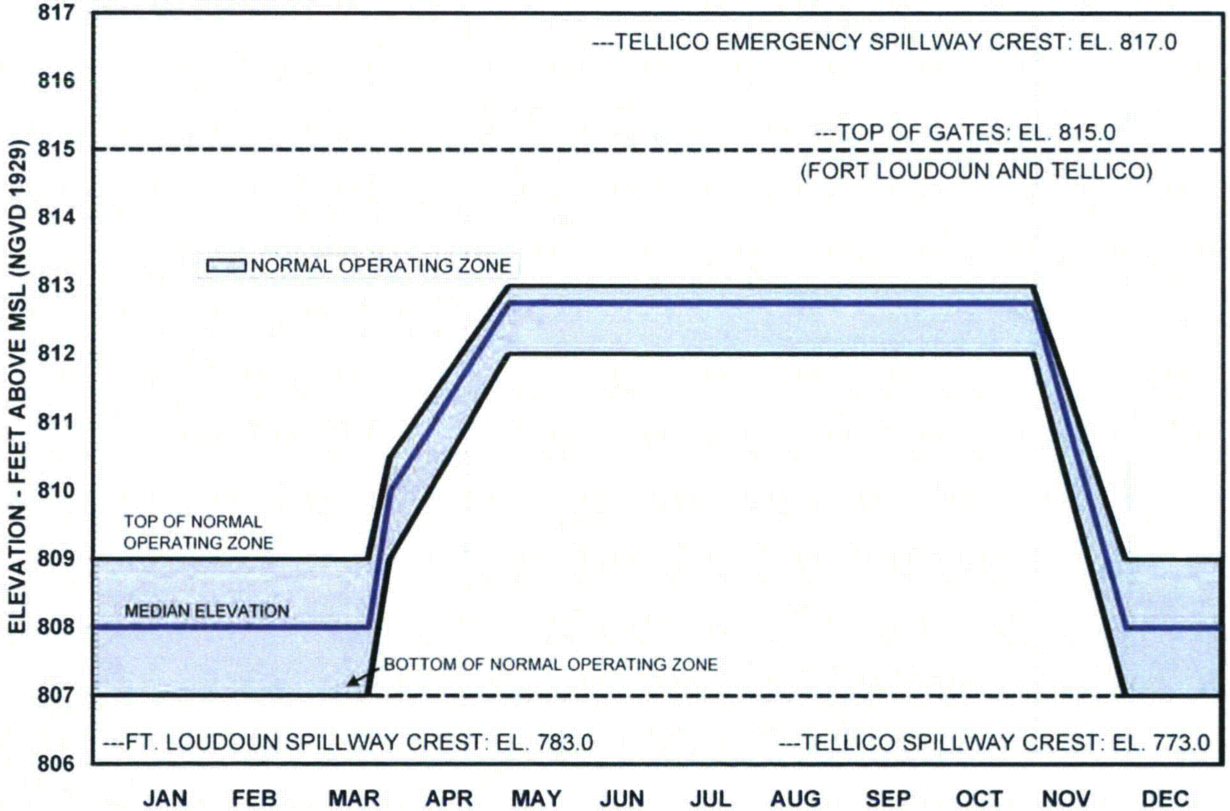
Figure 2.4.1-4 Seasonal Operating Curve, Chickamauga (Sheet 1 of 16)



Seasonal Operating Curve, Watts Bar

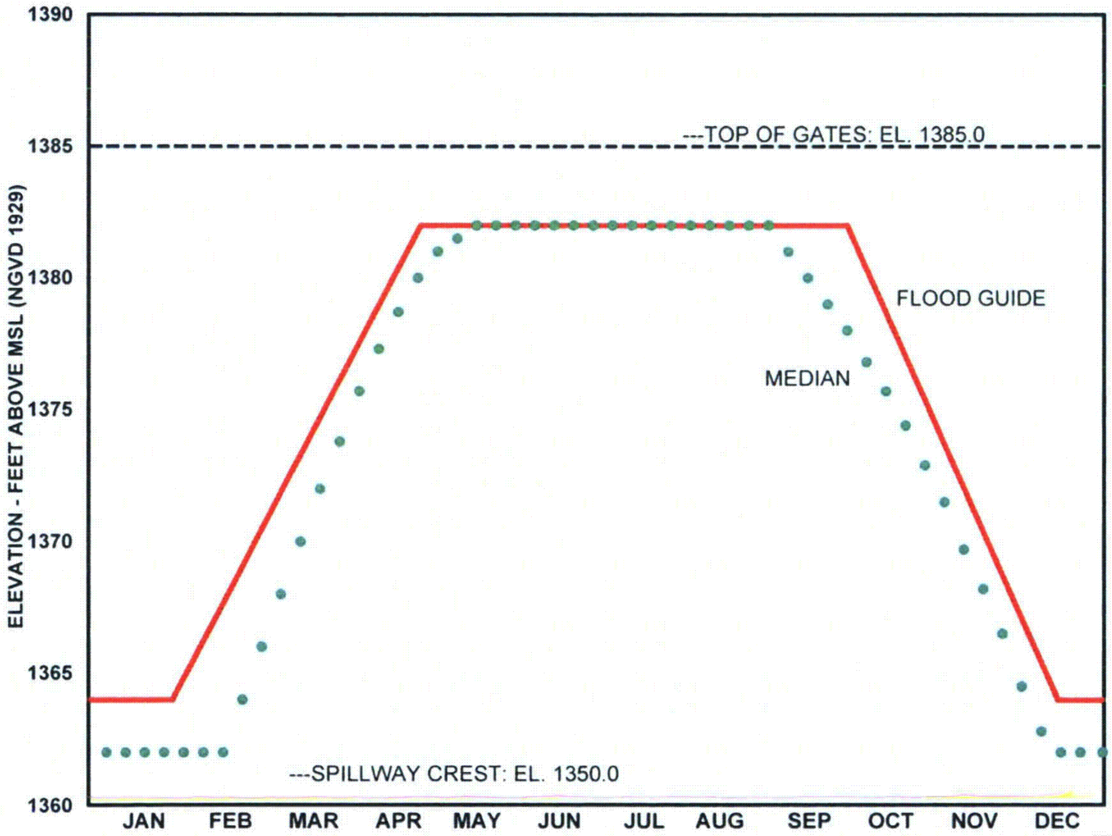
Figure 2.4.1-4 (Sheet 2 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Watts Bar (Sheet 2 of 16)



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Fort Loudoun - Tellico Figure 2.4.1-4 (Sheet 3 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Fort Loudoun - Tellico (Sheet 3 of 16)

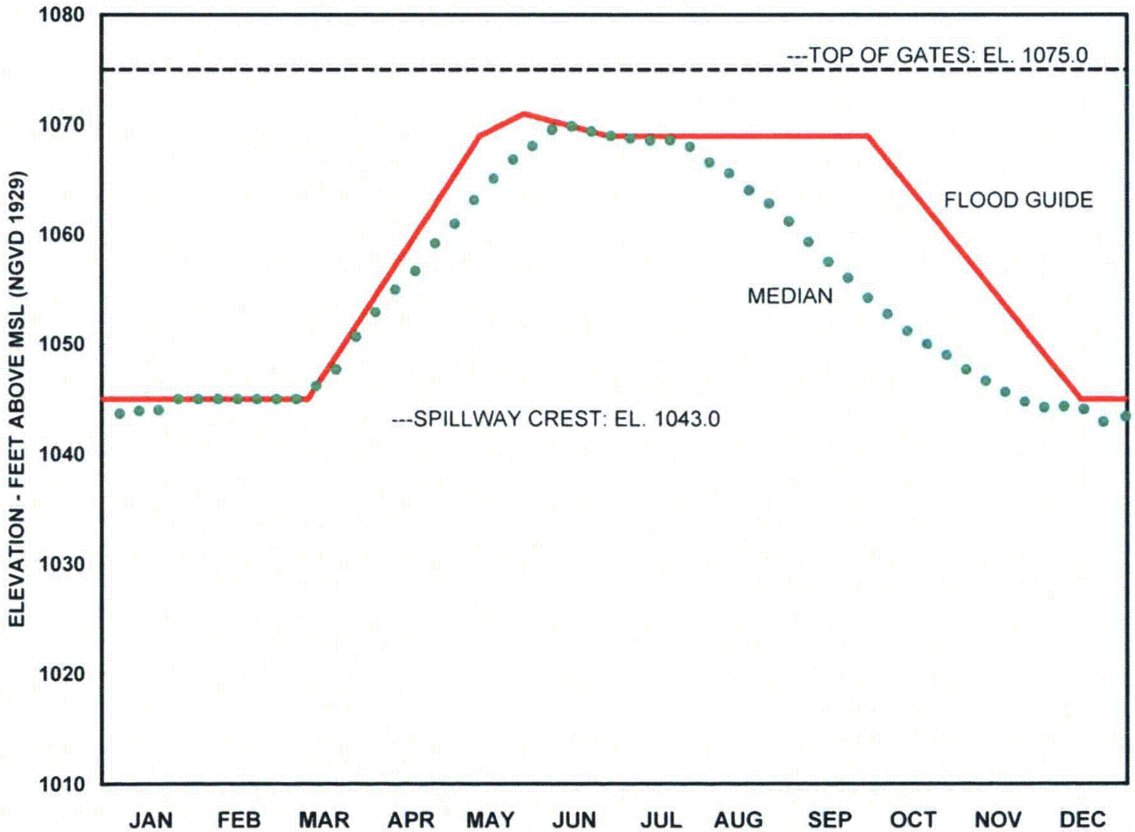


SEQUOYAH NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Seasonal Operating Curve, Boone

Figure 2.4.1-4 (Sheet 4 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Boone (Sheet 4 of 16)

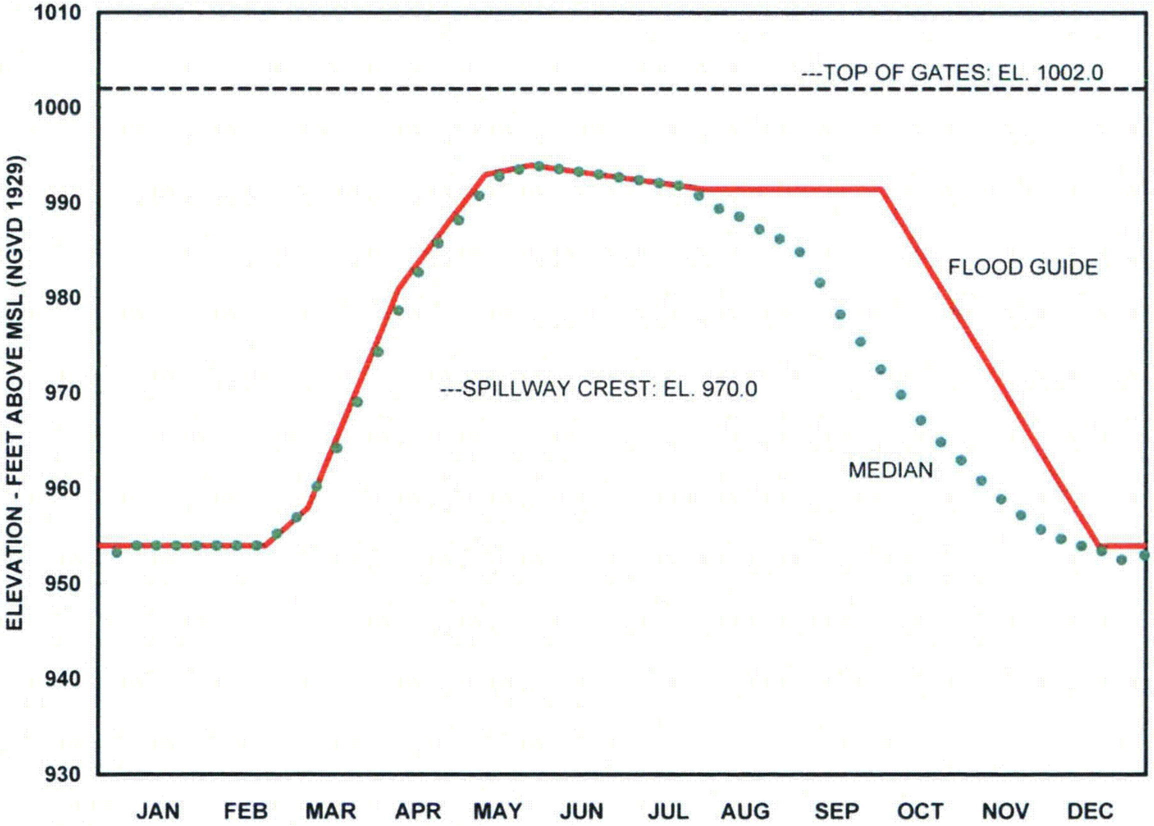


SEQUOYAH NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Seasonal Operating Curve, Cherokee

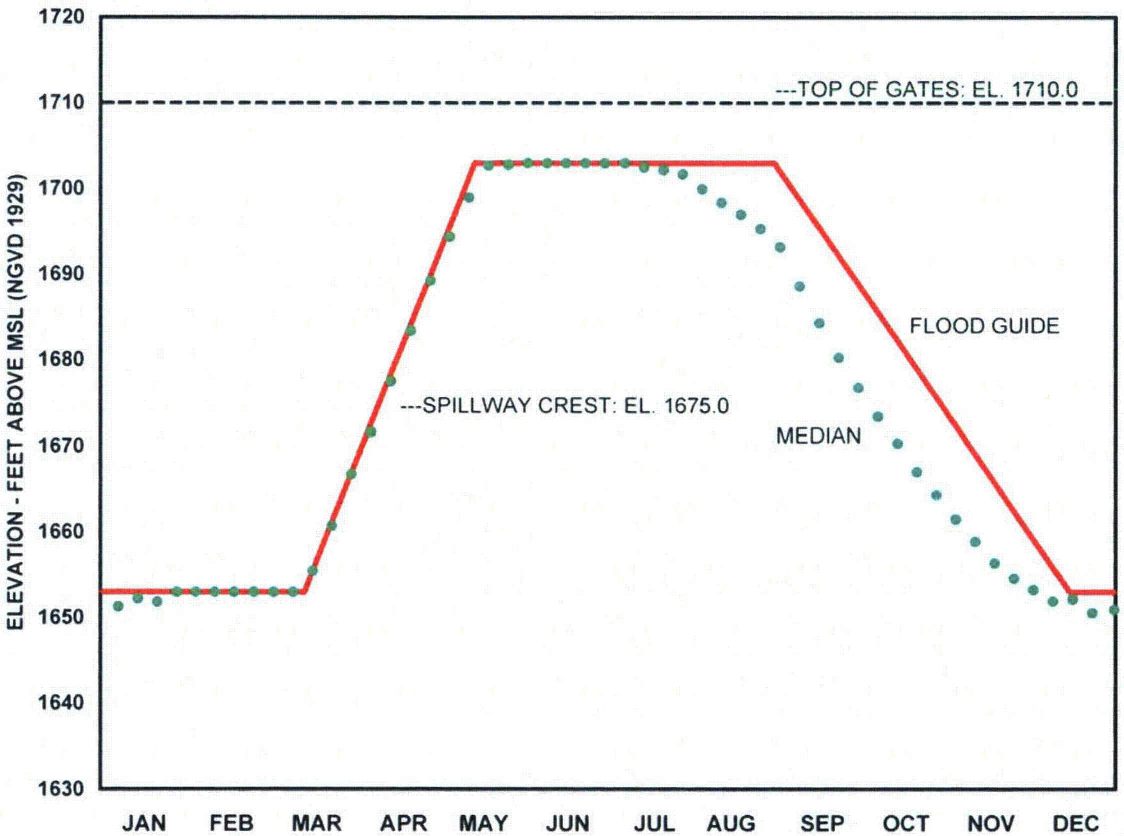
Figure 2.4.1-4 (Sheet 5 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Cherokee (Sheet 5 of 16)



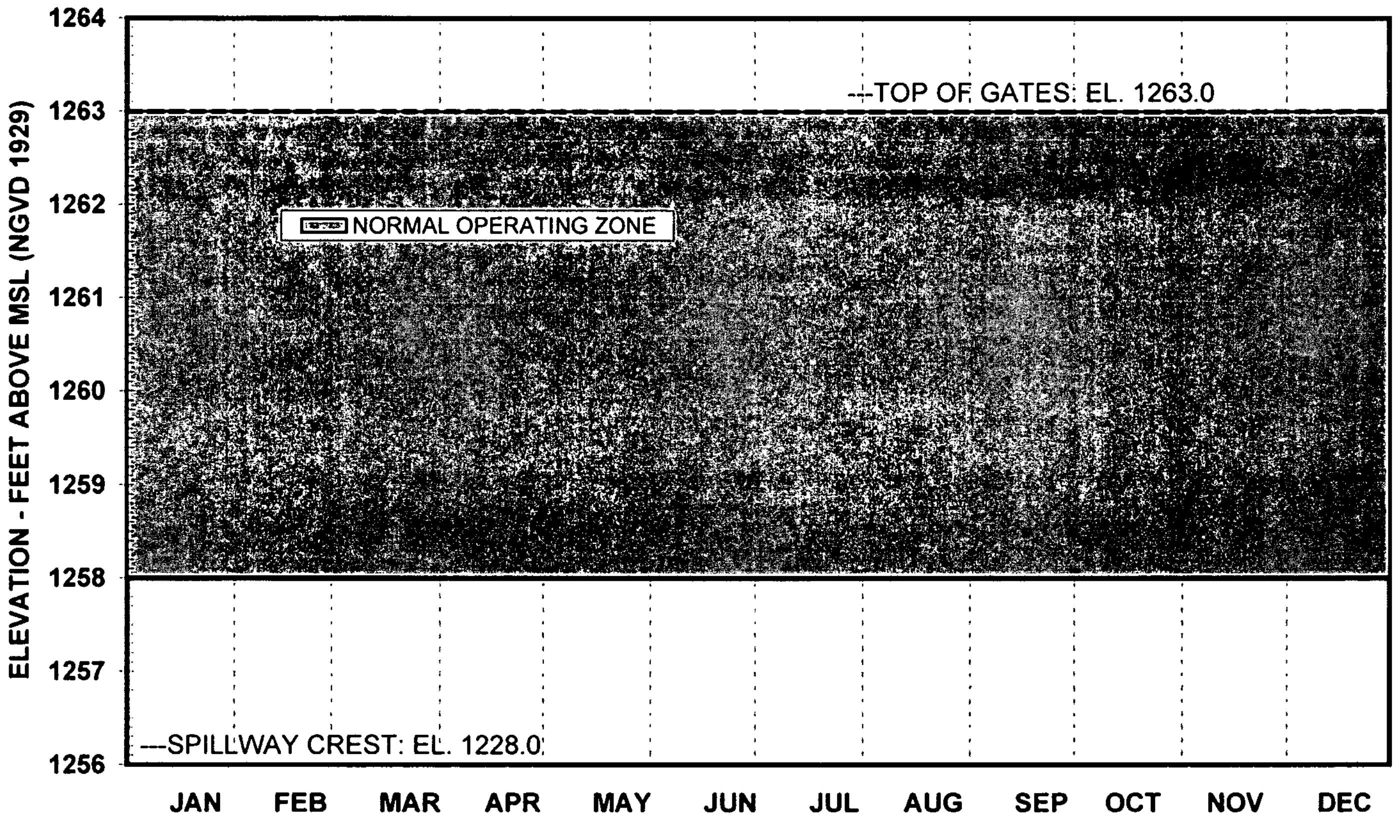
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Douglas
Figure 2.4.1-4 (Sheet 6 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Douglas (Sheet 6 of 16)



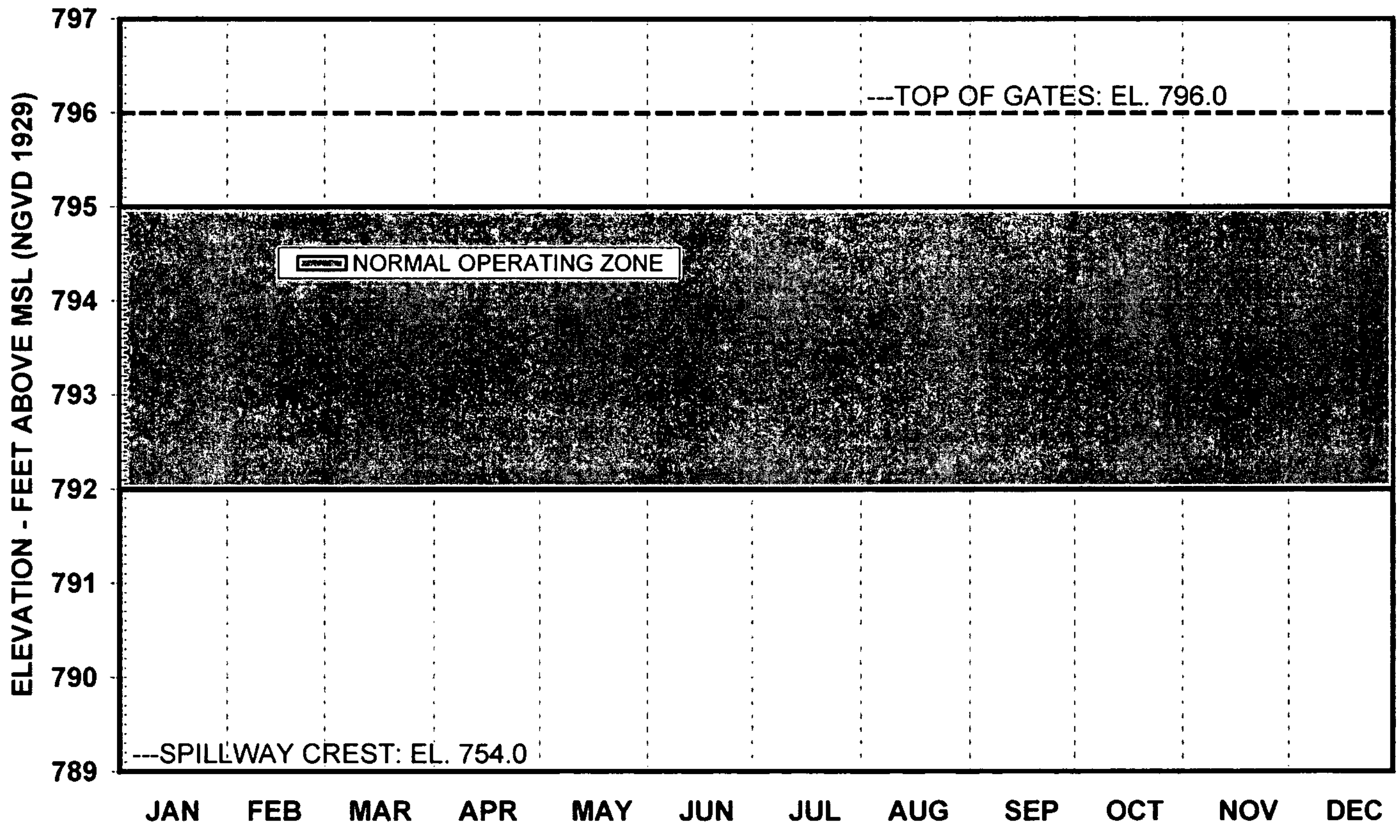
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Fontana Figure 2.4.1-4 (Sheet 7 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Fontana (Sheet 7 of 16)



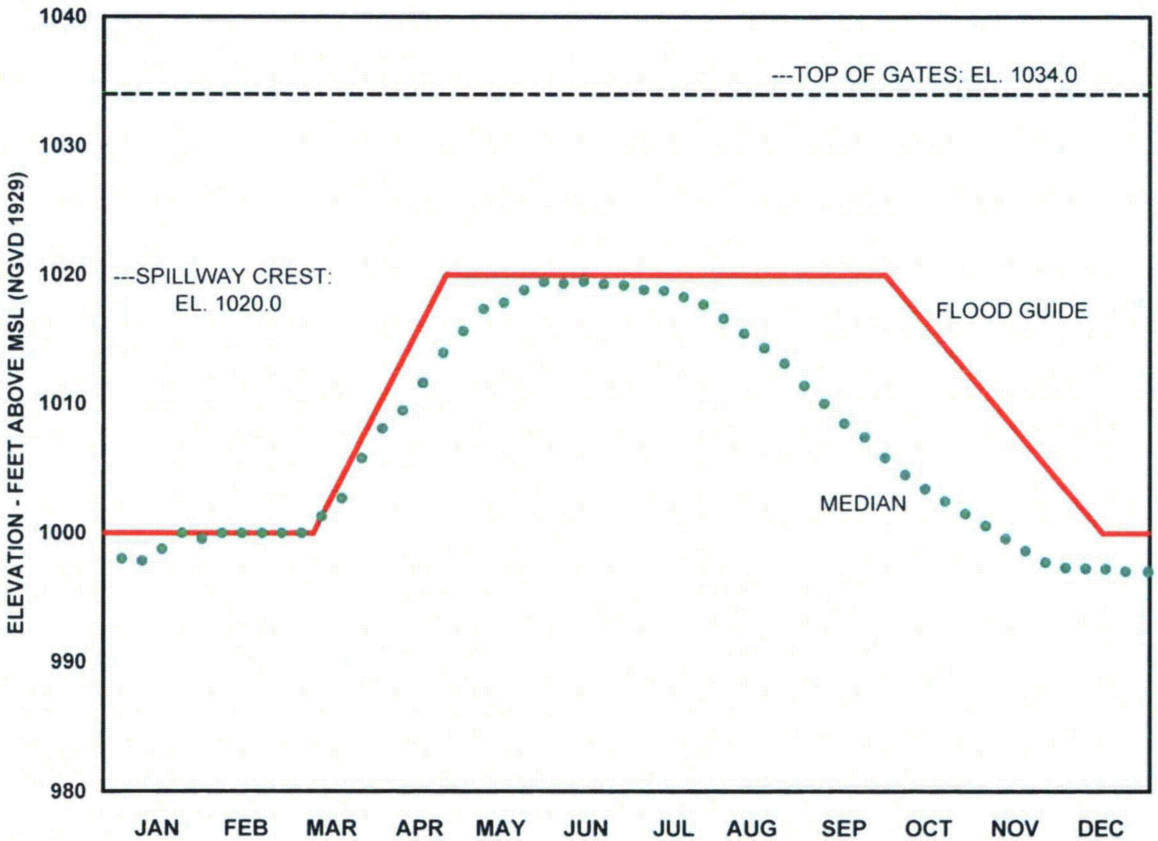
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Fort Patrick Henry Figure 2.4.1-4 (Sheet 8 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Fort Patrick Henry (Sheet 8 of 16)



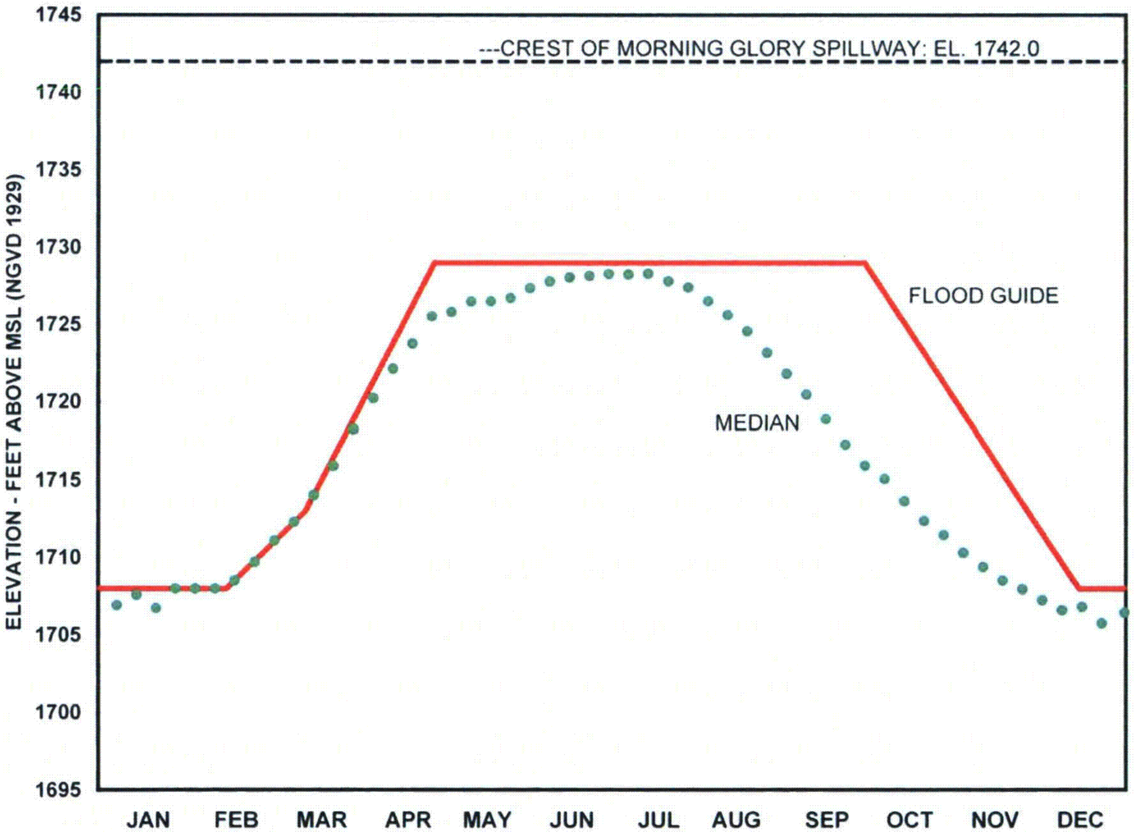
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Melton Hill Figure 2.4.1-4 (Sheet 9 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Melton Hill (Sheet 9 of 16)



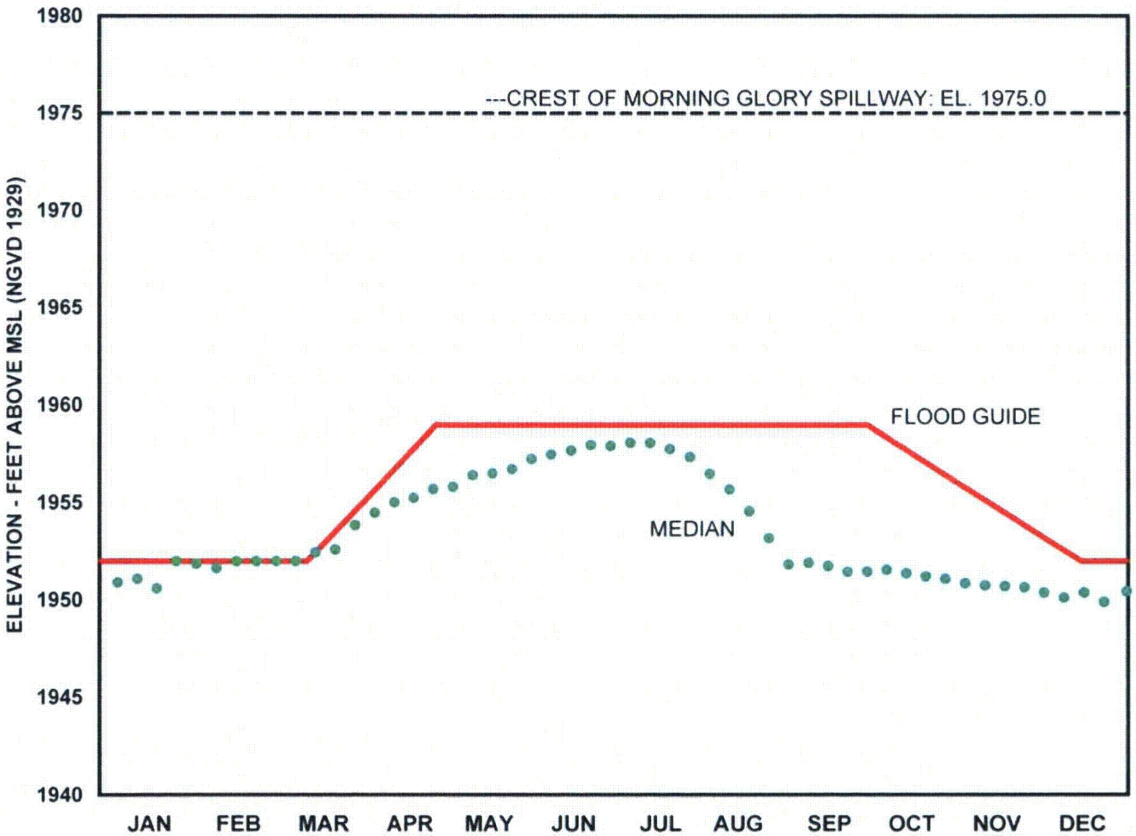
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Norris Figure 2.4.1-4 (Sheet 10 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Norris (Sheet 10 of 16)



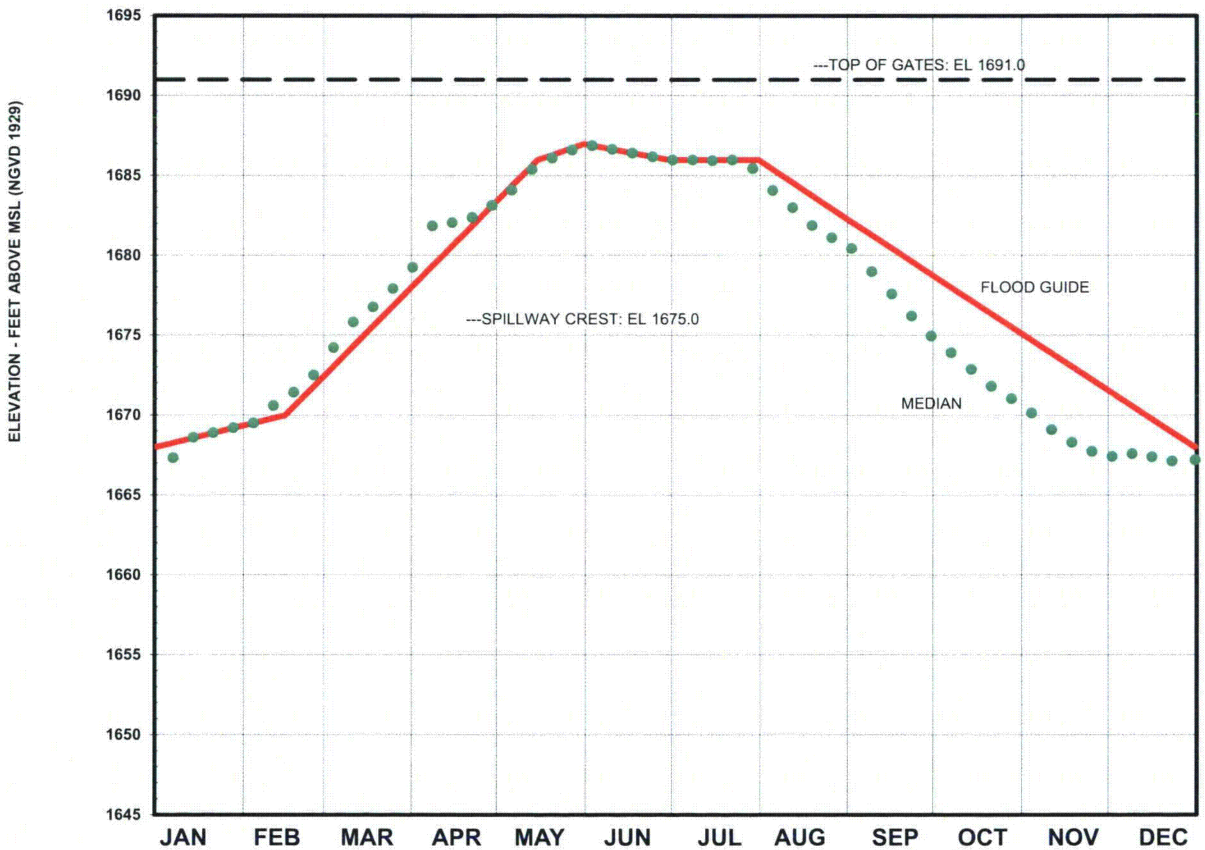
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, South Holston Figure 2.4.1-4 (Sheet 11 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, South Holston (Sheet 11 of 16)



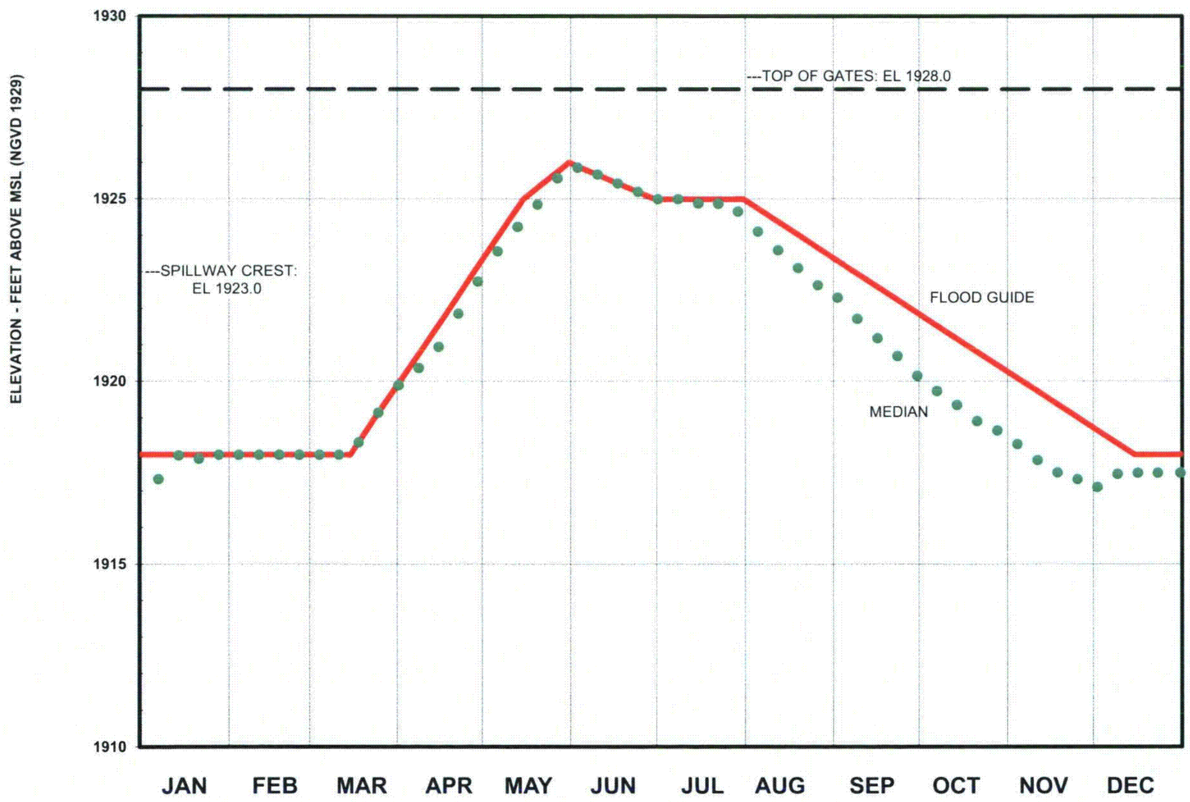
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Watauga Figure 2.4.1-4 (Sheet 12 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Watauga (Sheet 12 of 16)



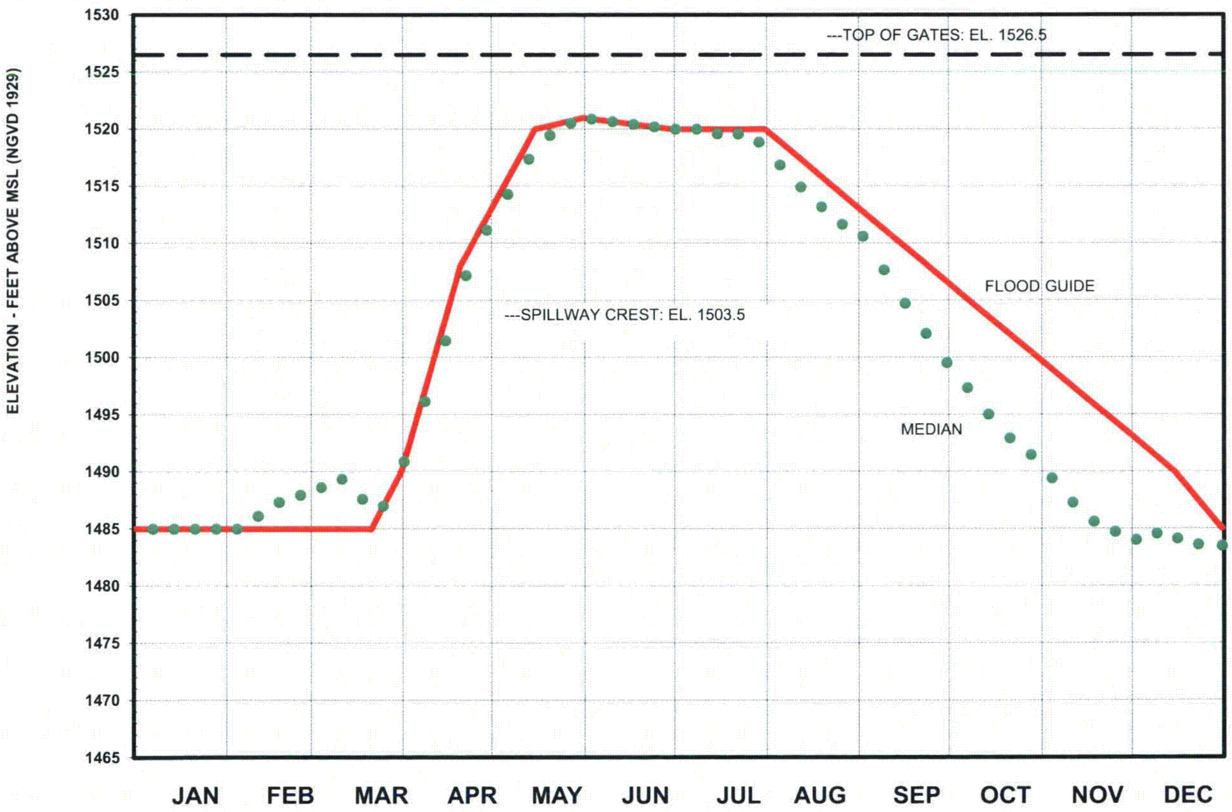
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Blue Ridge Figure 2.4.1-4 (Sheet 13 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Blue Ridge (Sheet 13 of 16)



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Chatuge Figure 2.4.1-4 (Sheet 14 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Chatuge (Sheet 14 of 16)



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Hiwassee Figure 2.4.1-4 (Sheet 15 of 16)

Figure 2.4.1-4 Seasonal Operating Curve, Hiwassee (Sheet 15 of 16)

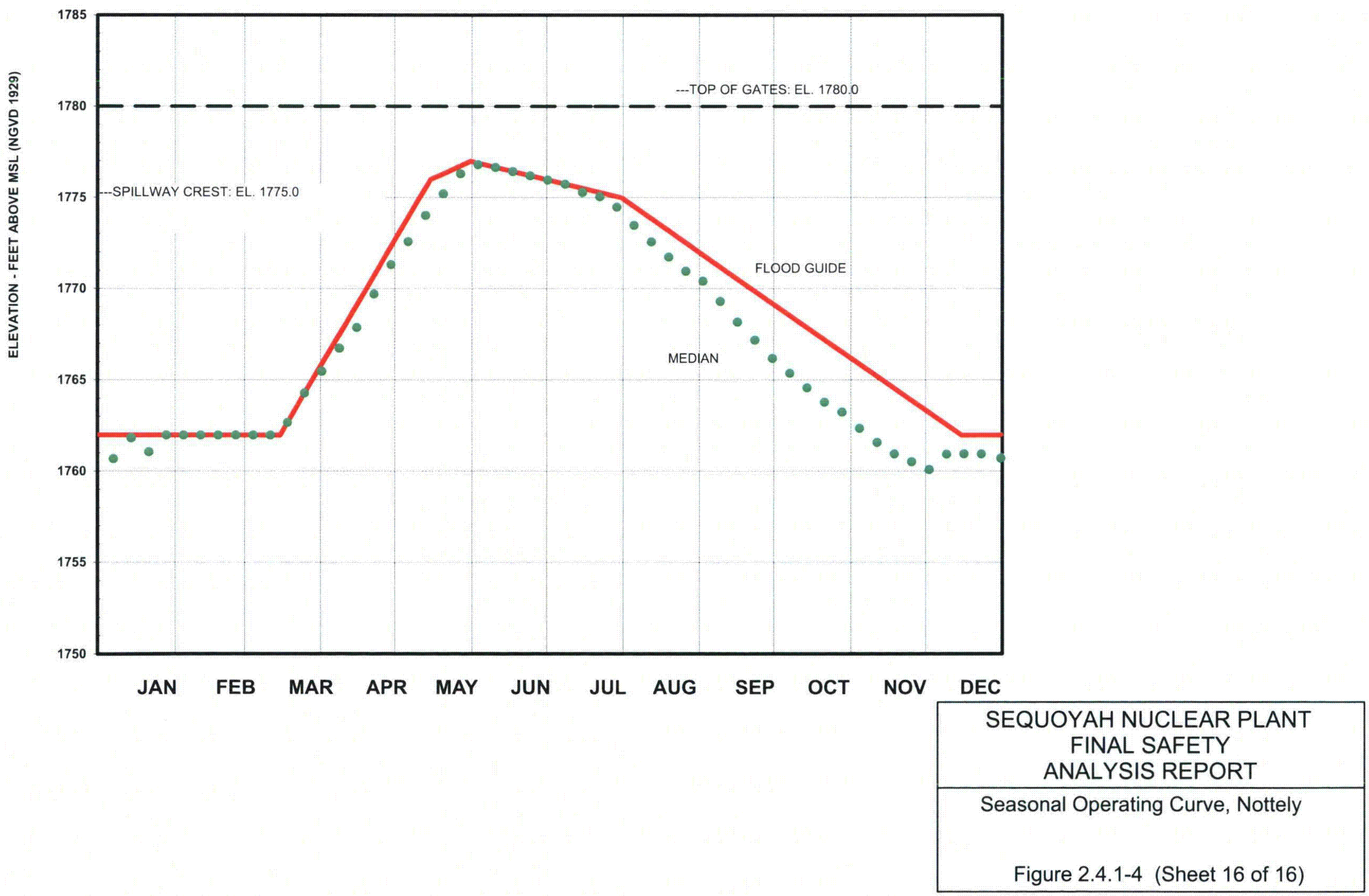
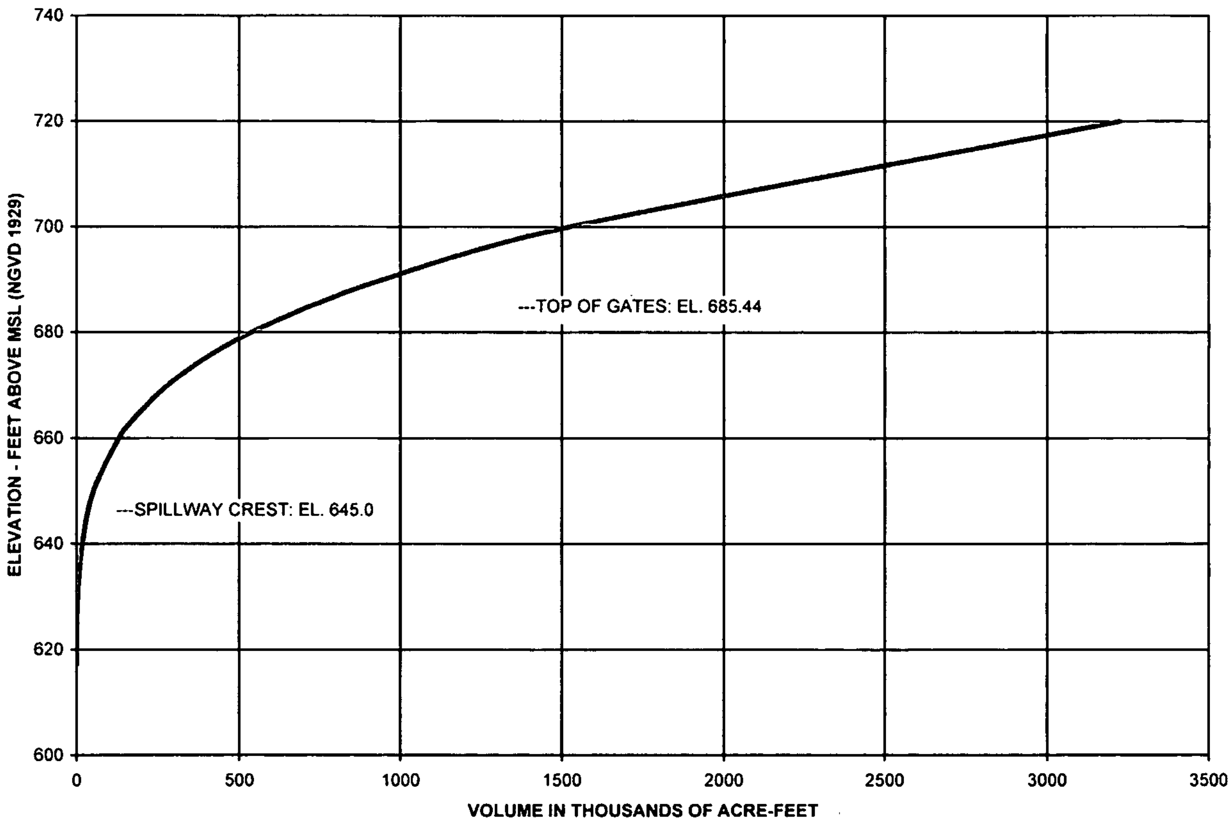
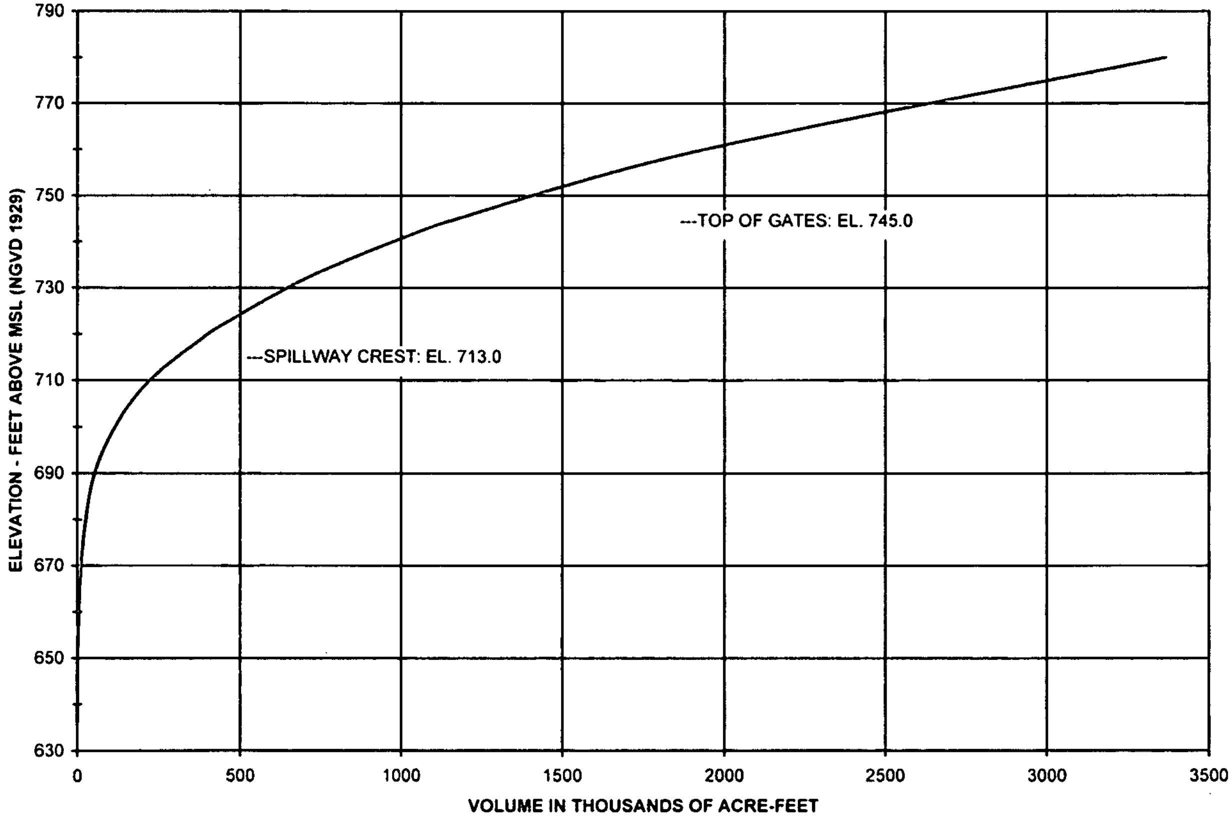


Figure 2.4.1-4 Seasonal Operating Curve, Nottely (Sheet 16 of 16)



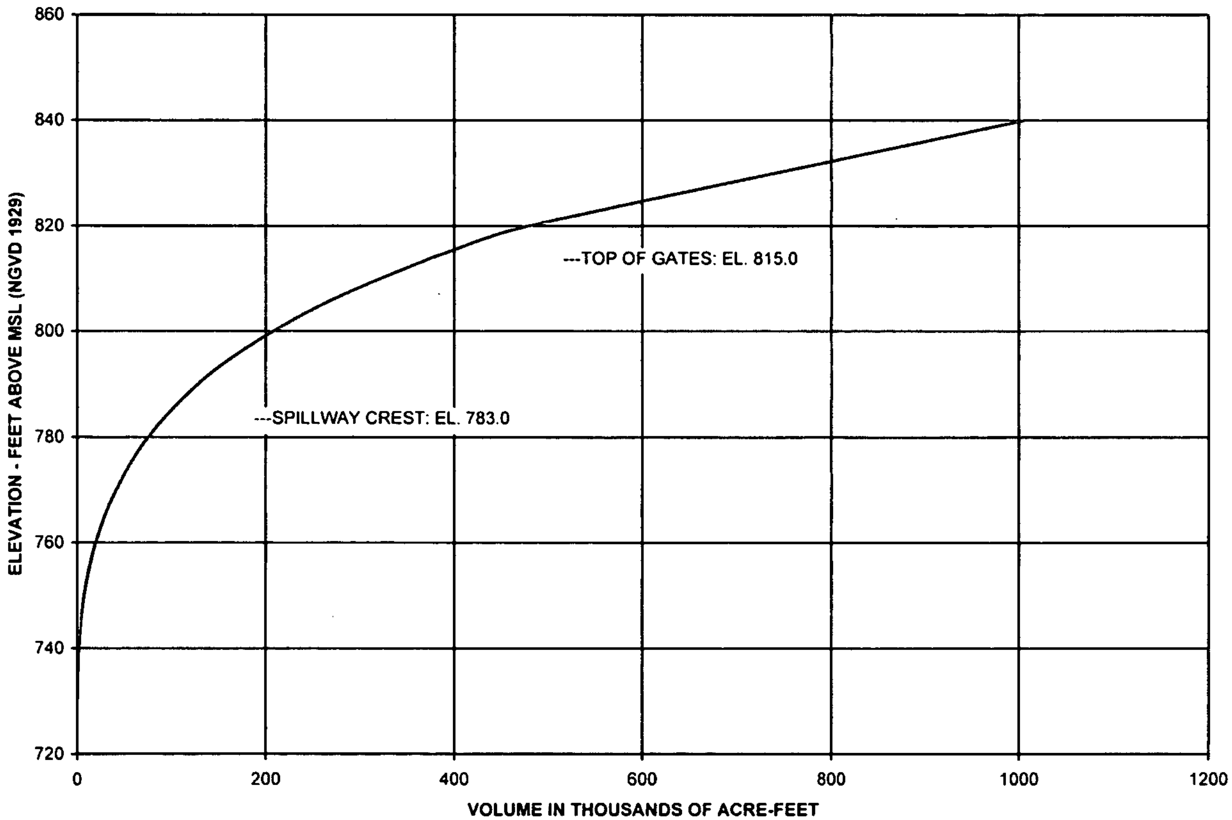
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Chickamauga Figure 2.4.1-5 (Sheet 1 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Chickamauga (Sheet 1 of 17)



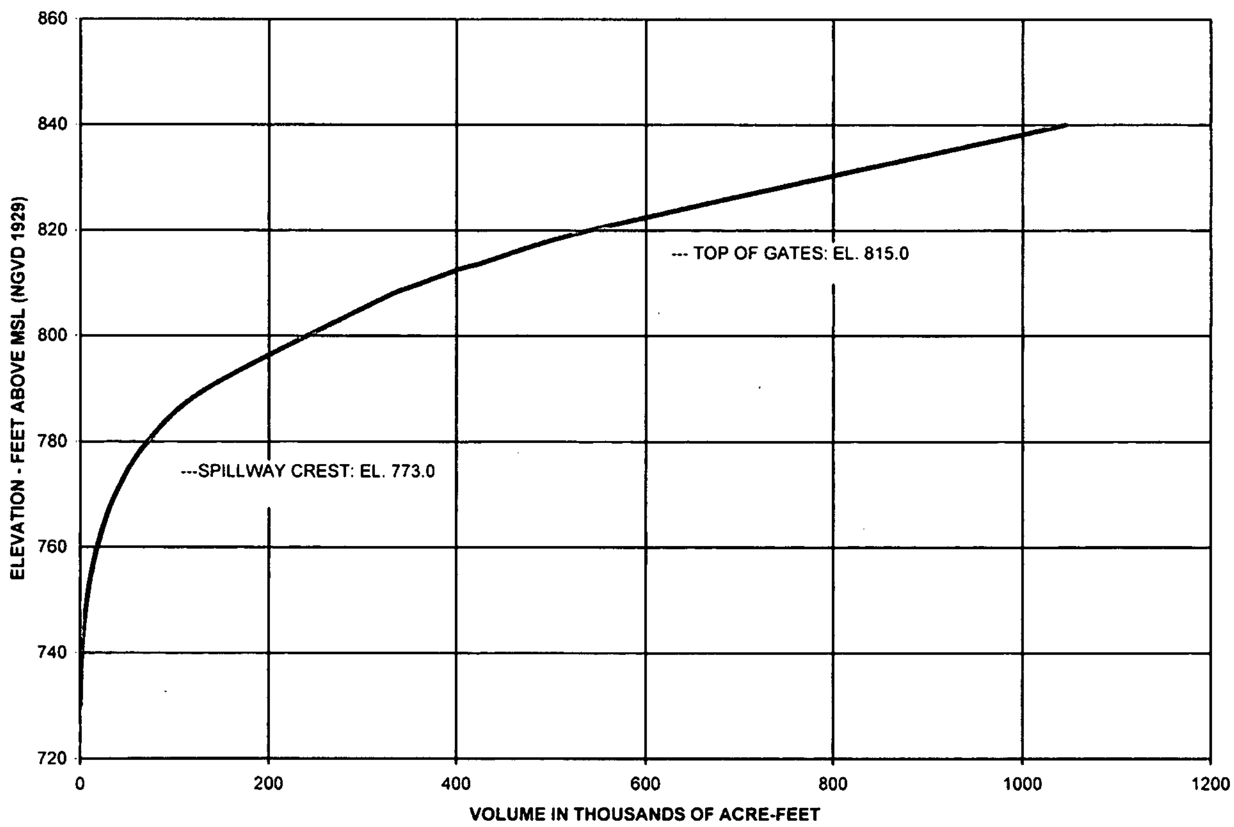
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Watts Bar Figure 2.4.1-5 (Sheet 2 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Watts Bar (Sheet 2 of 17)



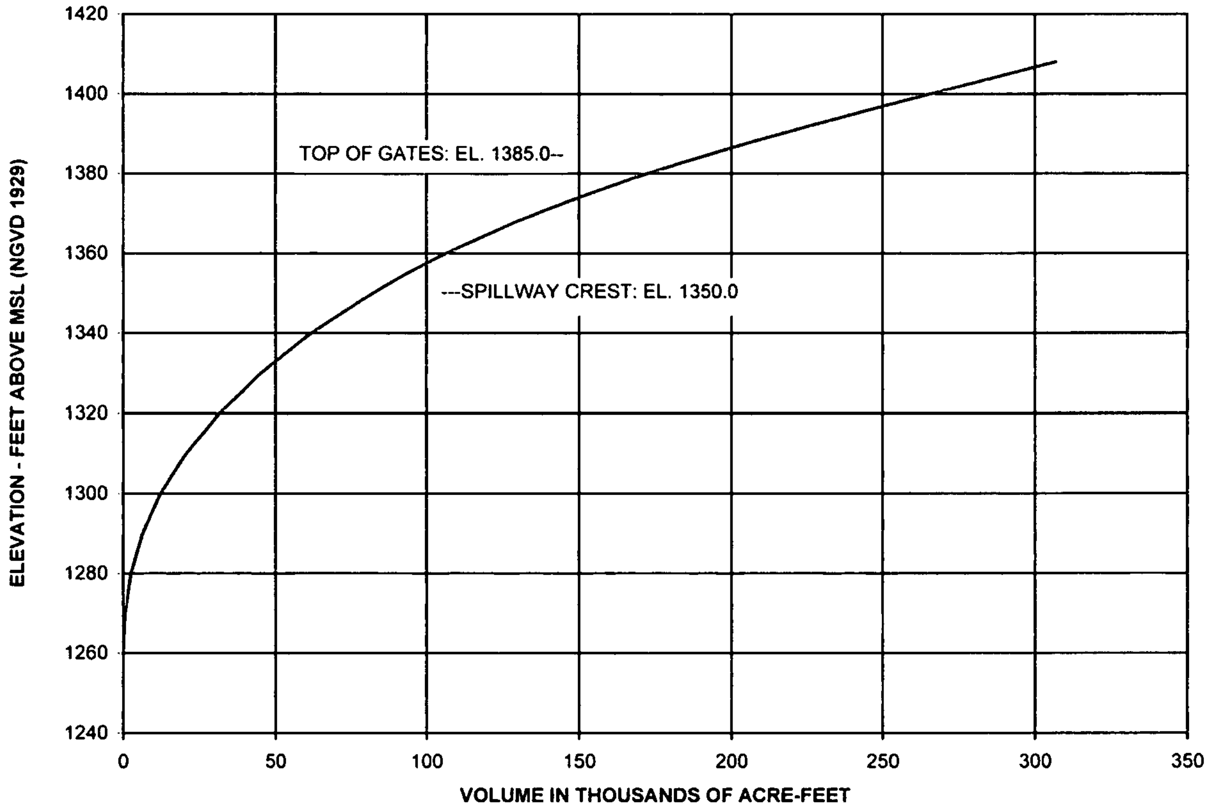
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Fort Loudoun Figure 2.4.1-5 (Sheet 3 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Fort Loudoun (Sheet 3 of 17)



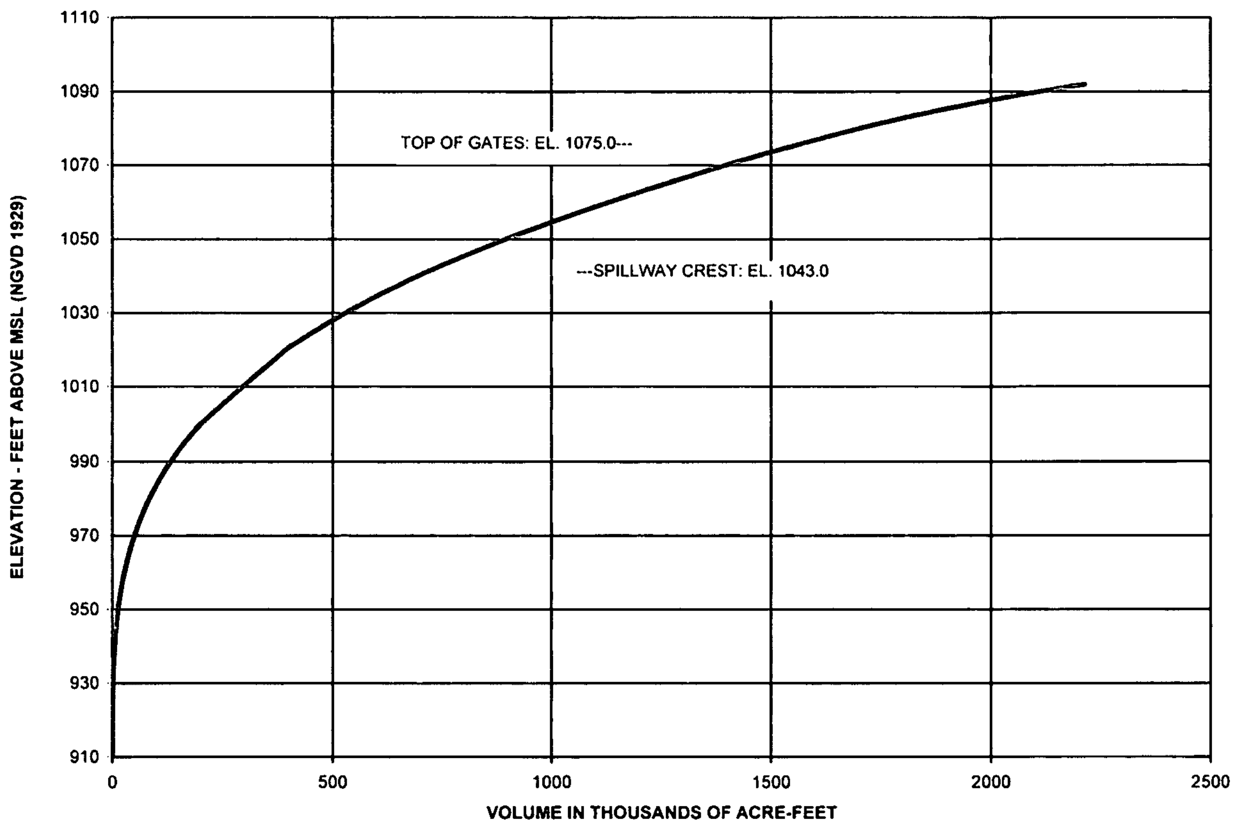
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Tellico Figure 2.4.1-5 (Sheet 4 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Tellico (Sheet 4 of 17)



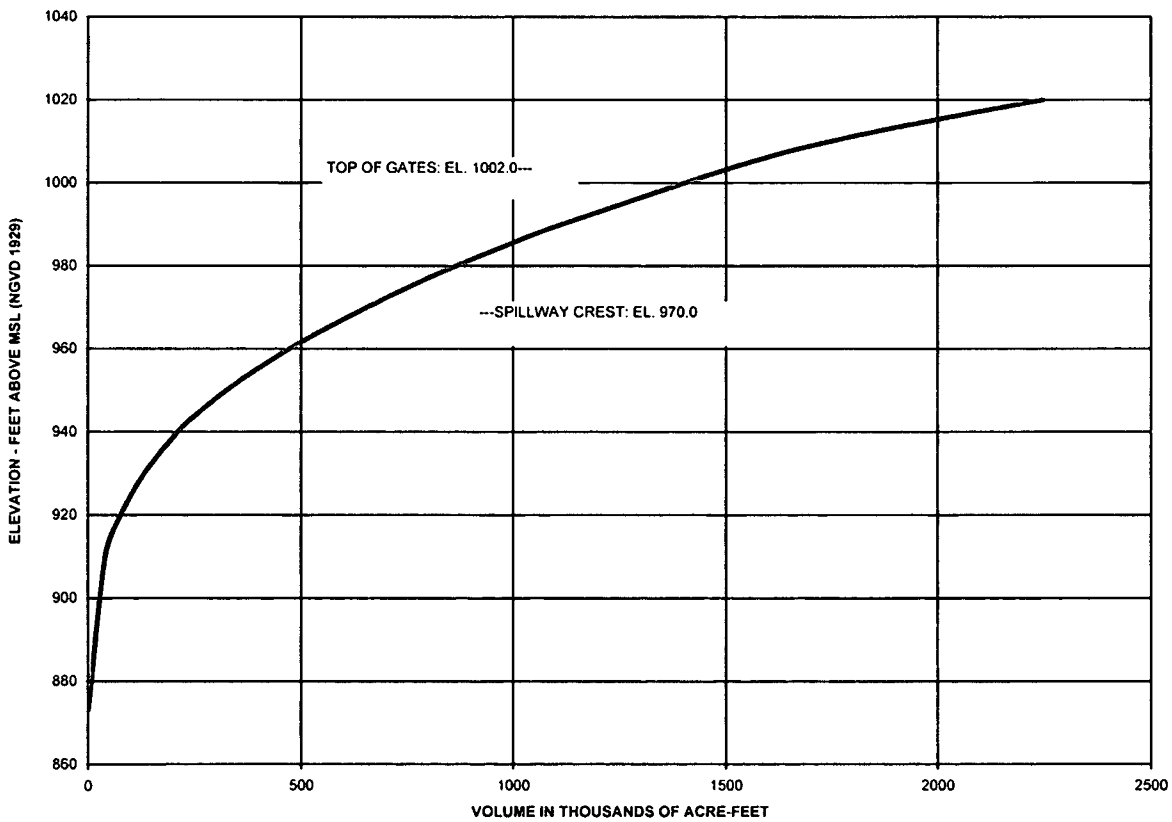
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Boone Figure 2.4.1-5 (Sheet 5 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Boone (Sheet 5 of 17)



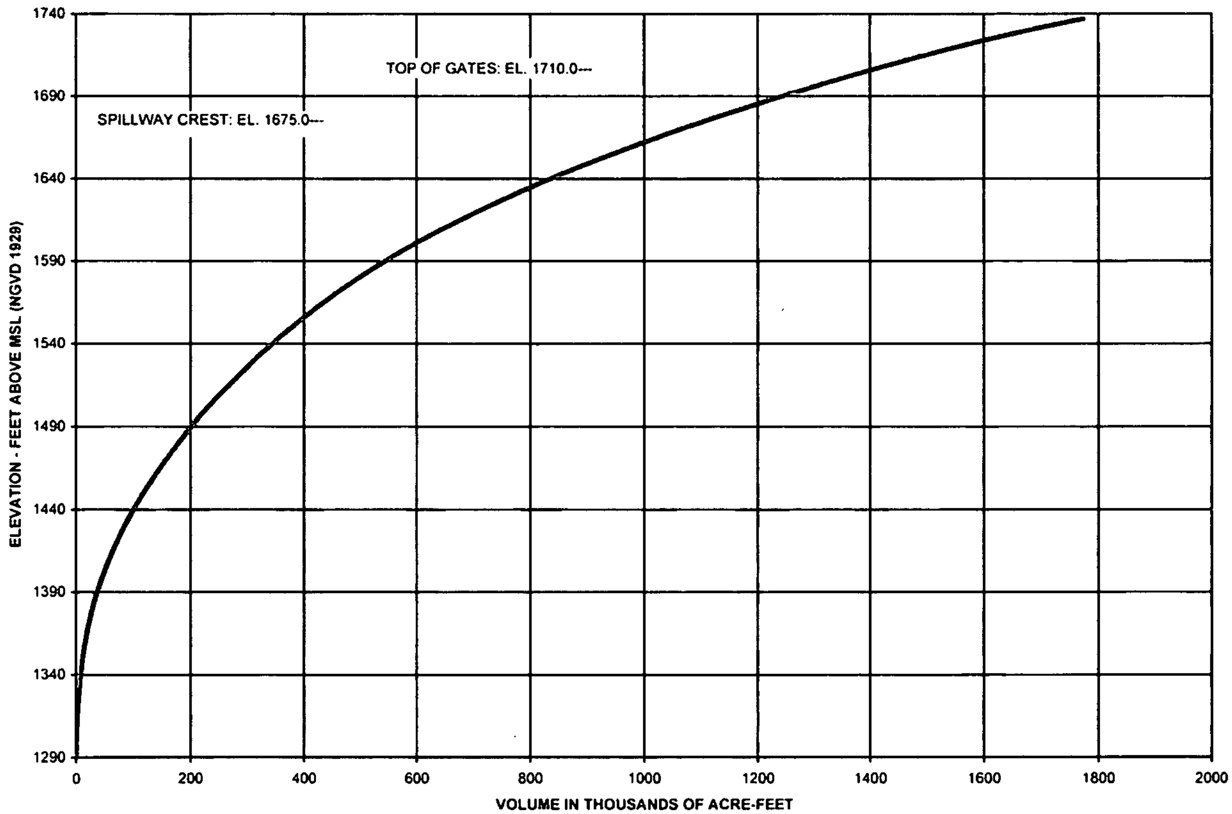
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Cherokee Figure 2.4.1-5 (Sheet 6 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Cherokee (Sheet 6 of 17)



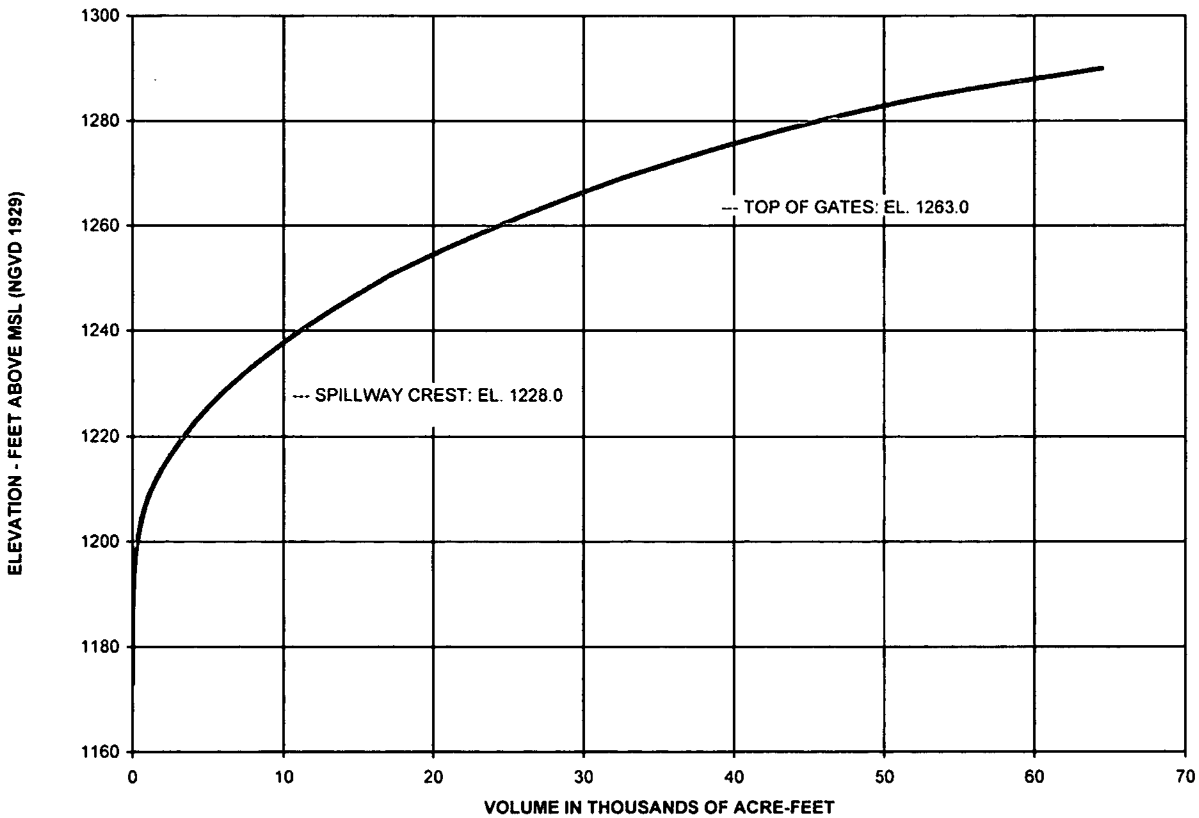
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Douglas Figure 2.4.1-5 (Sheet 7 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Douglas (Sheet 7 of 17)



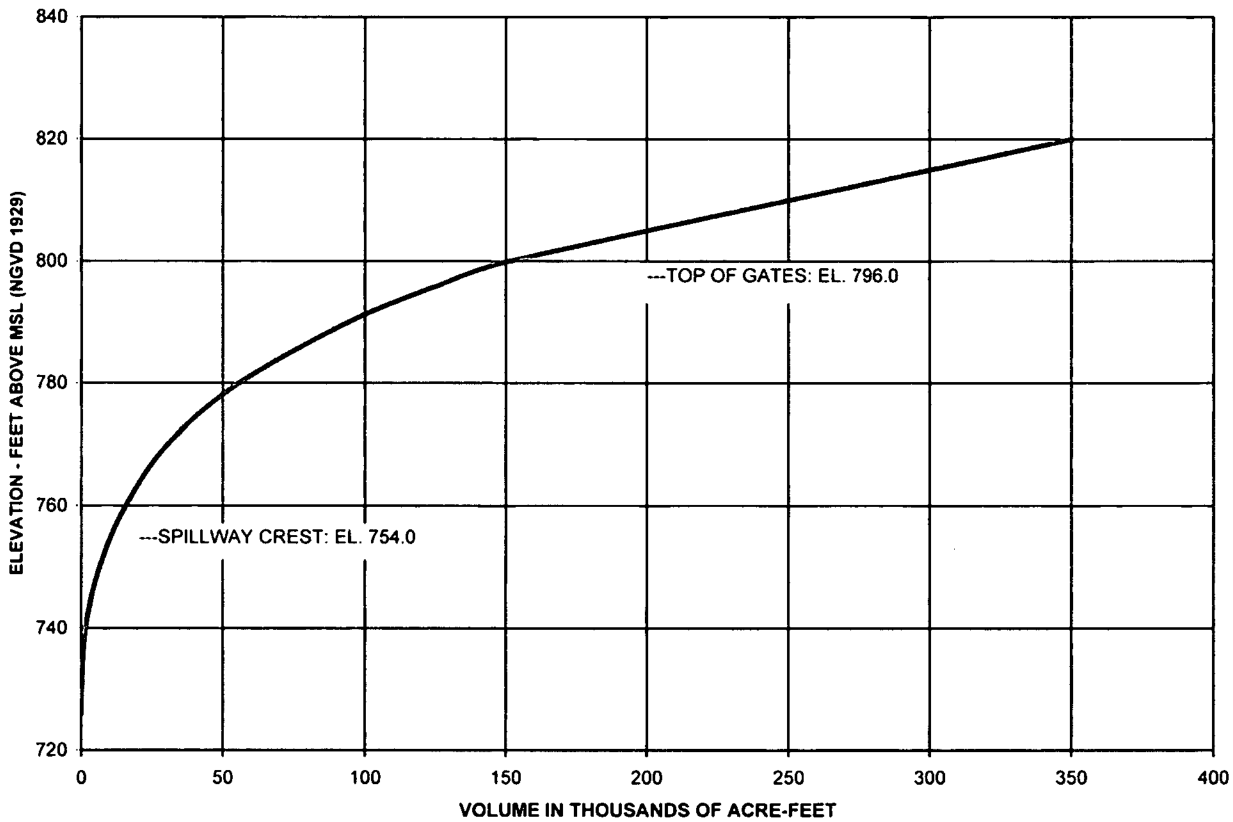
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Fontana Figure 2.4.1-5 (Sheet 8 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Fontana (Sheet 8 of 17)



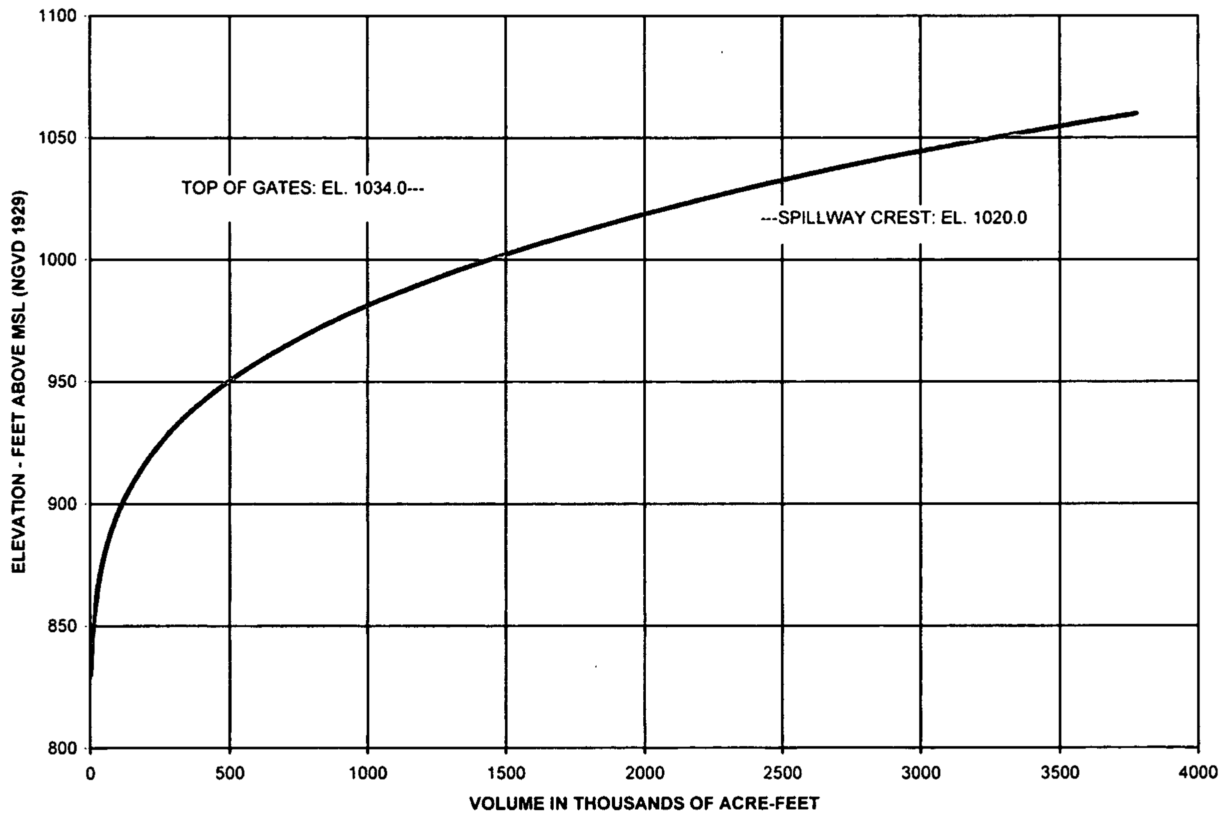
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Fort Patrick Henry Figure 2.4.1-5 (Sheet 9 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Fort Patrick Henry (Sheet 9 of 17)



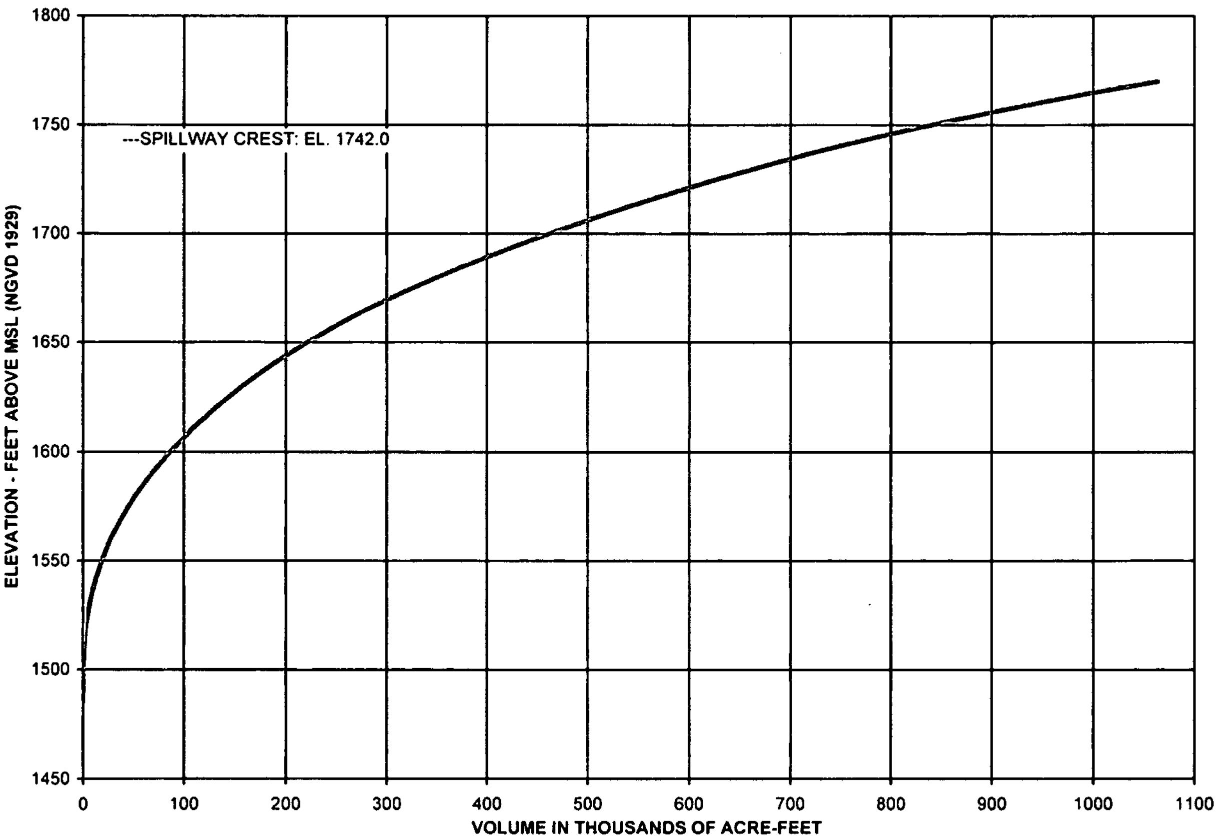
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Melton Hill Figure 2.4.1-5 (Sheet 10 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Melton Hill (Sheet 10 of 17)



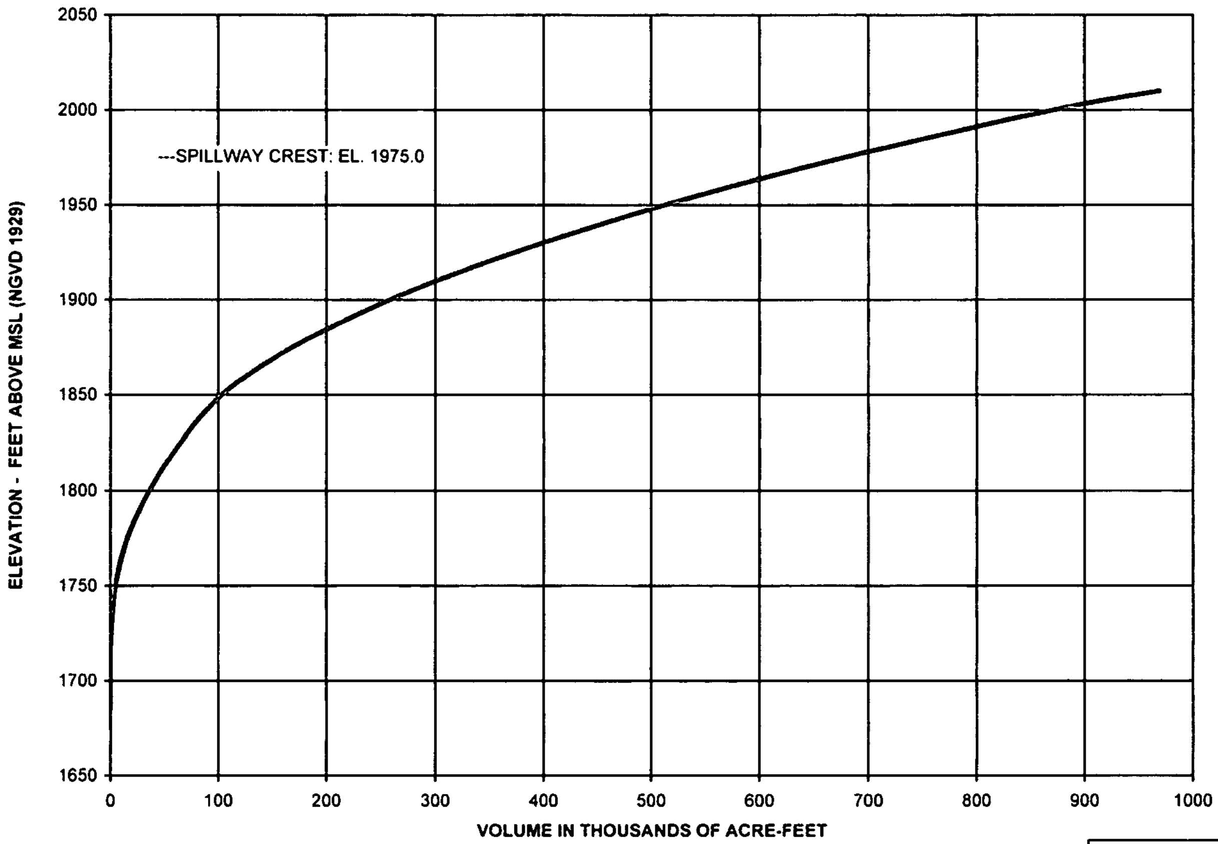
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Norris Figure 2.4.1-5 (Sheet 11 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Norris (Sheet 11 of 17)



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, South Holston Figure 2.4.1-5 (Sheet 12 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, South Holston (Sheet 12 of 17)

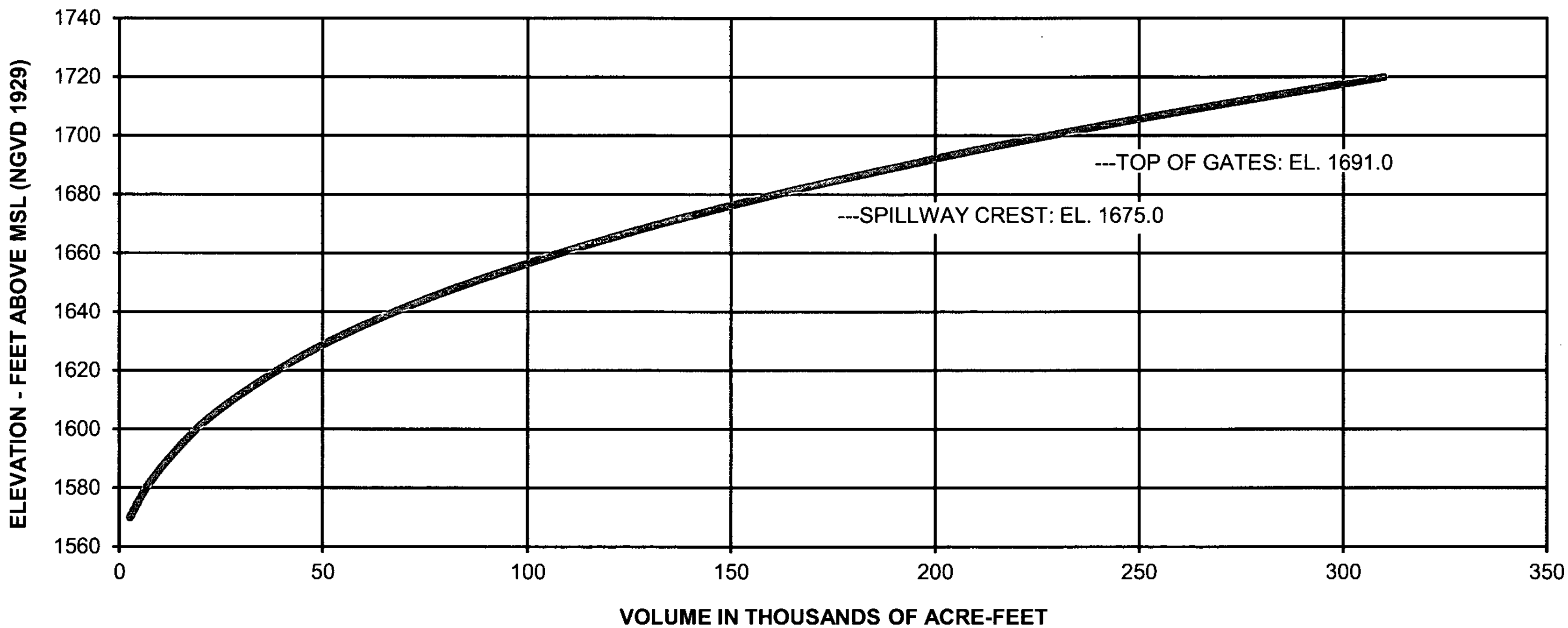


SEQUOYAH NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Reservoir Elevation - Storage
Relationship, Watauga

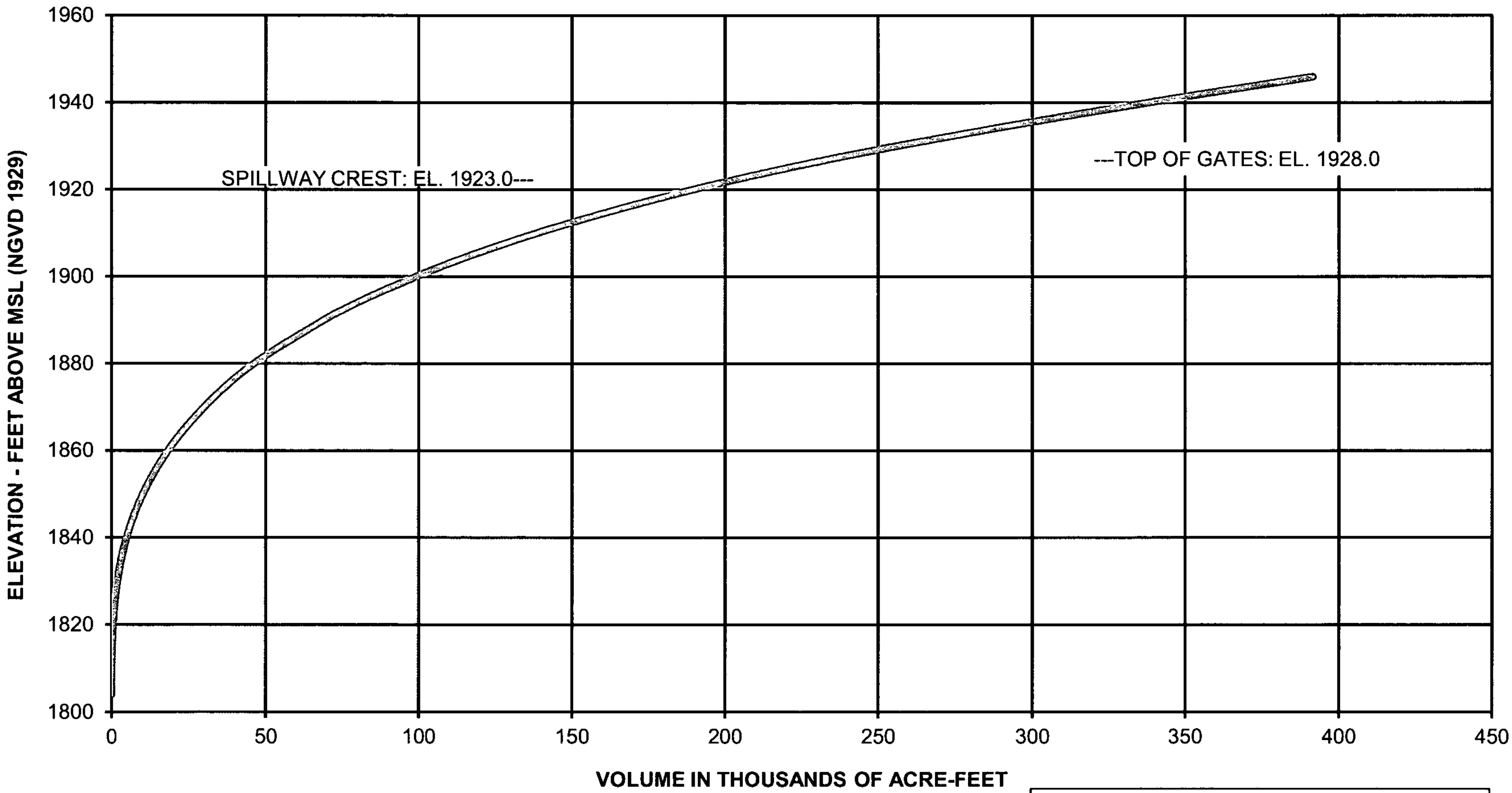
Figure 2.4.1-5 (Sheet 13 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Watauga (Sheet 13 of 17)



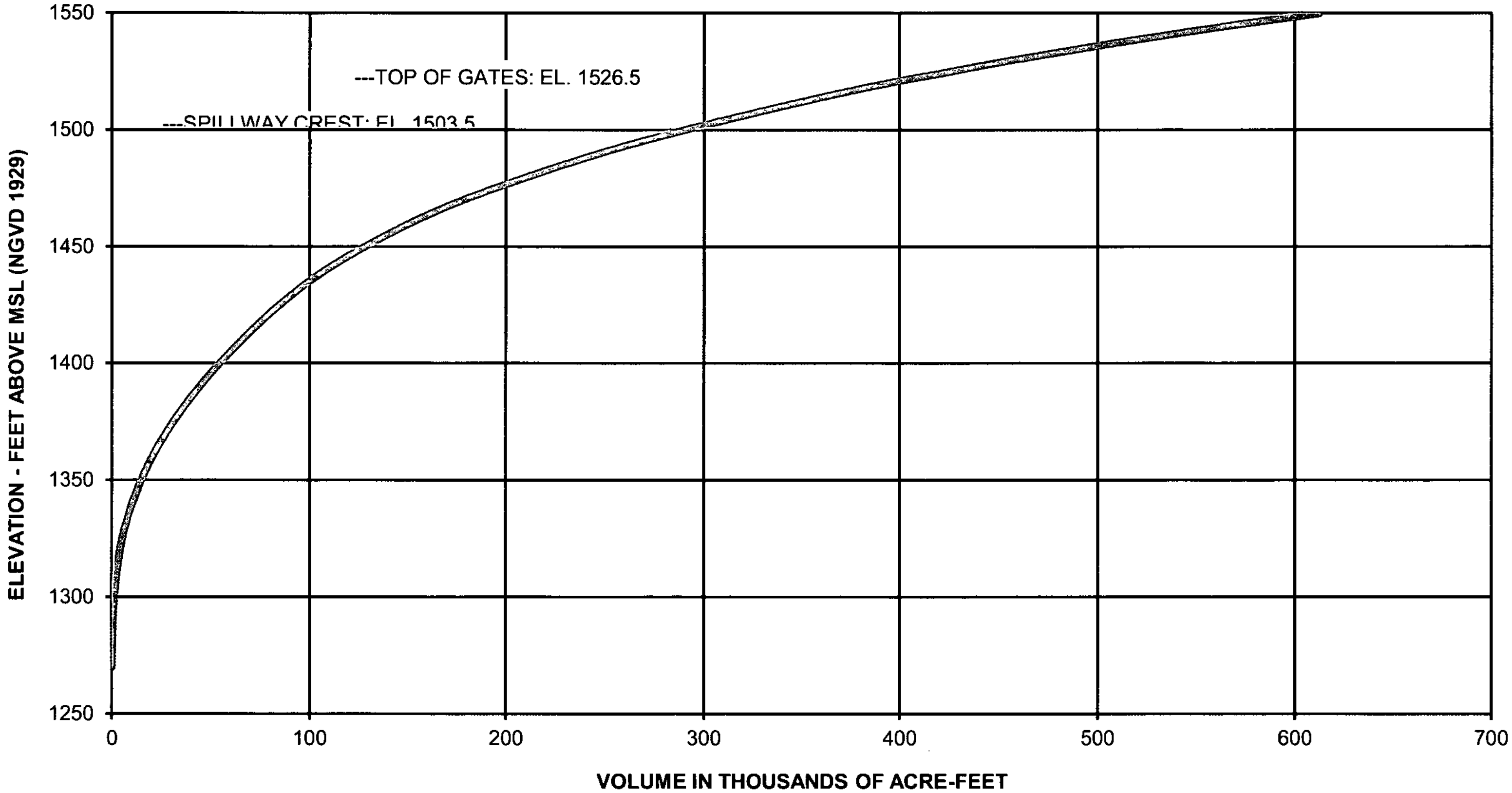
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Blue Ridge Figure 2.4.1-5 (Sheet 14 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Blue Ridge (Sheet 14 of 17)



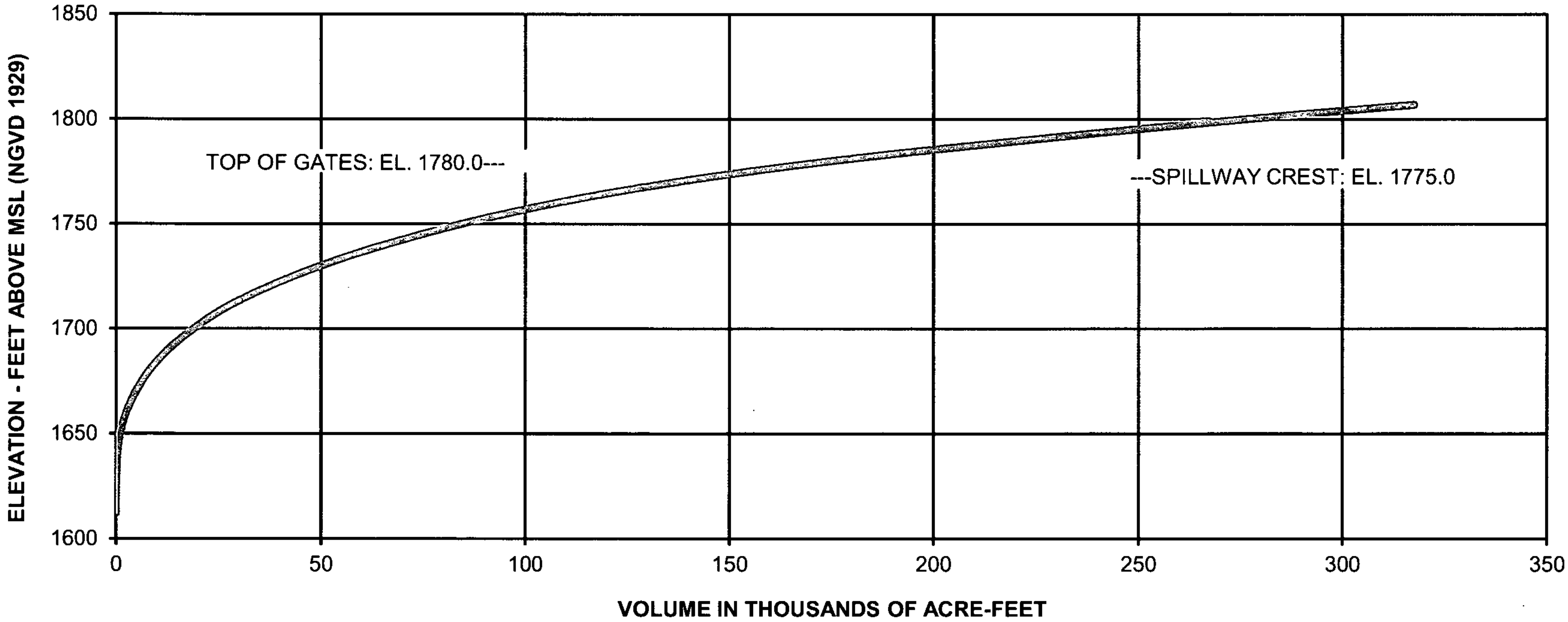
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Chatuge Figure 2.4.1-5 (Sheet 15 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Chatuge (Sheet 15 of 17)



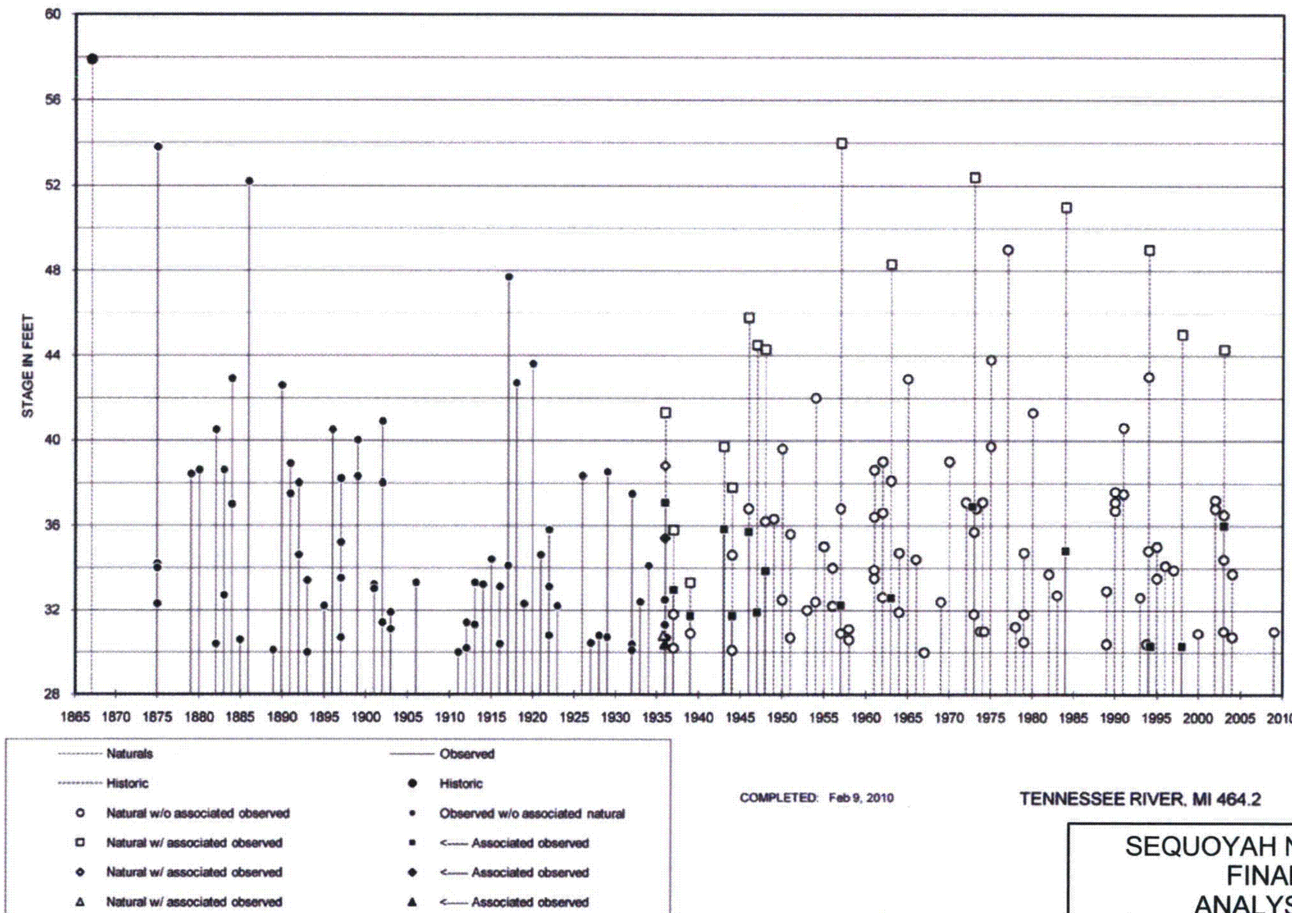
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Hiwassee Figure 2.4.1-5 (Sheet 16 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Hiwassee (Sheet 16 of 17)



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Nottely Figure 2.4.1-5 (Sheet 17 of 17)

Figure 2.4.1-5 Reservoir Elevation - Storage Relationship, Nottely (Sheet 17 of 17)

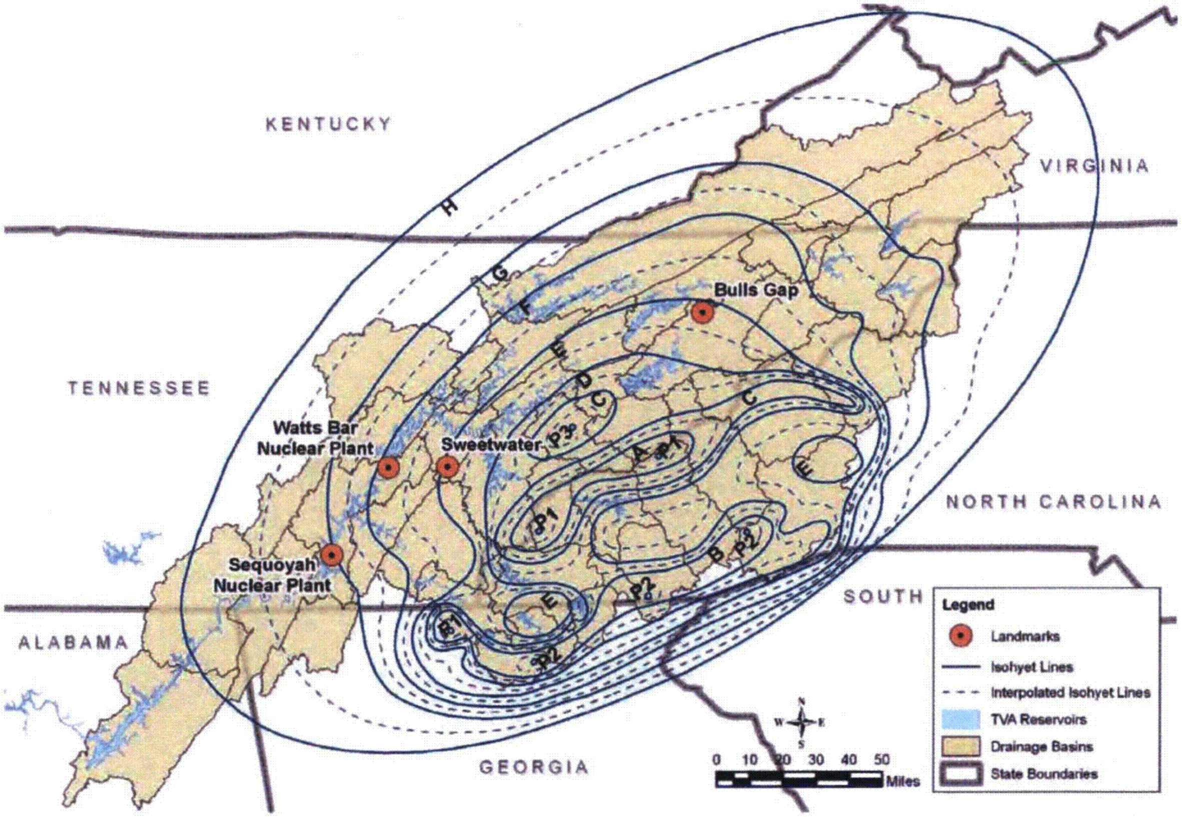


SEQUOYAH NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Tennessee River Mile 464.2 -
Distribution of Floods at
Chattanooga, Tennessee

Figure 2.4.2-1

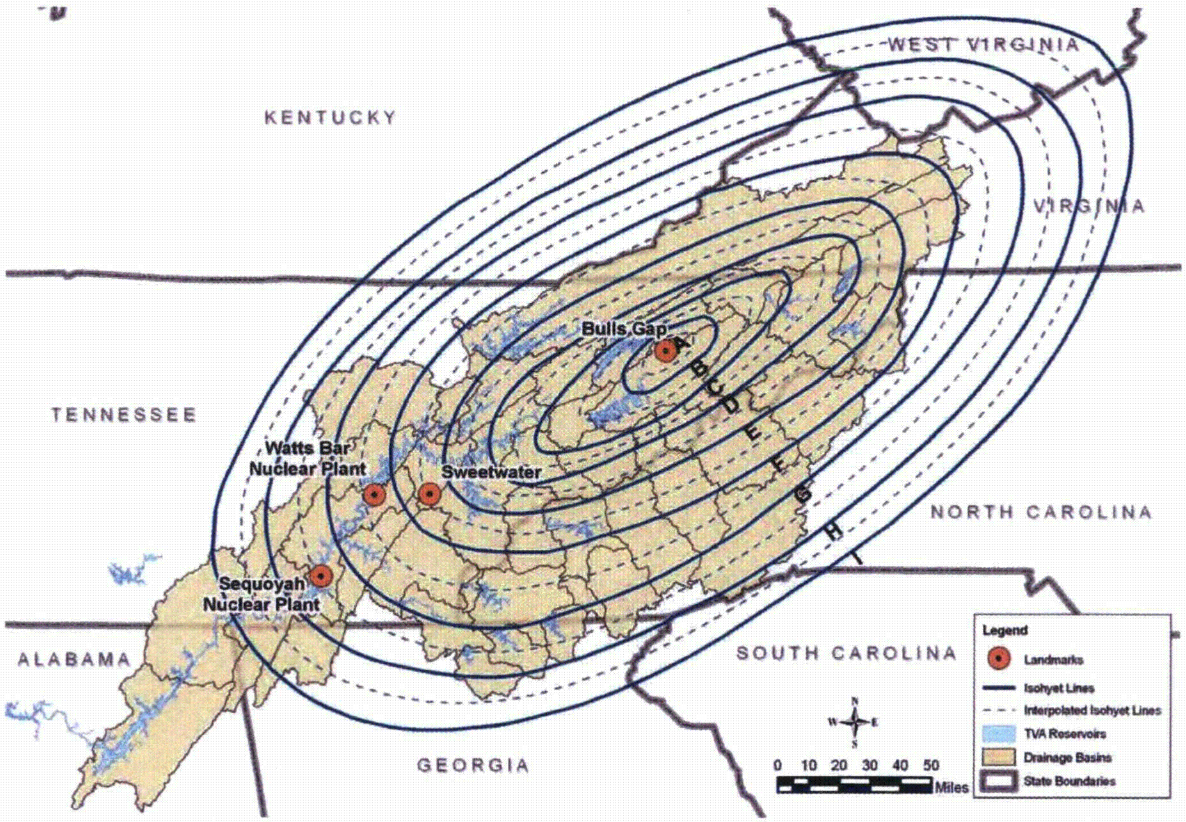
Figure 2.4.2-1 Tennessee River Mile 464.2 - Distribution of Floods at Chattanooga, Tennessee



SEQUOYAH NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

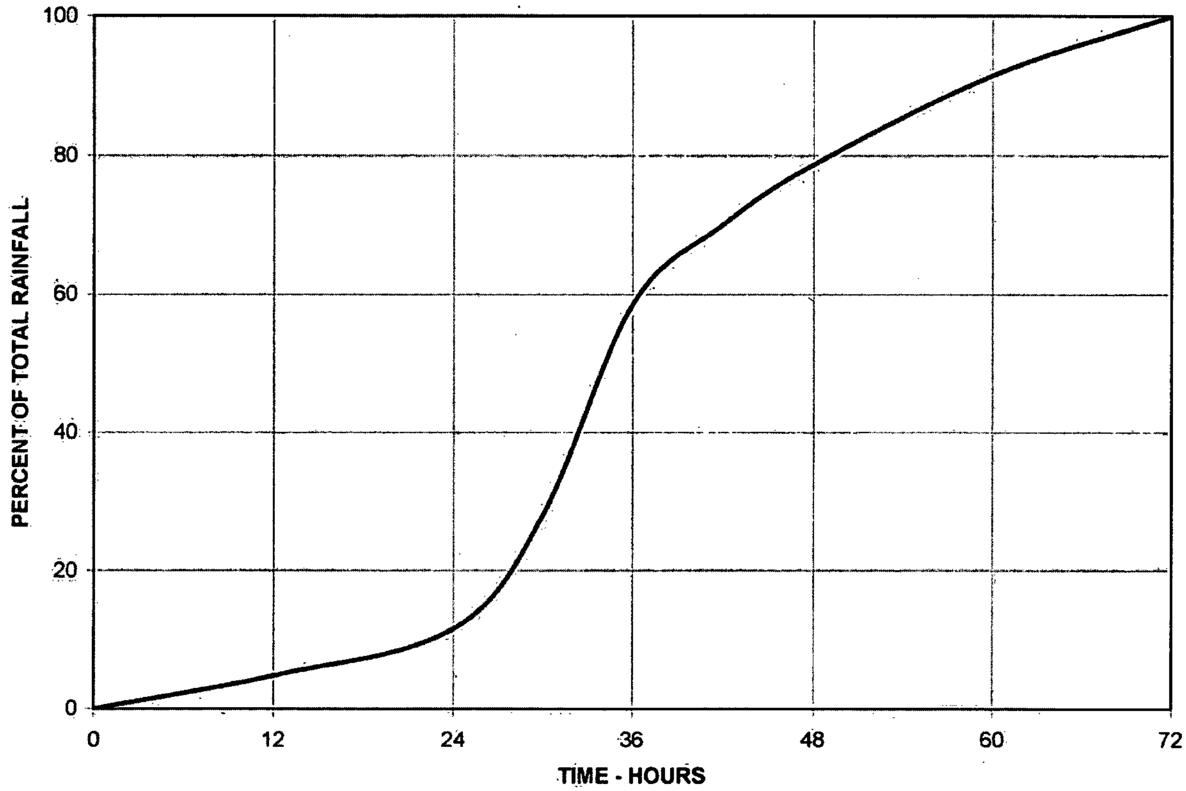
Probable Maximum Precipitation
Isohyets for 21,400 Sq. Mi. Event,
Downstream Placement
Figure 2.4.3-1

Figure 2.4.3-1 Probable Maximum Precipitation Isohyets for 21,400 Sq. Mi. Event, Downstream Placement



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Probable Maximum Precipitation Isohyets for 7980 Sq. Mi. Event, Centered at Bulls Gap, TN Figure 2.4.3-2

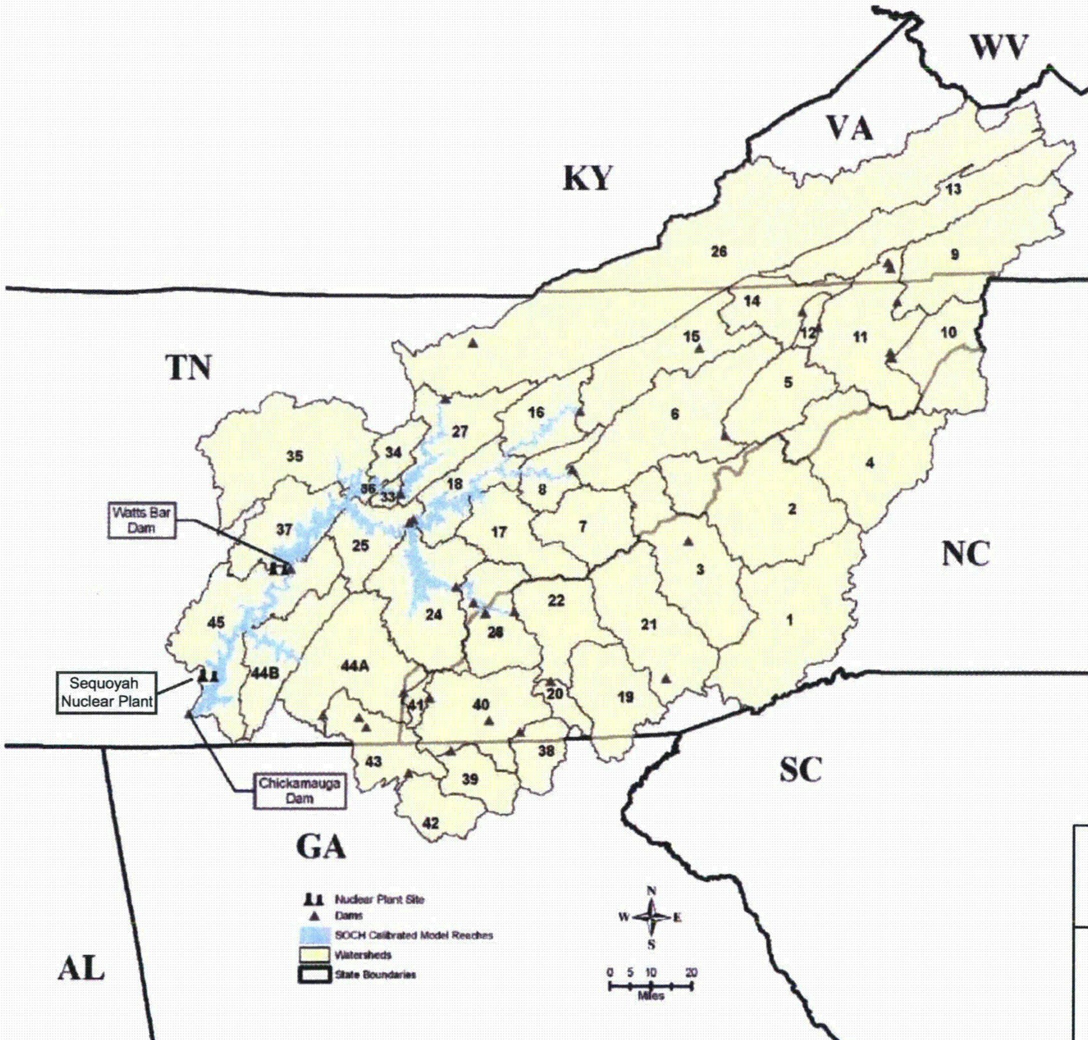
Figure 2.4.3-2 Probable Maximum Precipitation Isohyets for 7980 Sq. Mi. Event, Centered at Bulls Gap, TN



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Rainfall Time Distribution - Typical Mass Curve Figure 2.4.3-3

Figure 2.4.3-3 Rainfall Time Distribution - Typical Mass Curve

Figure 2.4.3-4 Not Used

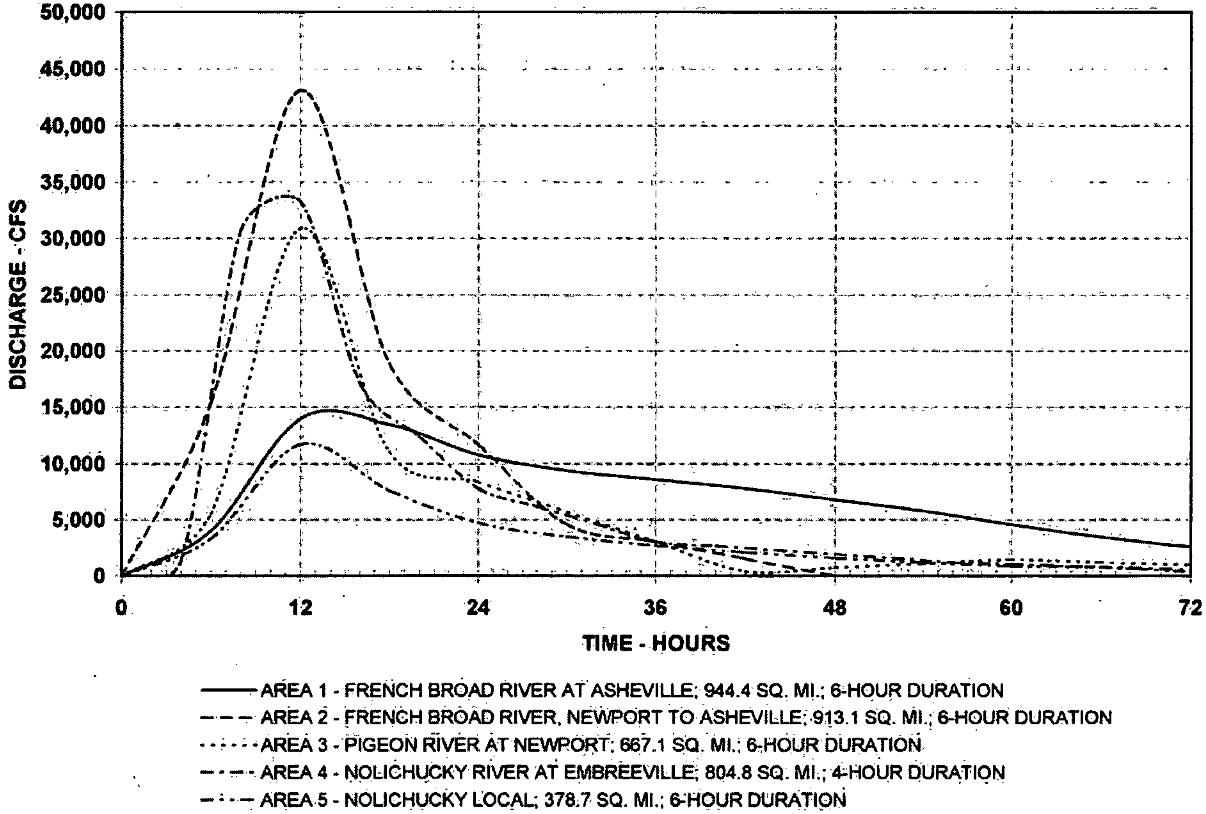


SEQUOYAH NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Drainage Areas above Chickamauga Dam

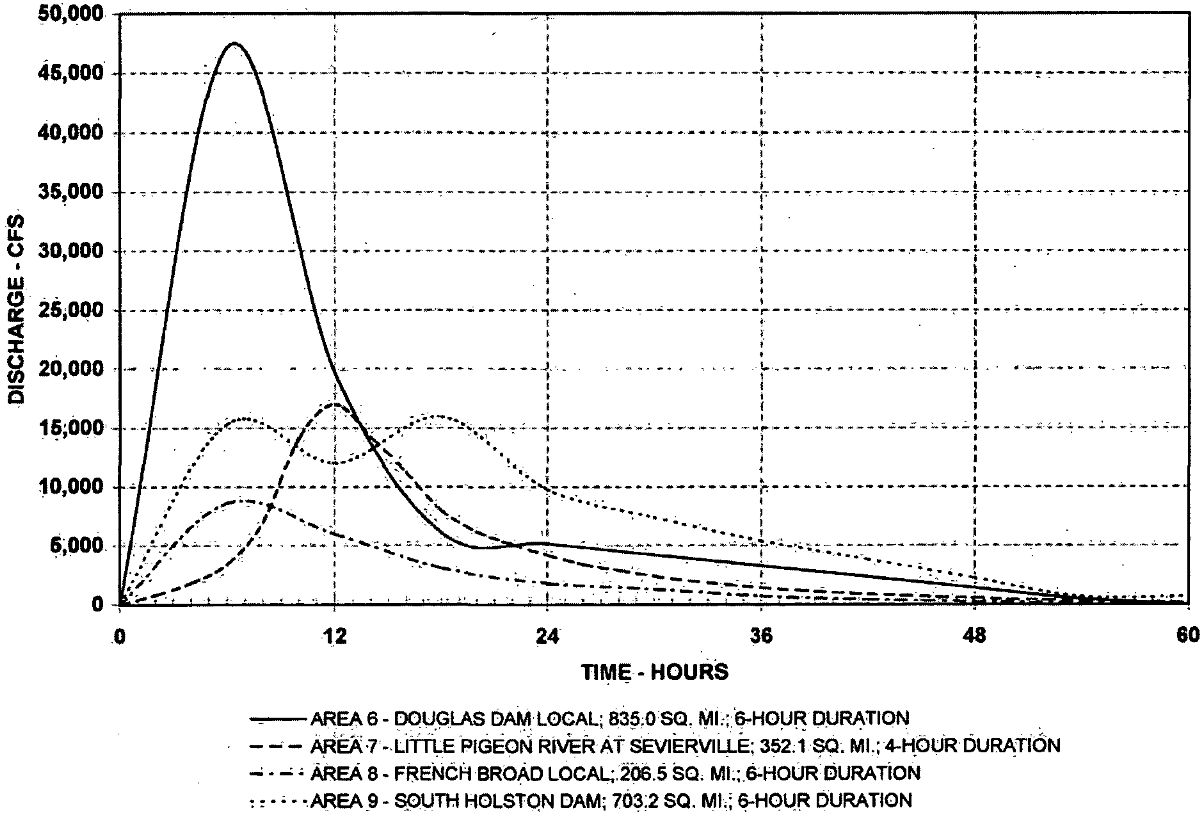
Figure 2.4.3-5

Figure 2.4.3-5 Drainage Areas above Chickamauga Dam



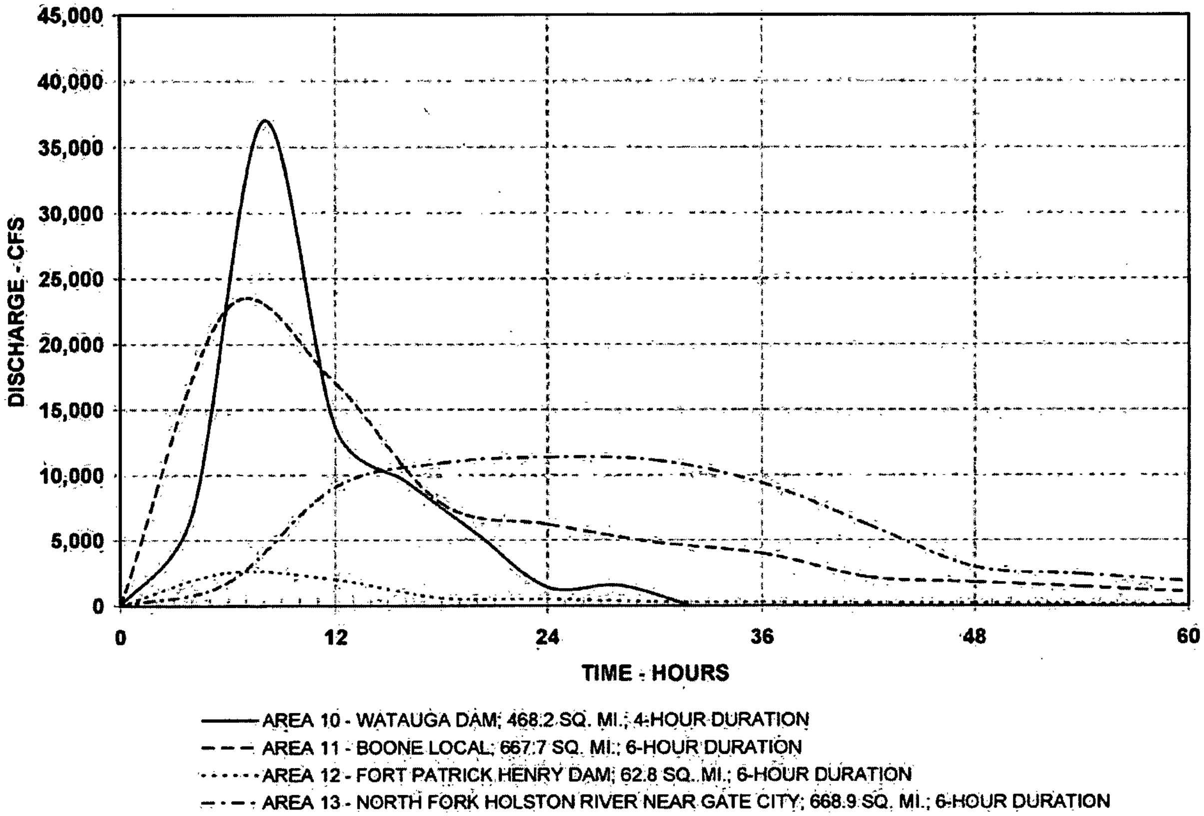
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 1-5
Figure 2.4.3-6 (Sheet 1 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 1–5 (Sheet 1 of 11)



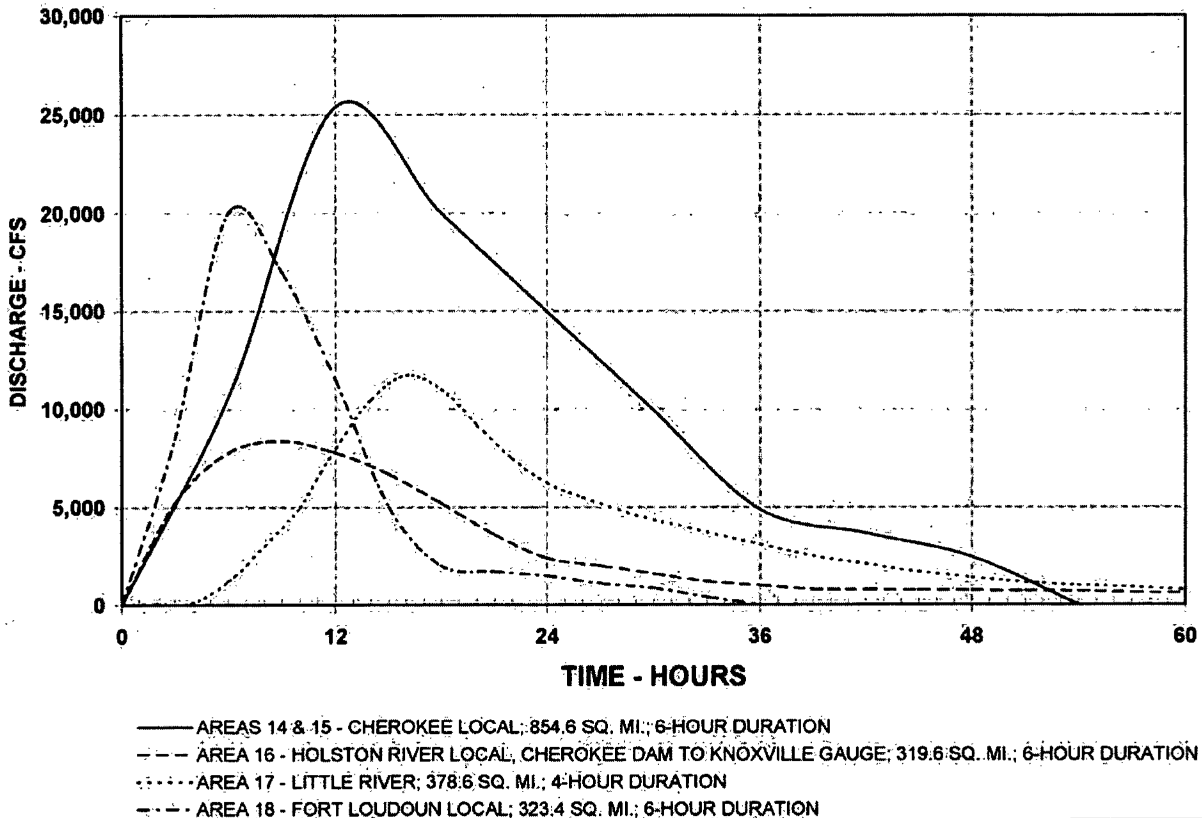
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 6-9
Figure 2.4.3-6 (Sheet 2 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 6–9 (Sheet 2 of 11)



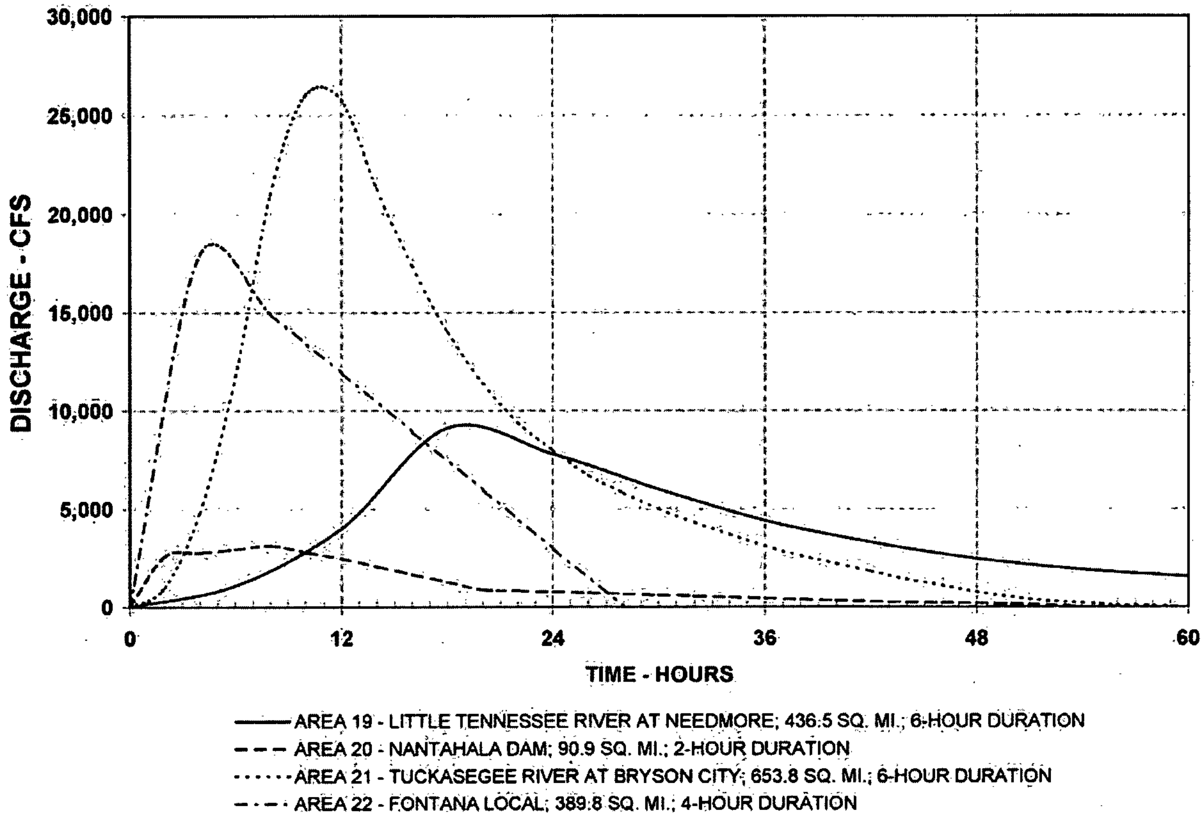
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 10-13 Figure 2.4.3-6 (Sheet 3 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 10–13 (Sheet 3 of 11)



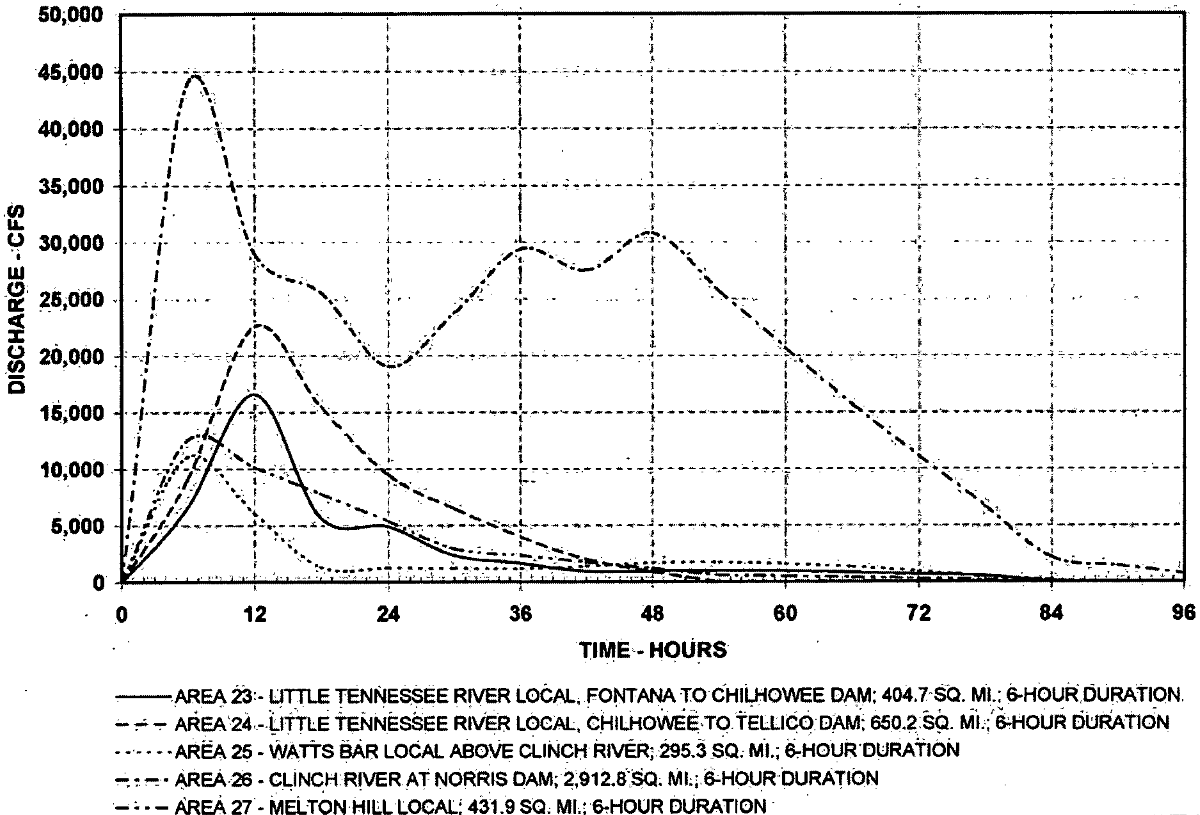
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 14-18
Figure 2.4.3-6 (Sheet 4 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 14–18 (Sheet 4 of 11)



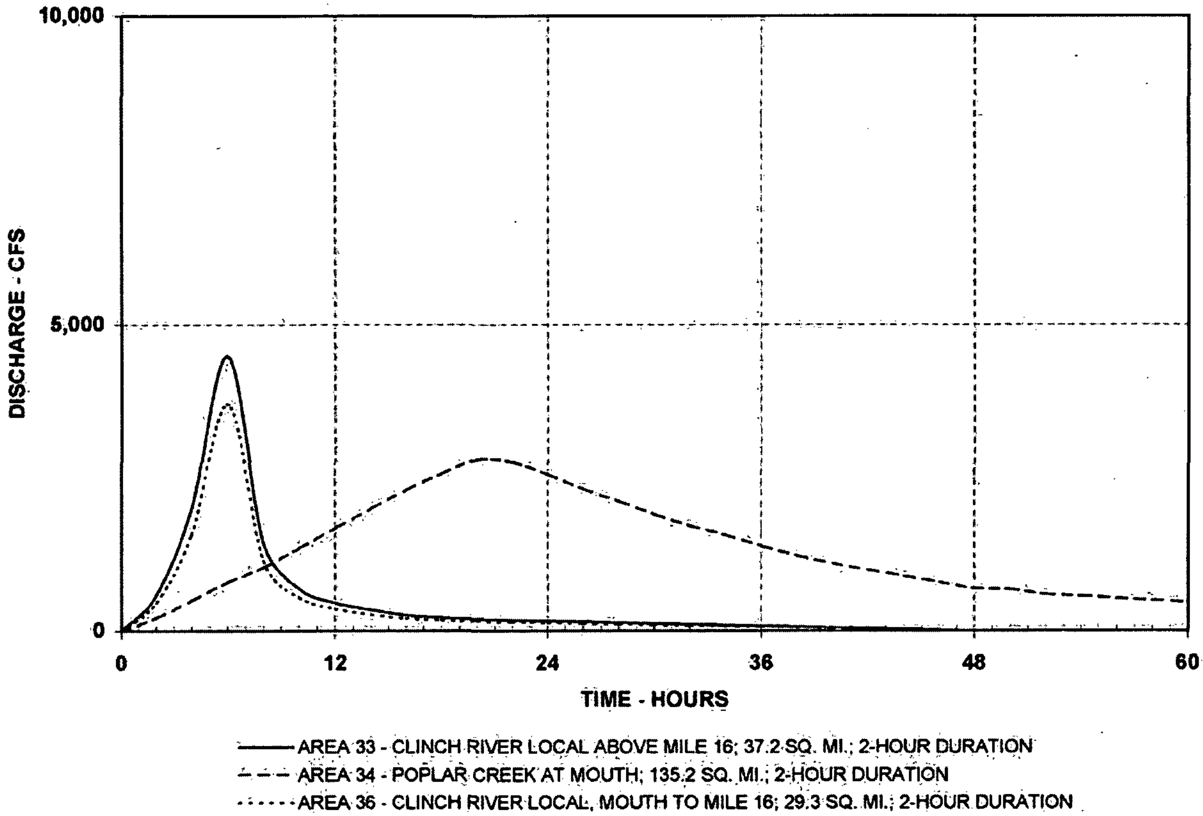
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 19-22 Figure 2.4.3-6 (Sheet 5 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 19–22 (Sheet 5 of 11)



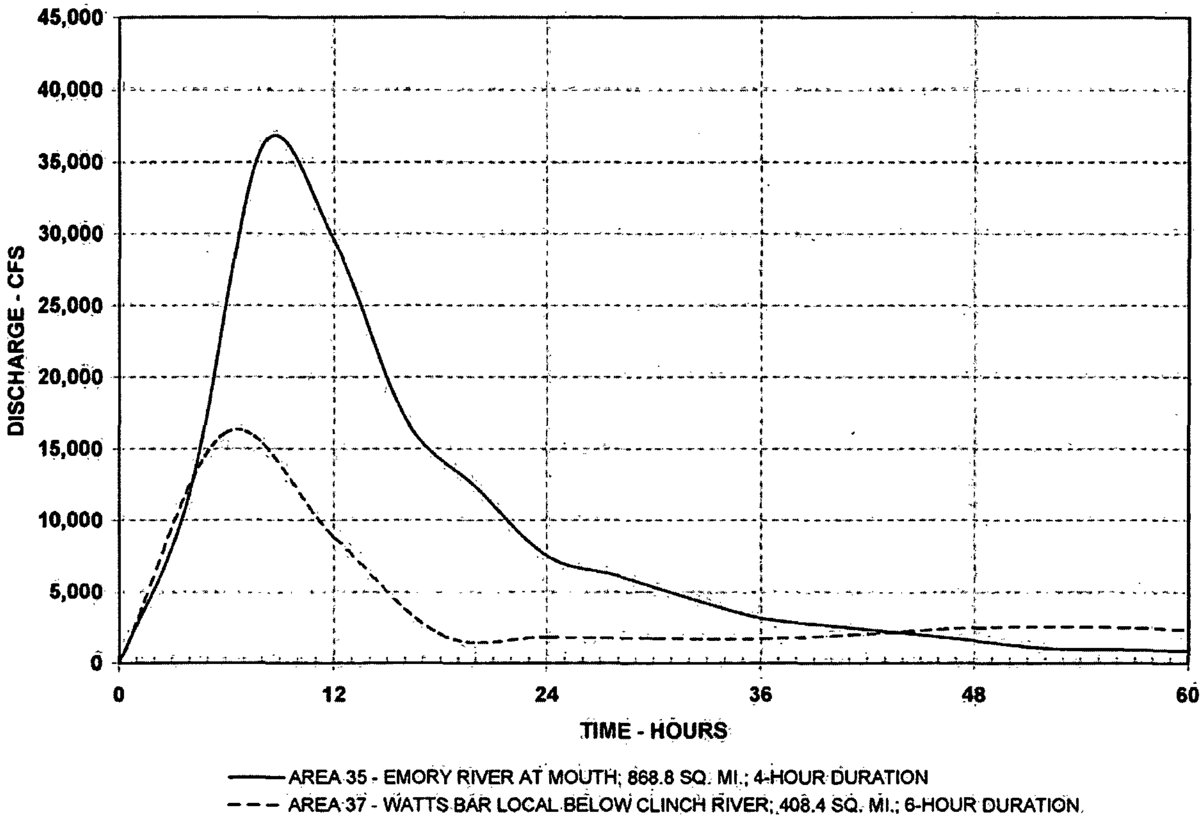
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 23-27 Figure 2.4.3-6 (Sheet 6 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 23–27 (Sheet 6 of 11)



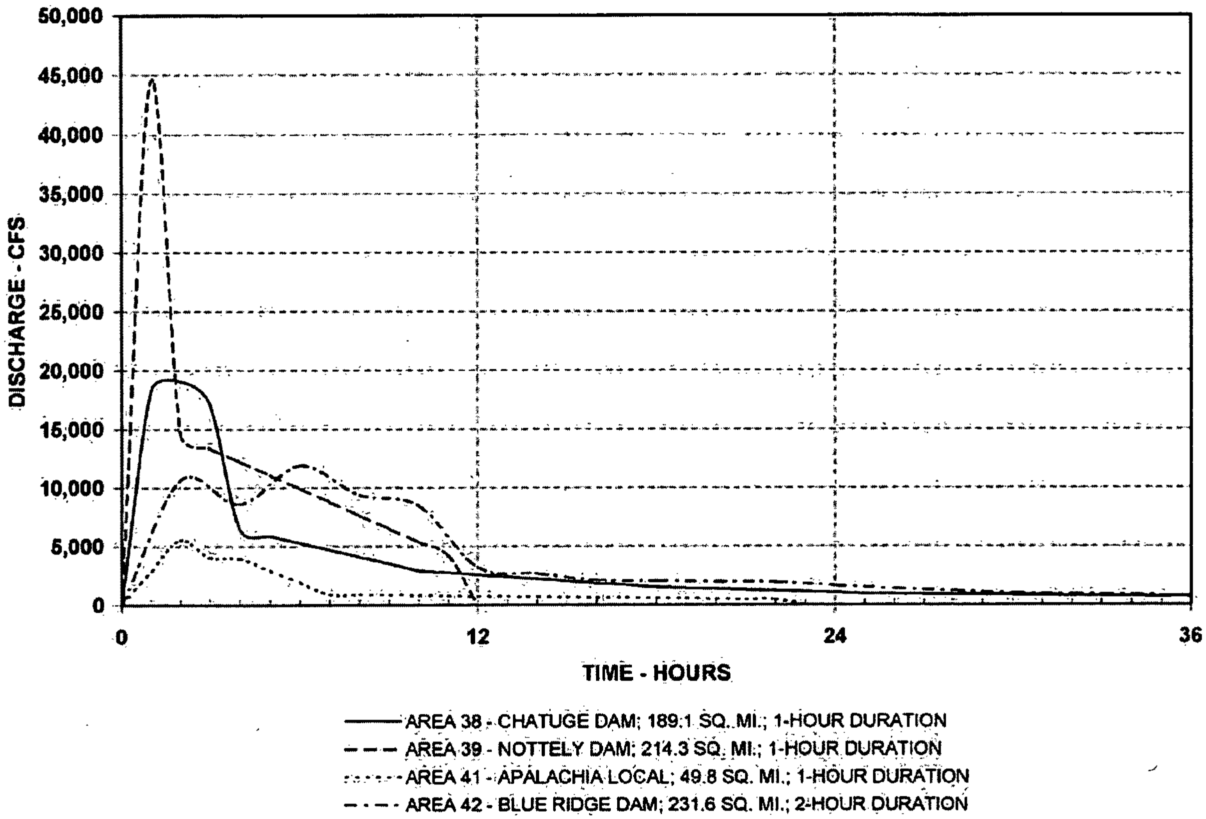
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 33, 34, 36 Figure 2.4.3-6 (Sheet 7 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 33, 34, 36 (Sheet 7 of 11)



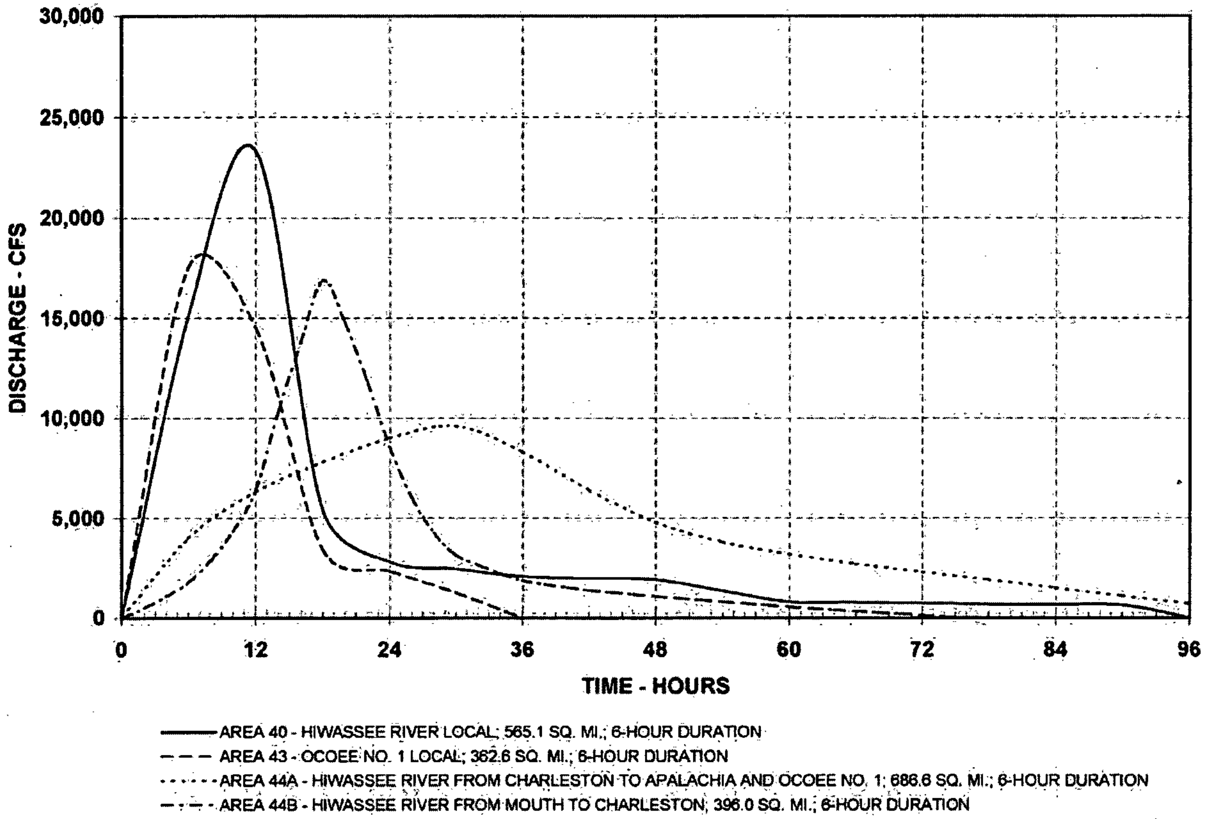
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 35, 37 Figure 2.4.3-6 (Sheet 8 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 35, 37 (Sheet 8 of 11)



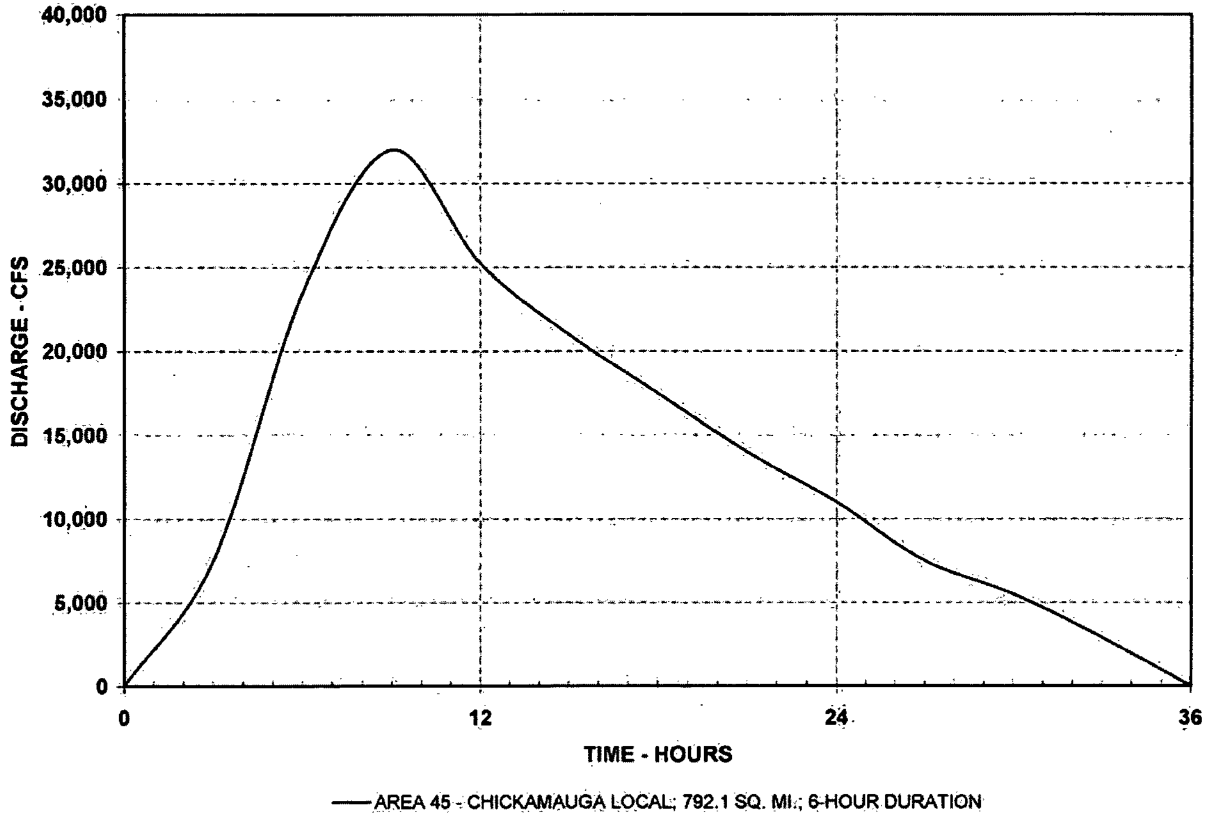
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 38, 39, 41, 42
Figure 2.4.3-6 (Sheet 9 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 38, 39, 41, 42 (Sheet 9 of 11)



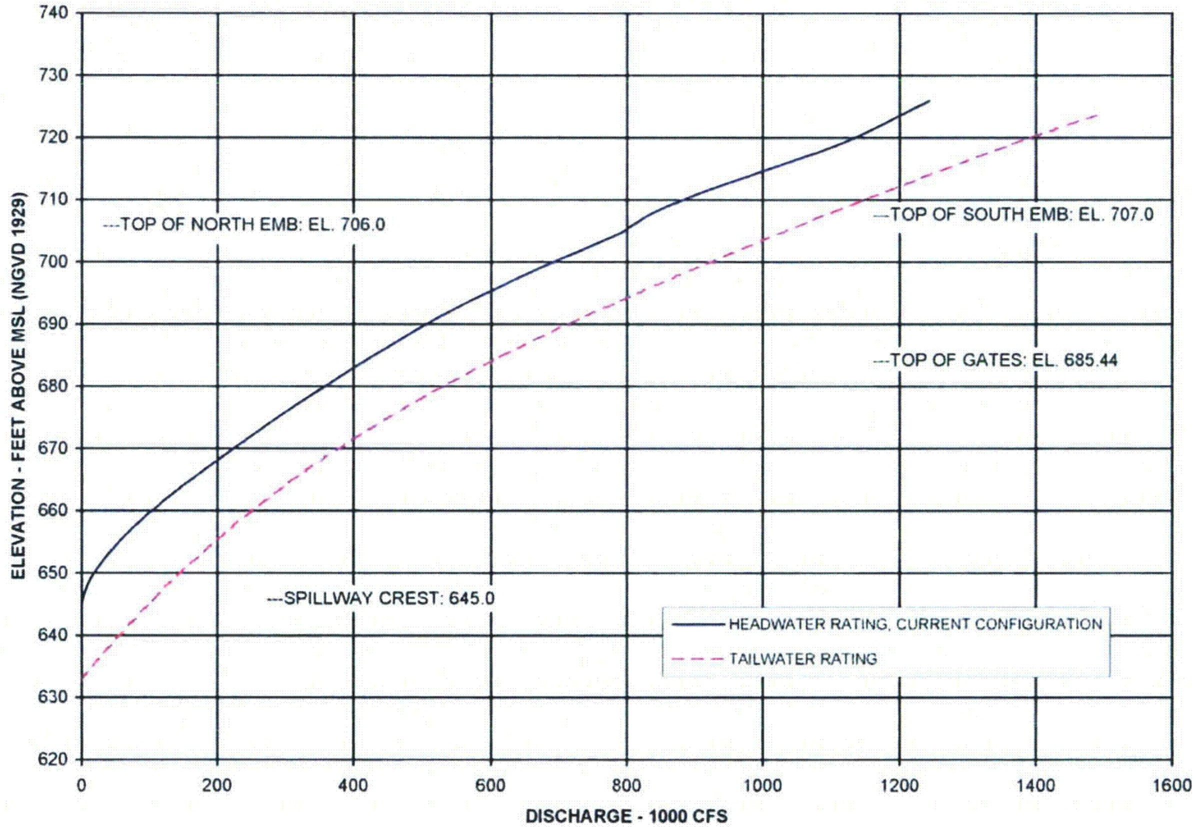
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 40, 43, 44A, 44B Figure 2.4.3-6 (Sheet 10 of 11)

Figure 2.4.3-6 Unit Hydrographs, Areas 40, 43, 44A, 44B (Sheet 10 of 11)



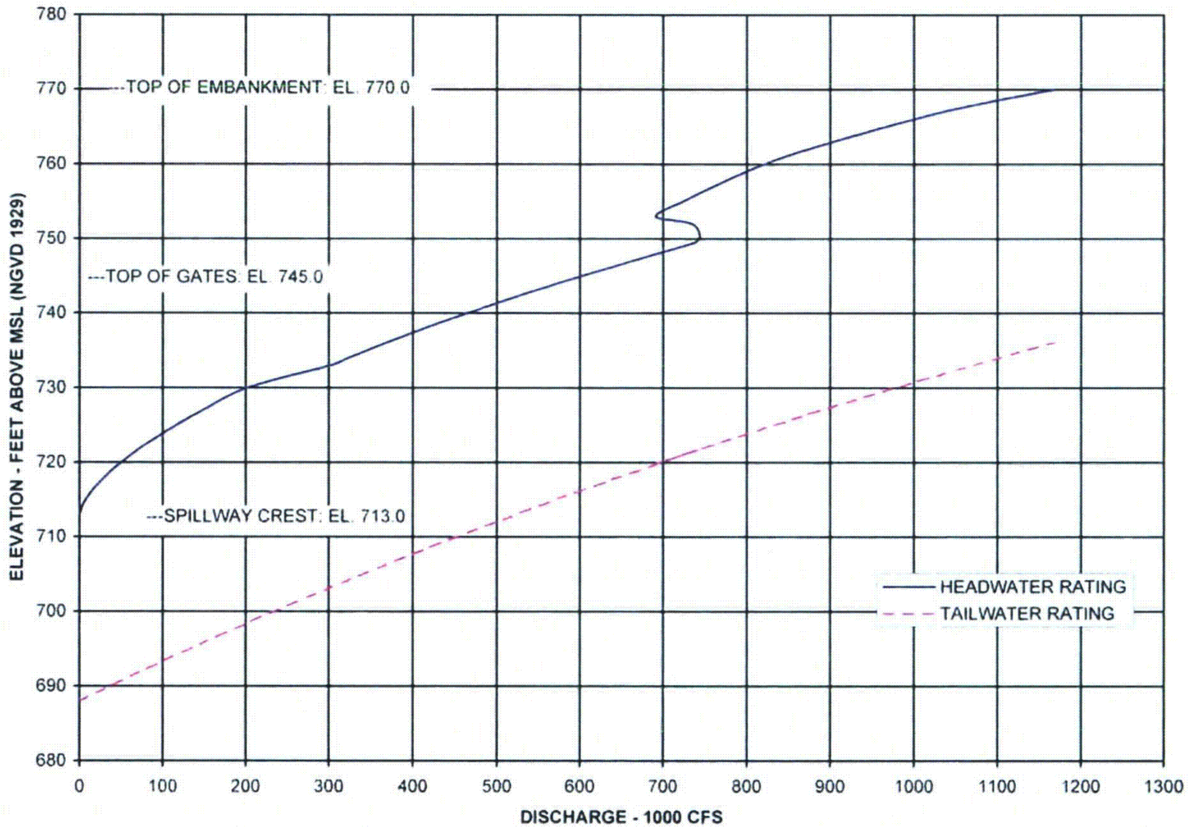
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Area 45
Figure 2.4.3-6 (Sheet 11 of 11)

Figure 2.4.3-6 Unit Hydrographs, Area 45 (Sheet 11 of 11)



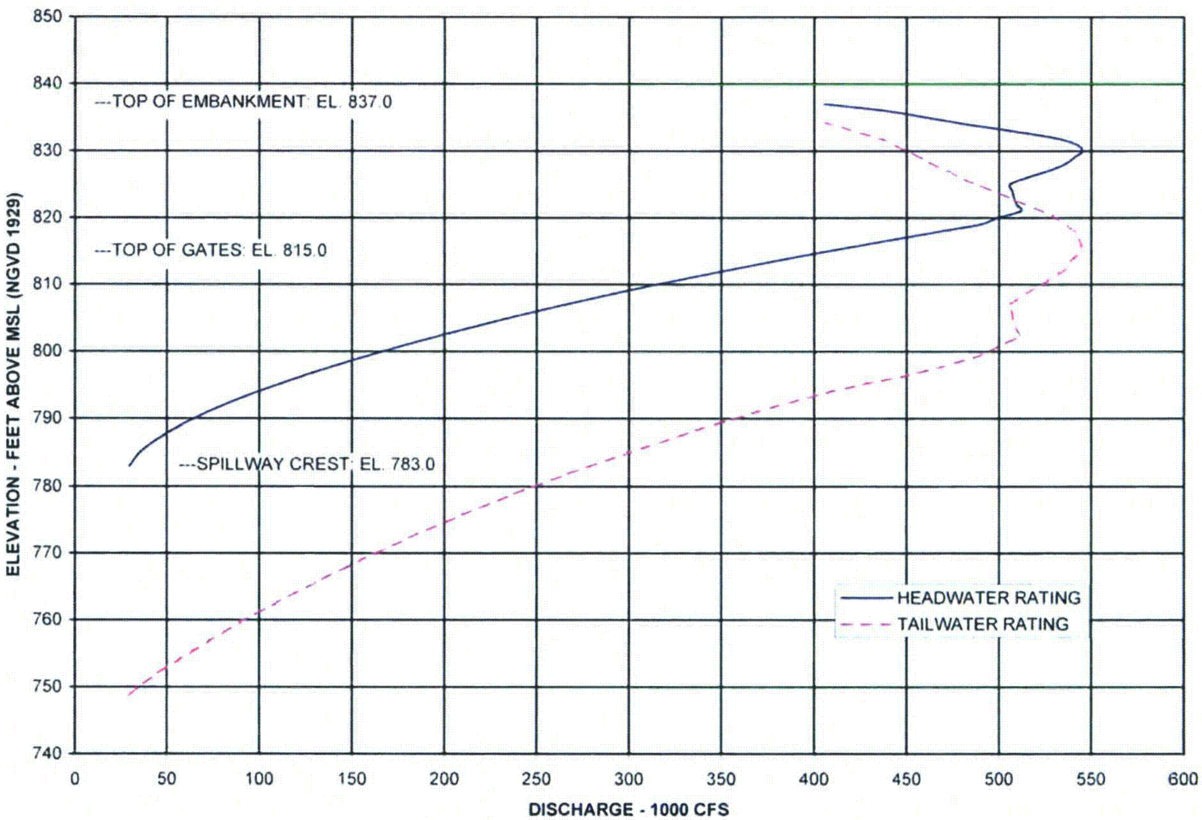
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Chickamauga Dam Figure 2.4.3-7(Sheet 1 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Chickamauga Dam (Sheet 1 of 17)



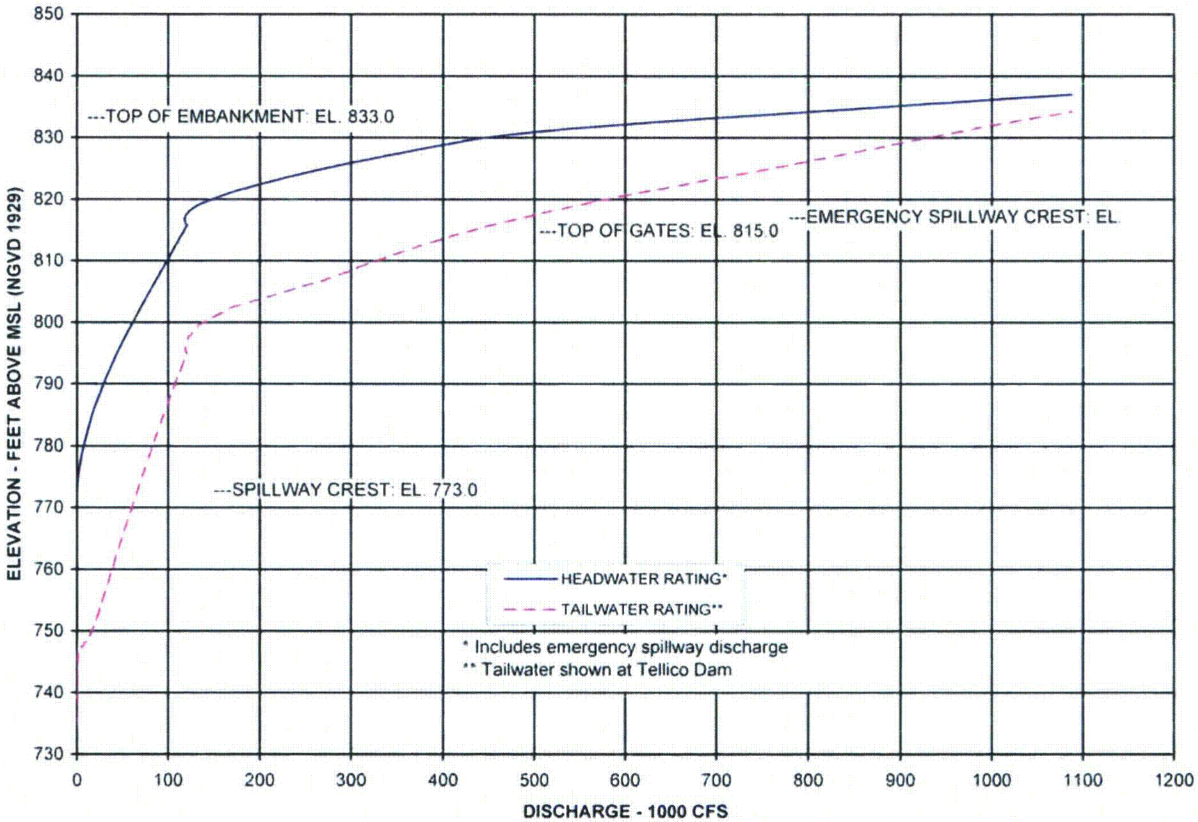
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Watts Bar Dam Figure 2.4.3-7 (Sheet 2 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Watts Bar Dam (Sheet 2 of 17)



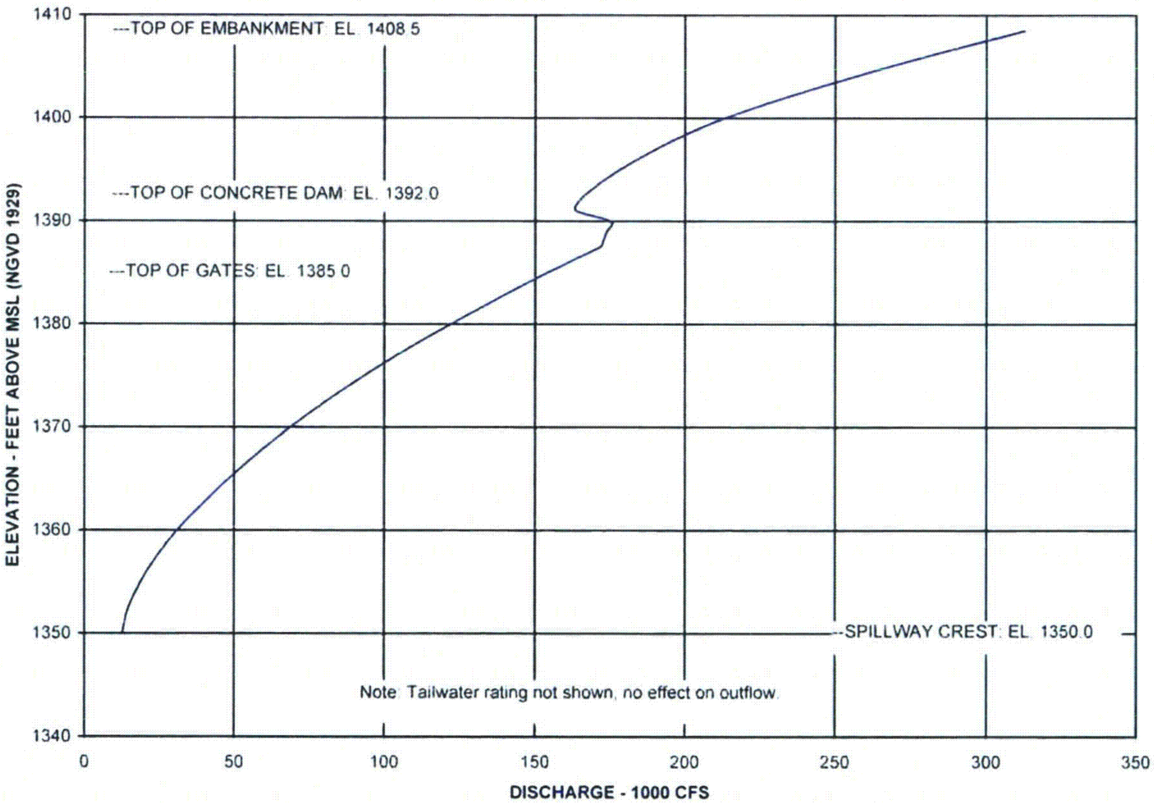
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Fort Loudoun Dam Figure 2.4.3-7 (Sheet 3 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Fort Loudoun Dam (Sheet 3 of 17)



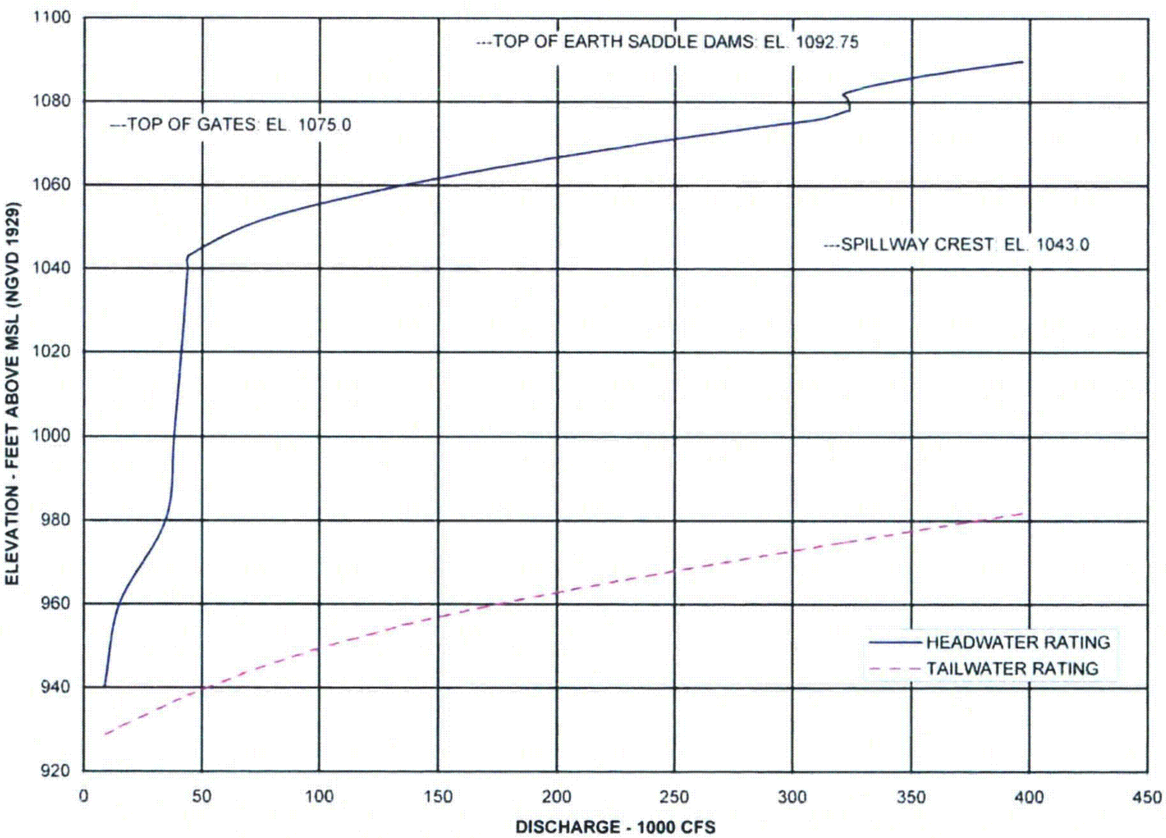
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Tellico Dam Figure 2.4.3-7 (Sheet 4 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Tellico Dam (Sheet 4 of 17)



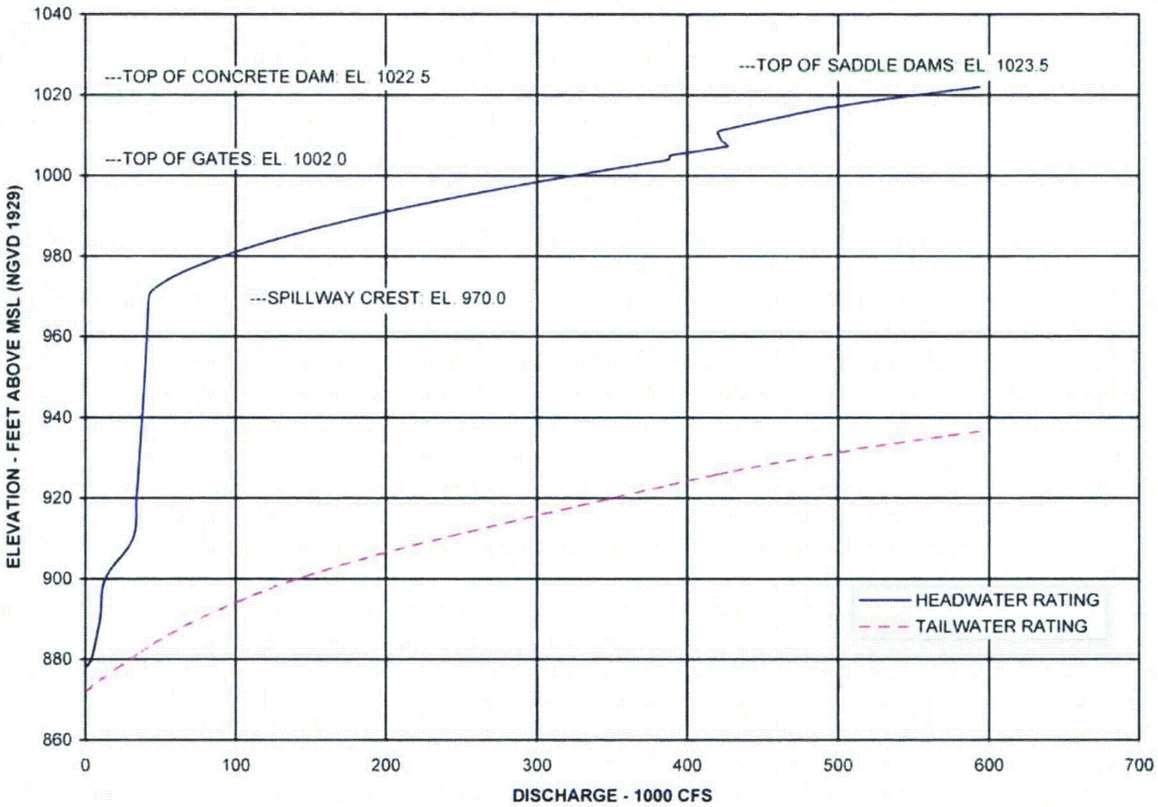
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Boone Dam Figure 2.4.3-7 (Sheet 5 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Boone Dam (Sheet 5 of 17)



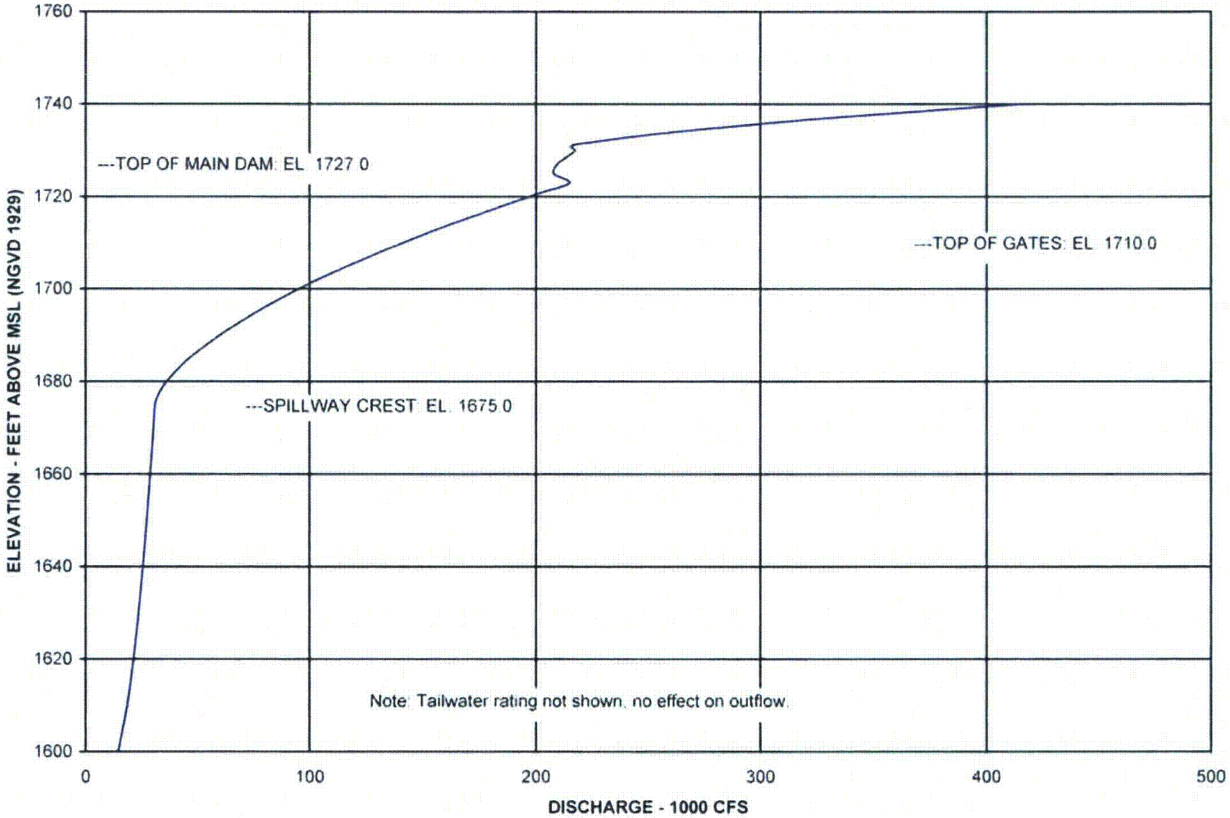
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Cherokee Dam
Figure 2.4.3-7 (Sheet 6 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Cherokee Dam (Sheet 6 of 17)



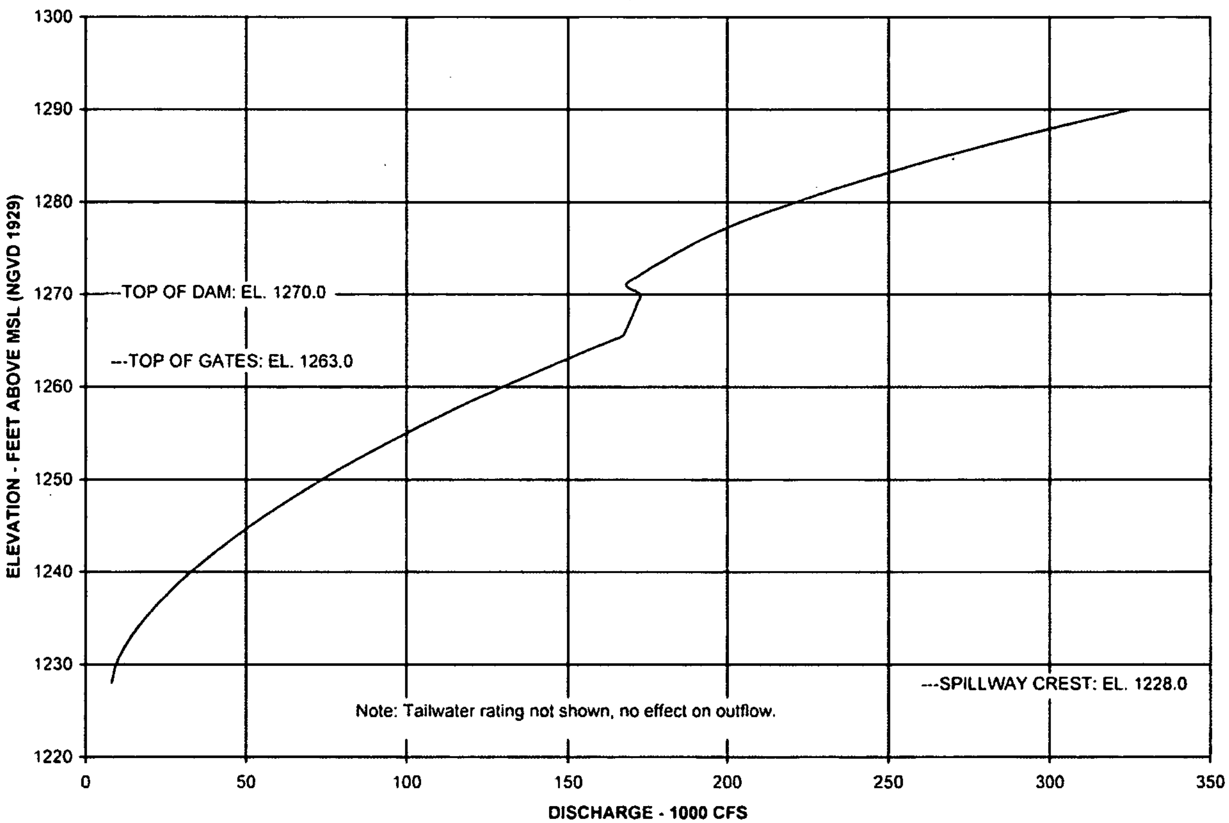
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Douglas Dam
Figure 2.4.3-7(Sheet 7 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Douglas Dam (Sheet 7 of 17)



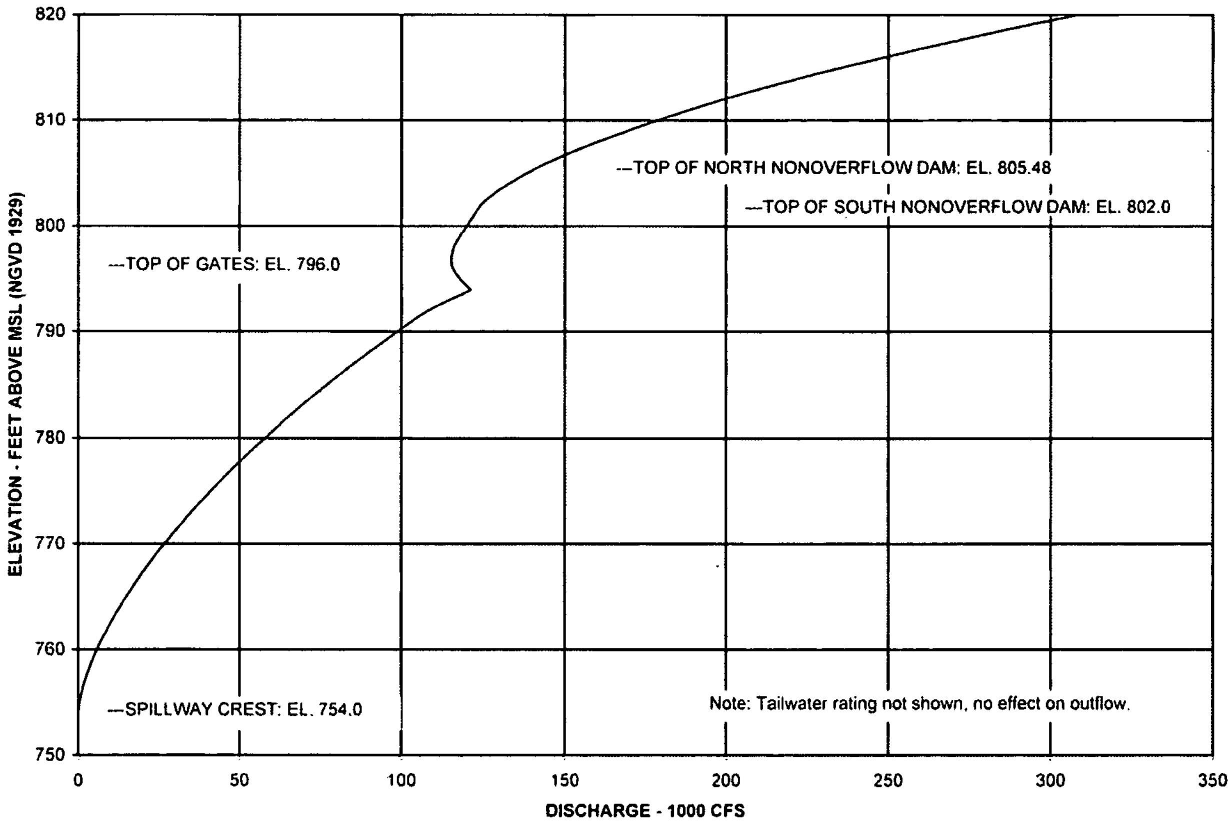
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Fontana Dam Figure 2.4.3-7 (Sheet 8 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Fontana Dam (Sheet 8 of 17)



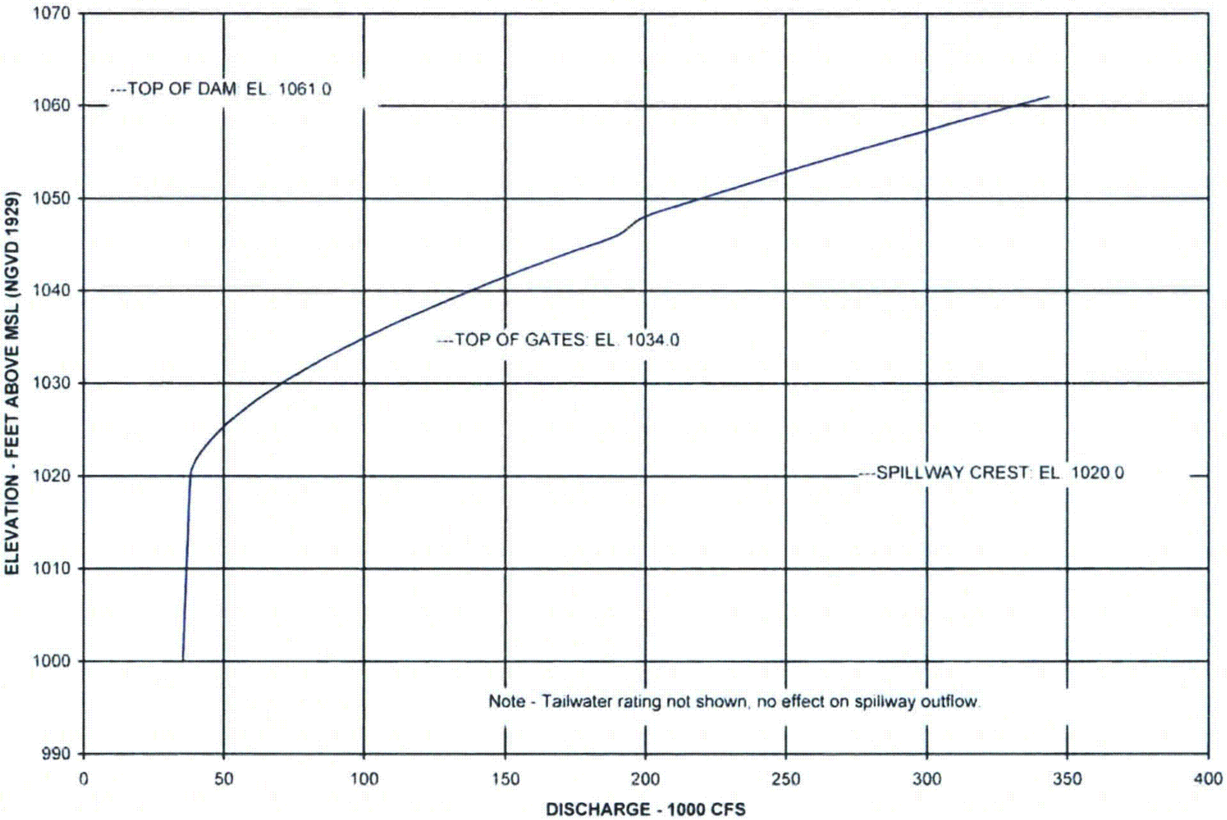
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Fort Patrick Henry Dam Figure 2.4.3-7 (Sheet 9 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Fort Patrick Henry Dam (Sheet 9 of 17)



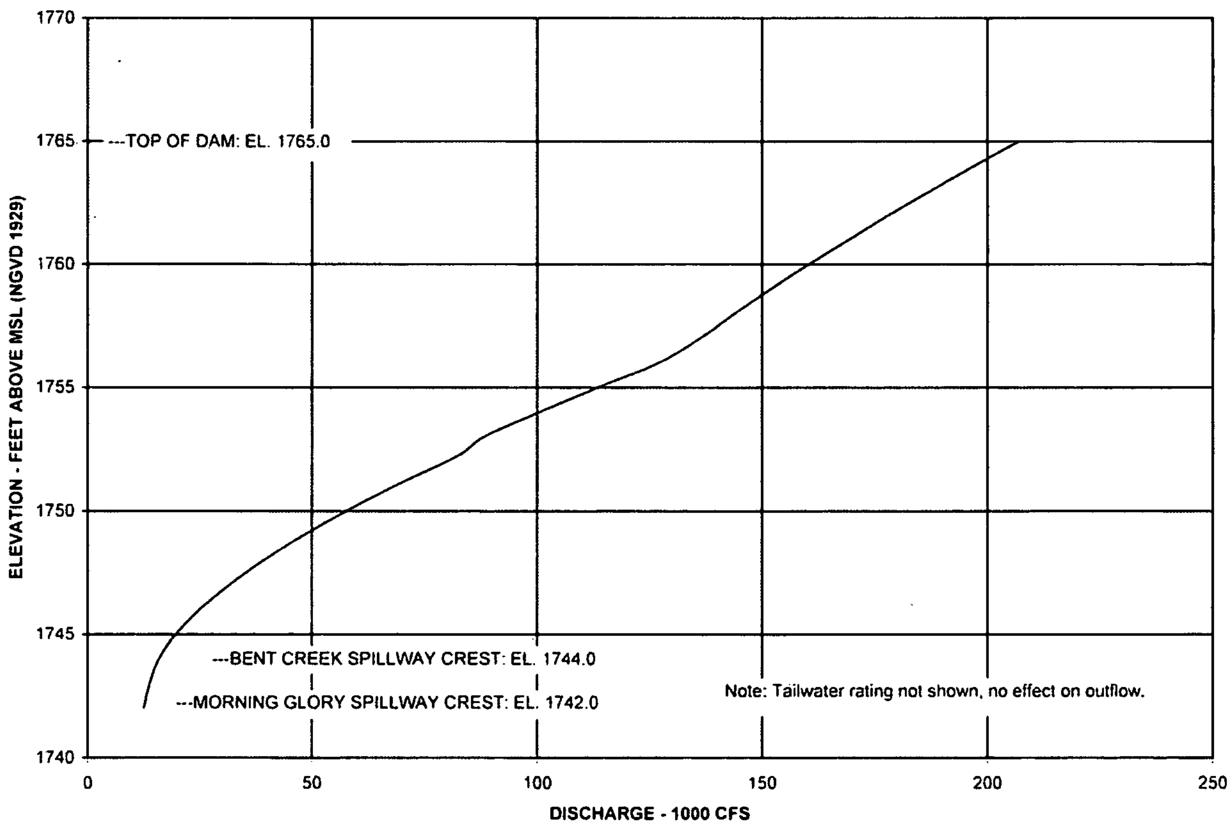
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Melton Hill Dam Figure 2.4.3-7 (Sheet 10 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Melton Hill Dam (Sheet 10 of 17)



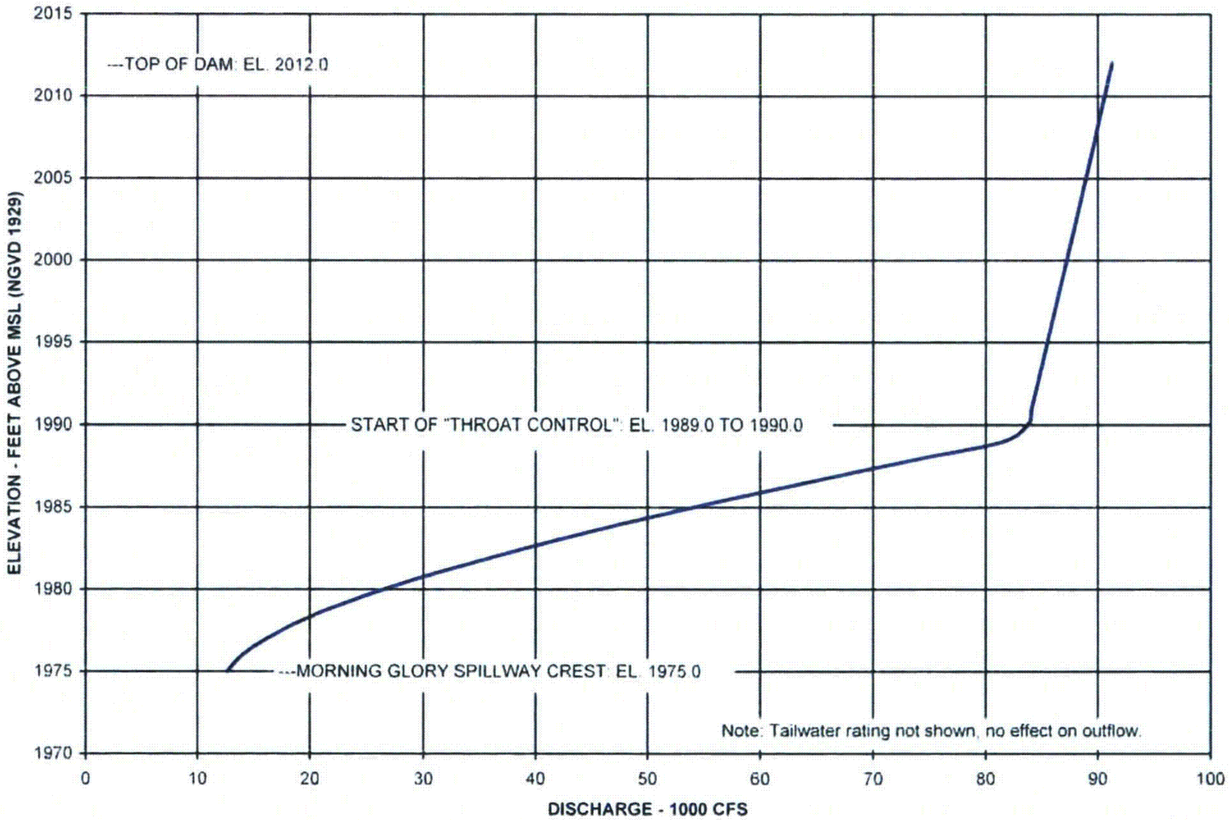
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Norris Dam Figure 2.4.3-7 (Sheet 11 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Norris Dam (Sheet 11 of 17)



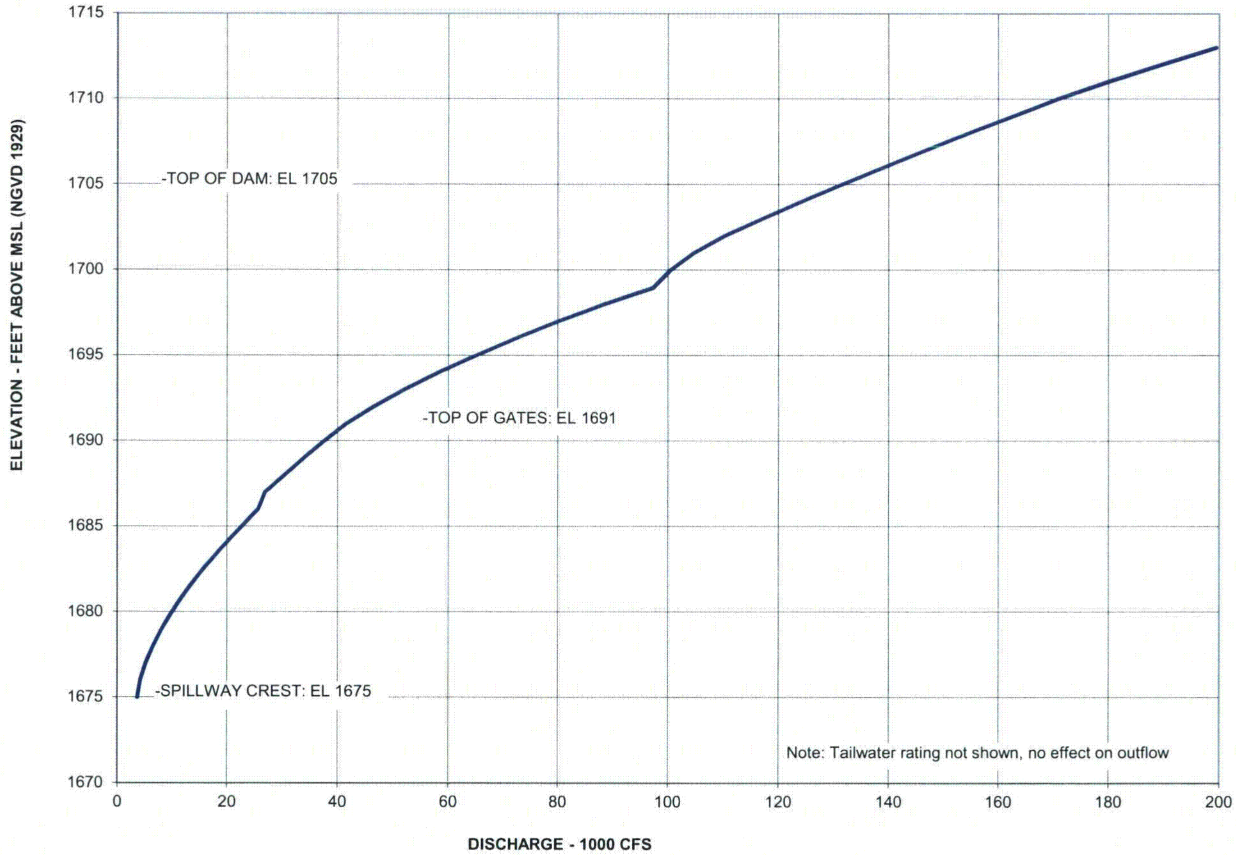
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, South Holston Dam Figure 2.4.3-7 (Sheet 12 of 17)

Figure 2.4.3-7 Discharge Rating Curve, South Holston Dam (Sheet 12 of 17)



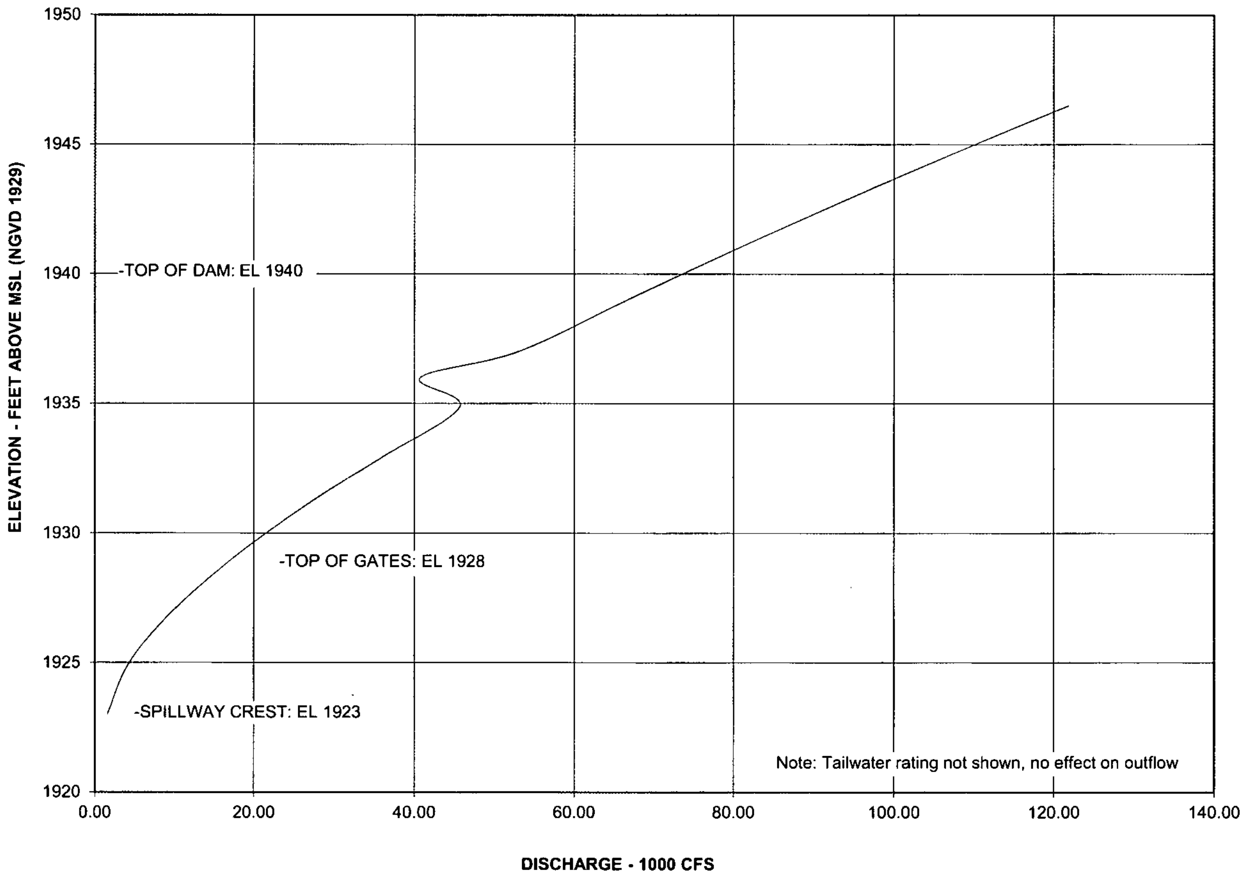
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Watauga Dam
Figure 2.4.3-7 (Sheet 13 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Watauga Dam (Sheet 13 of 17)



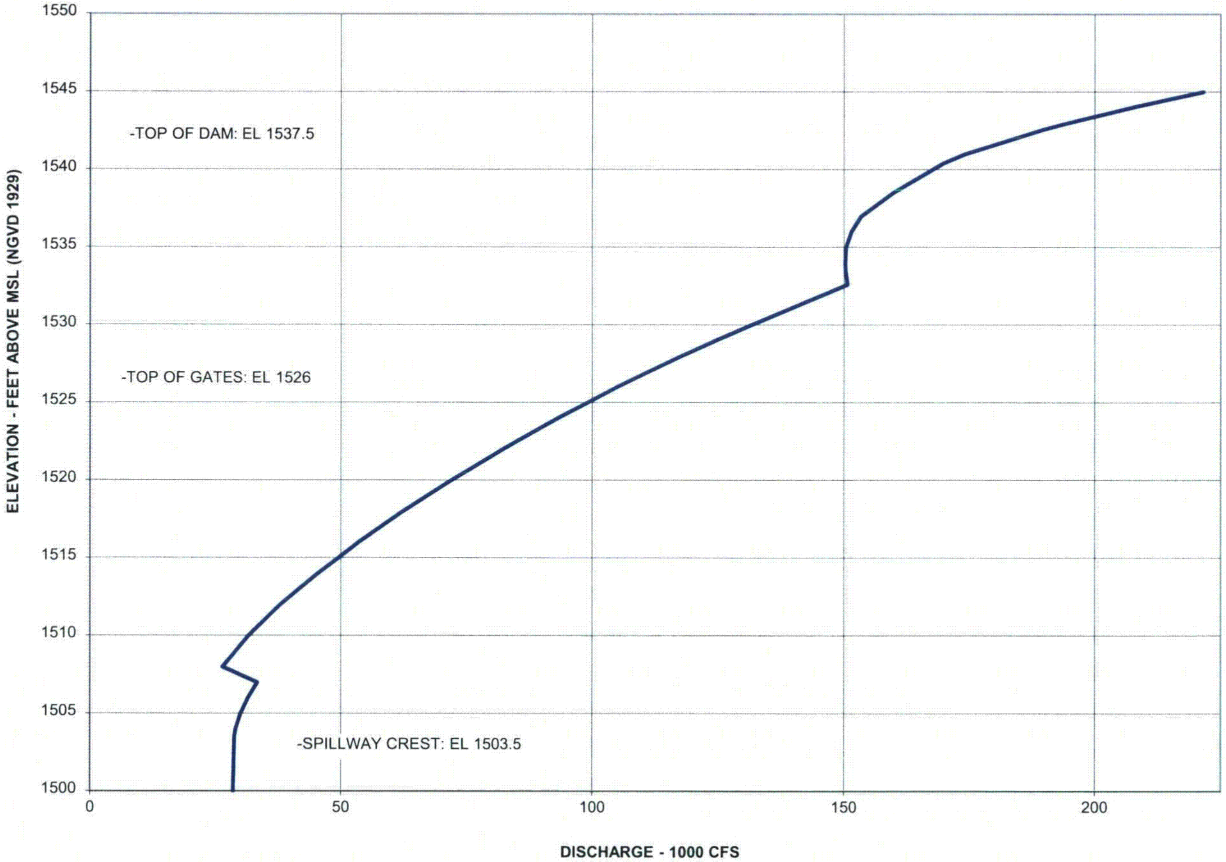
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Blue Ridge Dam
Figure 2.4.3-7 (Sheet 14 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Blue Ridge Dam (Sheet 14 of 17)



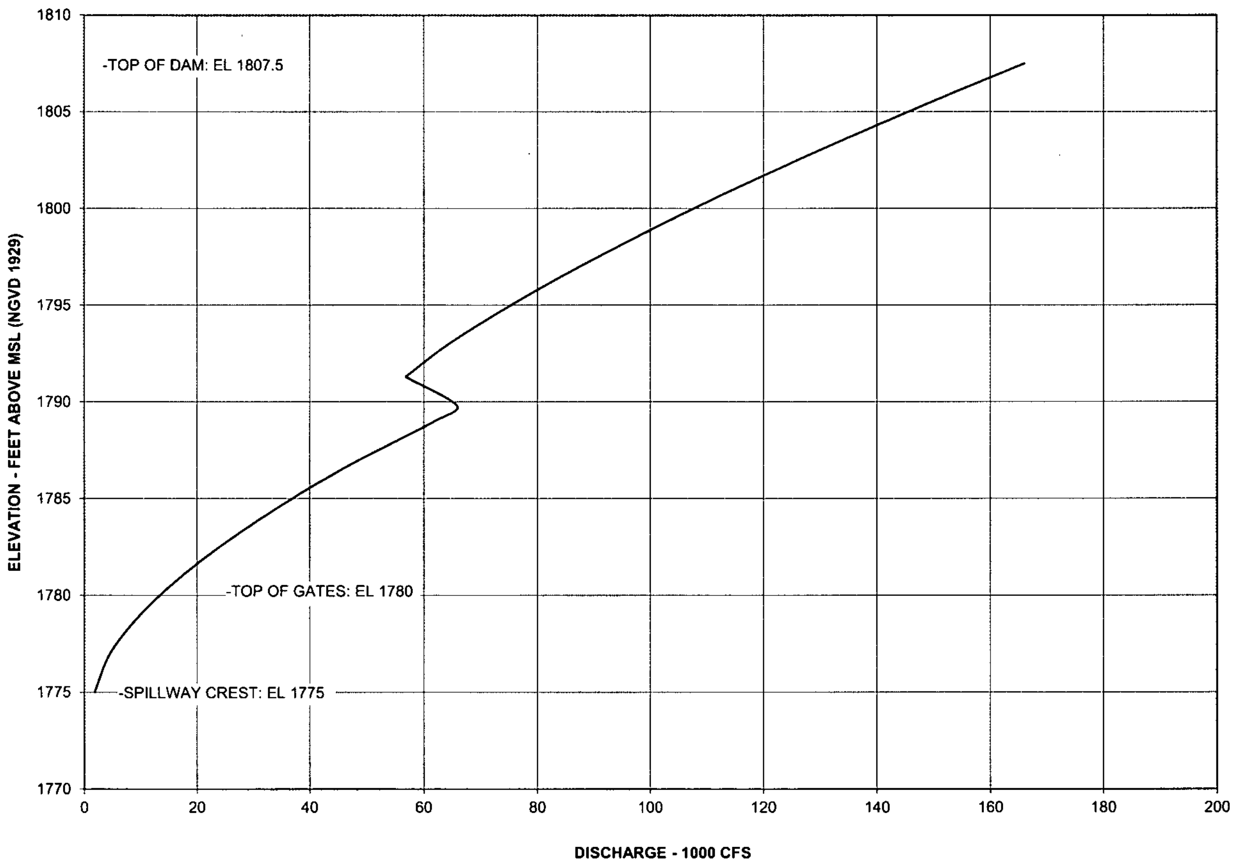
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Chatuge Dam Figure 2.4.3-7 (Sheet 15 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Chatuge Dam (Sheet 15 of 17)



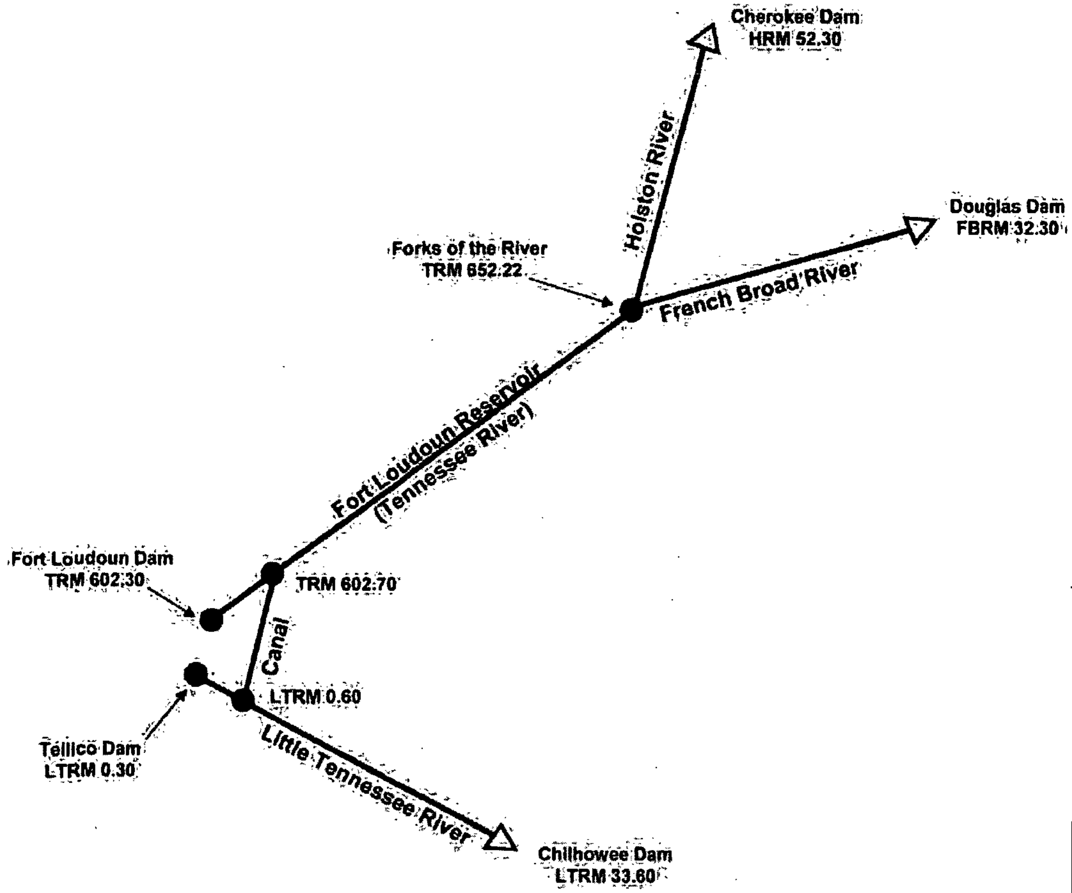
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Hiwassee Dam
Figure 2.4.3-7 (Sheet 16 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Hiwassee Dam (Sheet 16 of 17)



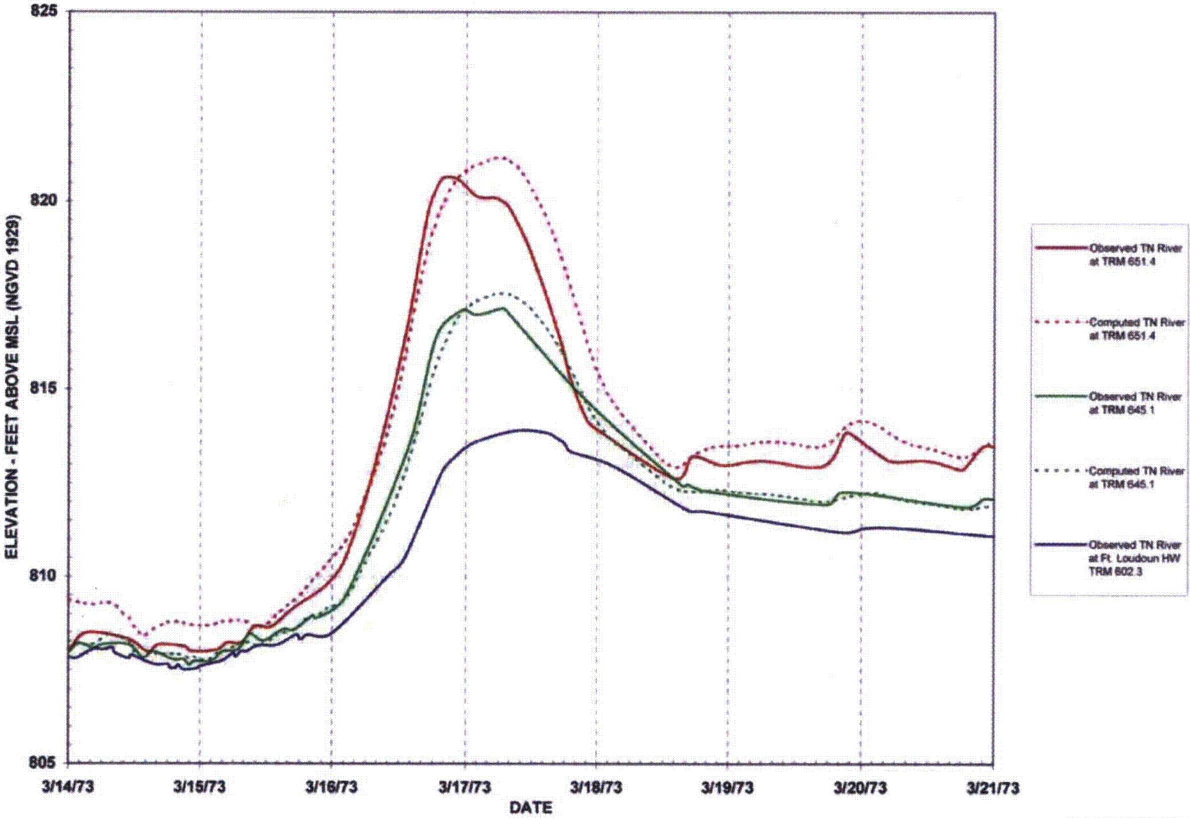
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Nottely Dam Figure 2.4.3-7 (Sheet 17 of 17)

Figure 2.4.3-7 Discharge Rating Curve, Nottely Dam (Sheet 17 of 17)



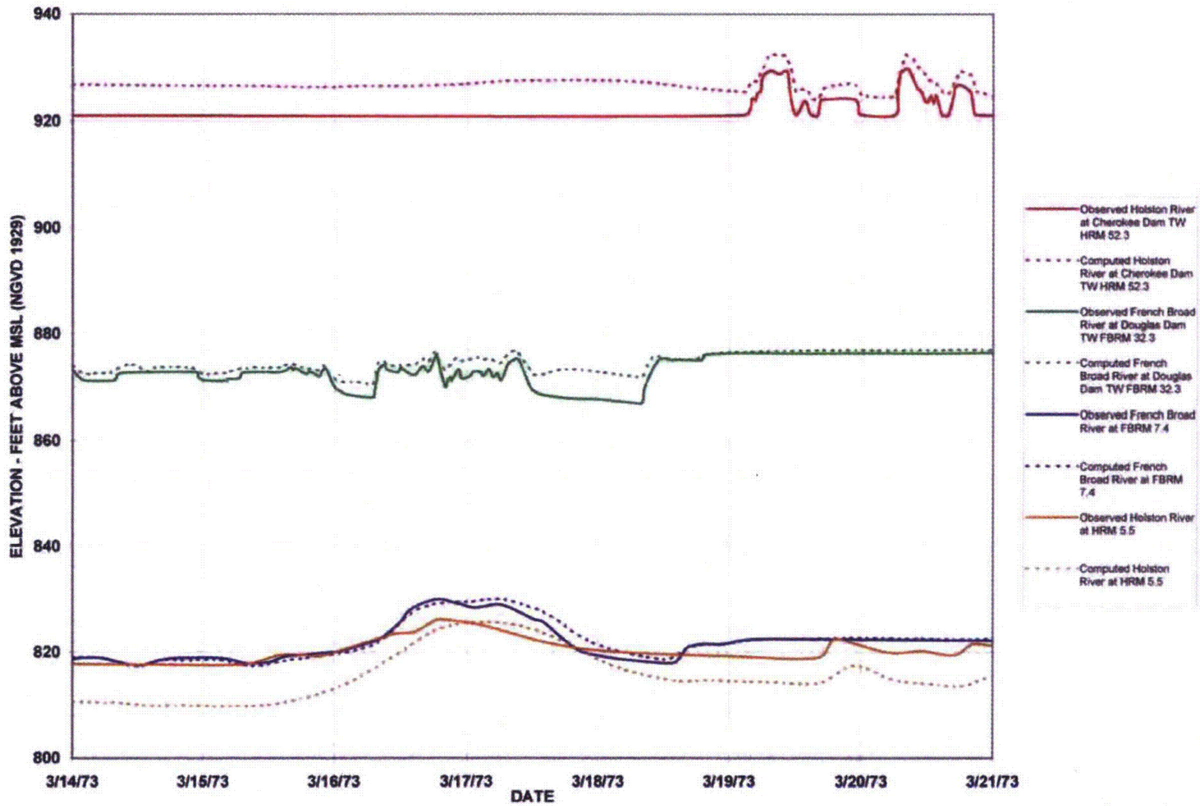
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Fort Loudoun - Tellico SOCH Unsteady Flow Model Schematic Figure 2.4.3-8

Figure 2.4.3-8 Fort Loudoun - Tellico SOCH Unsteady Flow Model Schematic



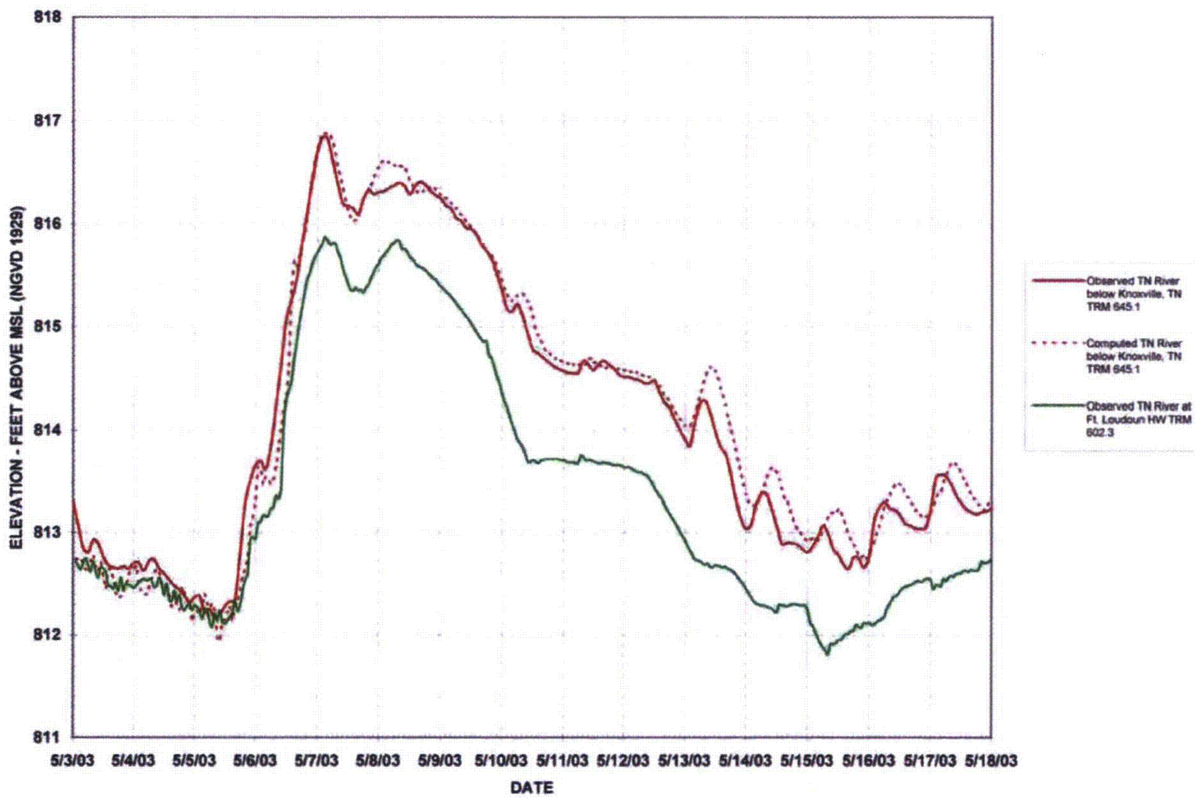
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood Figure 2.4.3-9 (Sheet 1 of 2)

Figure 2.4.3-9 Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood (Sheet 1 of 2)



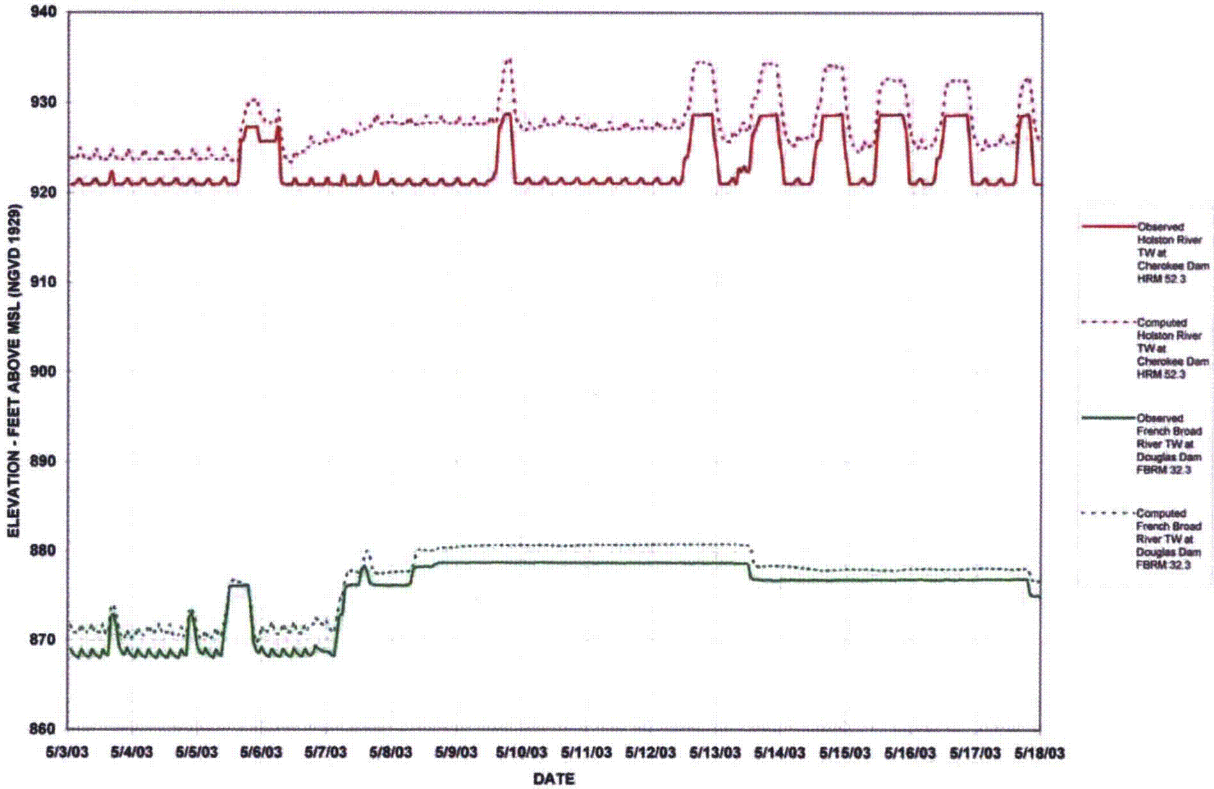
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood Figure 2.4.3-9 (Sheet 2 of 2)

Figure 2.4.3-9 Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood (Sheet 2 of 2)



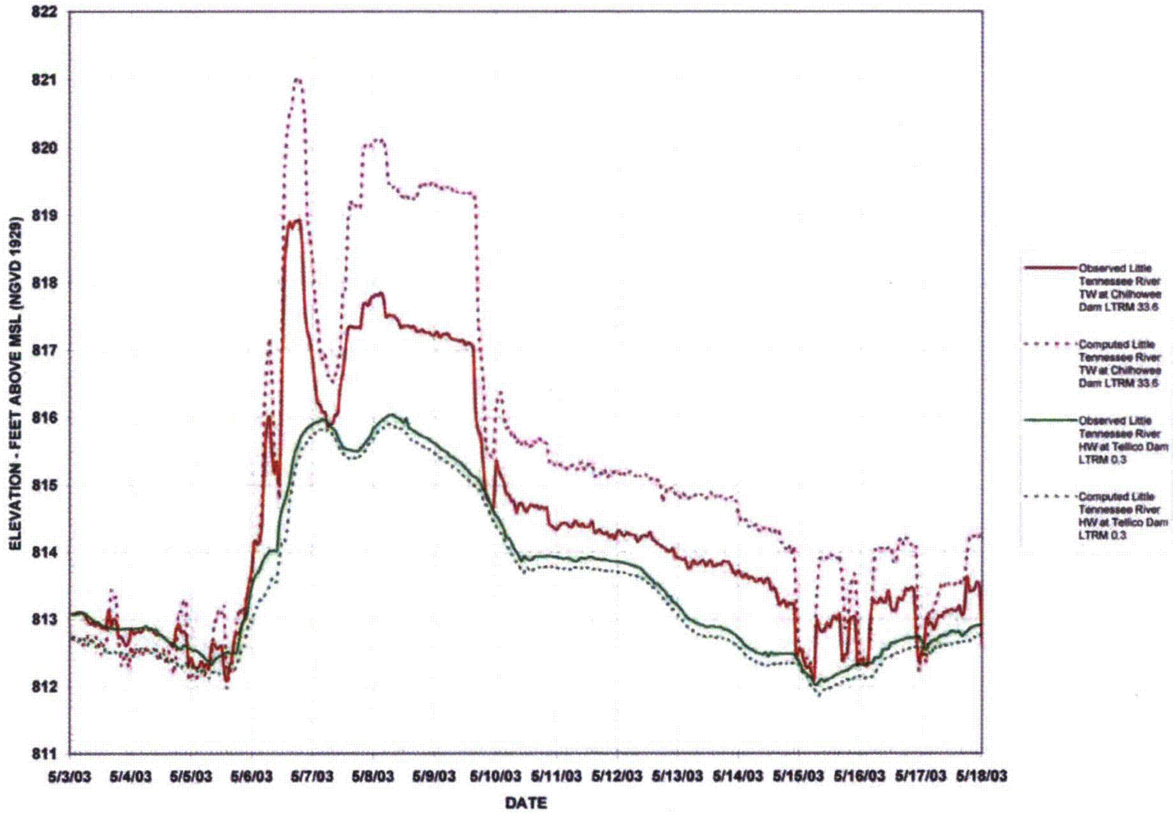
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood Figure 2.4.3-10 (Sheet 1 of 3)

Figure 2.4.3-10 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 1 of 3)



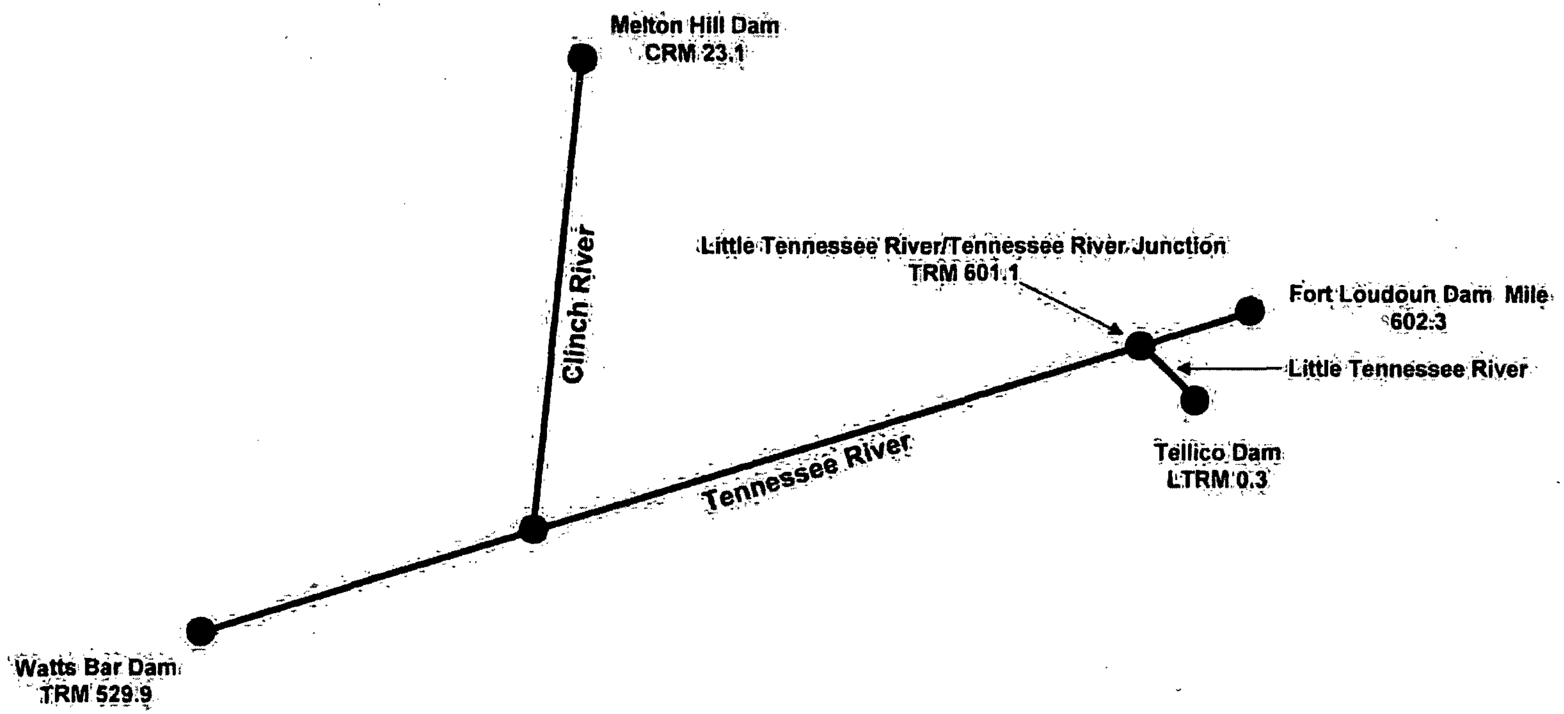
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood Figure 2.4.3-10 (Sheet 2 of 3)

Figure 2.4.3-10 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 2 of 3)



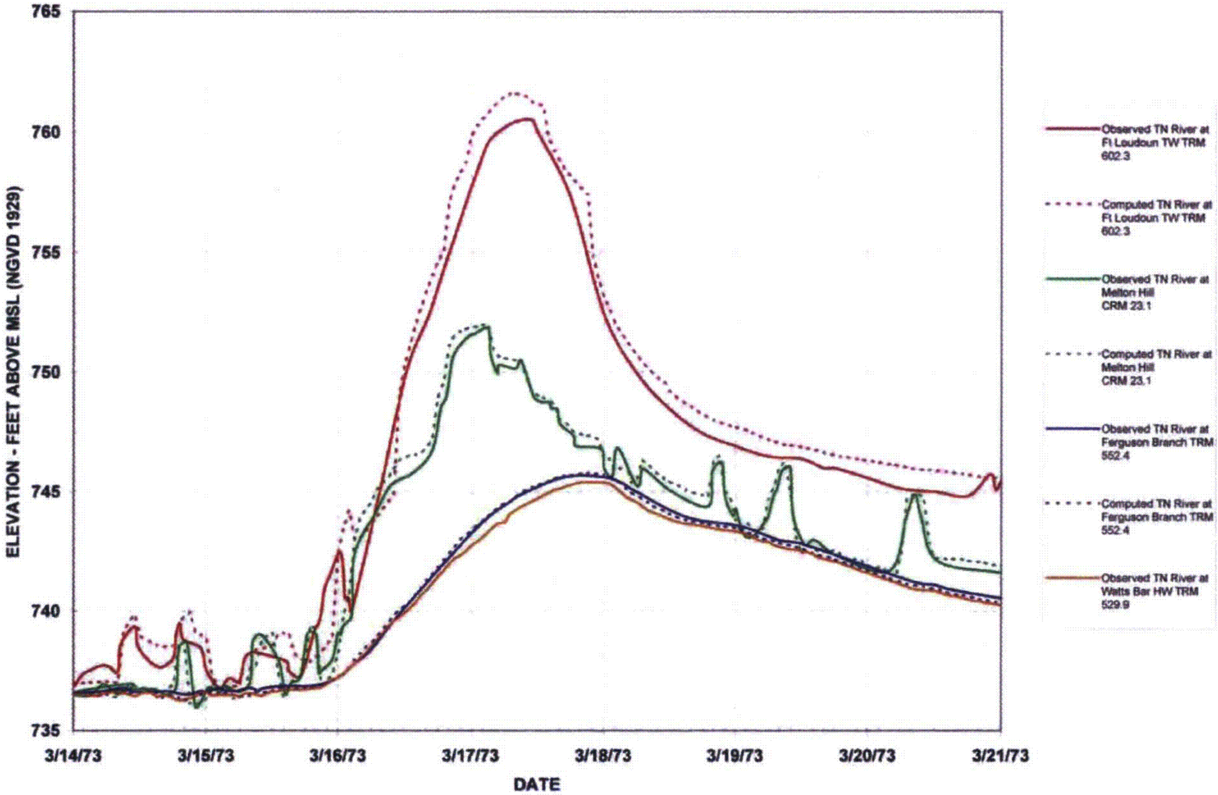
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood Figure 2.4.3-10 (Sheet 3 of 3)

Figure 2.4.3-10 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 3 of 3)



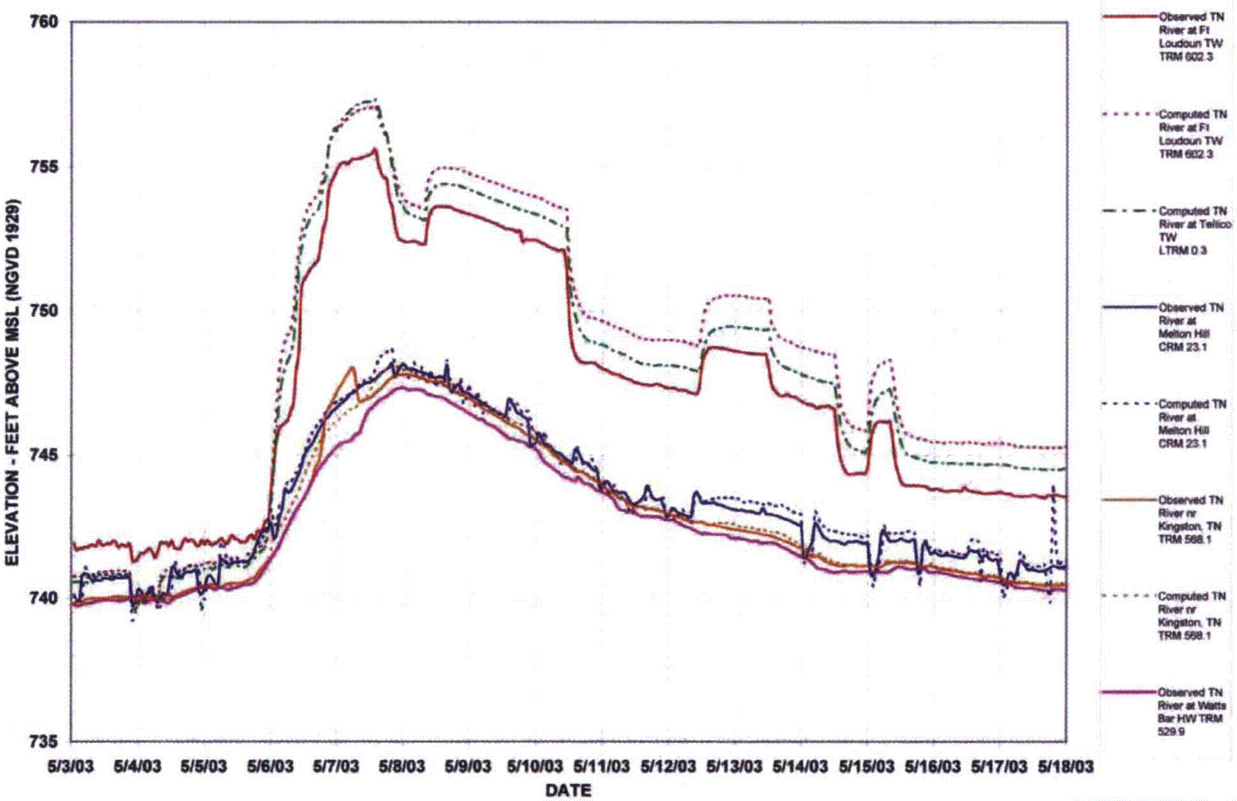
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Watts Bar SOCH Unsteady Flow Model Schematic Figure 2.4.3-11

Figure 2.4.3-11 Watts Bar SOCH Unsteady Flow Model Schematic



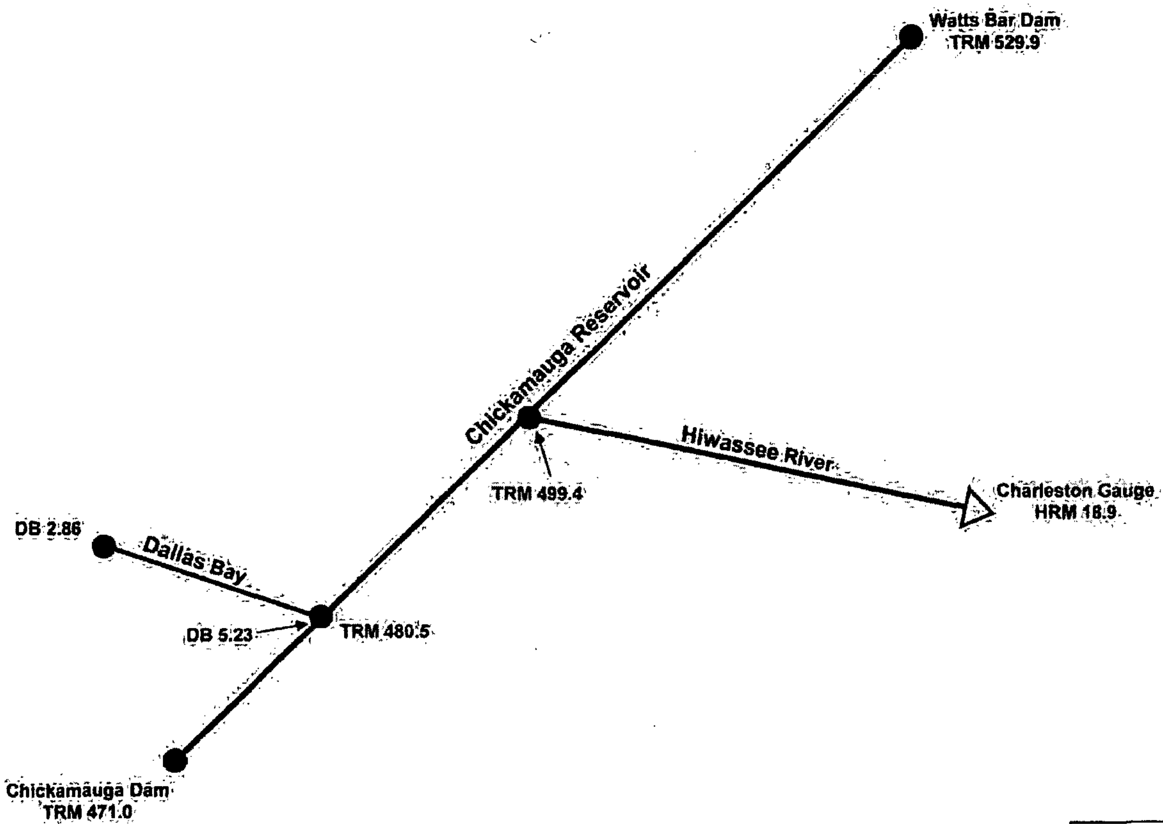
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Watts Bar Reservoir March 1973 Flood Figure 2.4.3-12

Figure 2.4.3-12 Unsteady Flow Model Watts Bar Reservoir March 1973 Flood



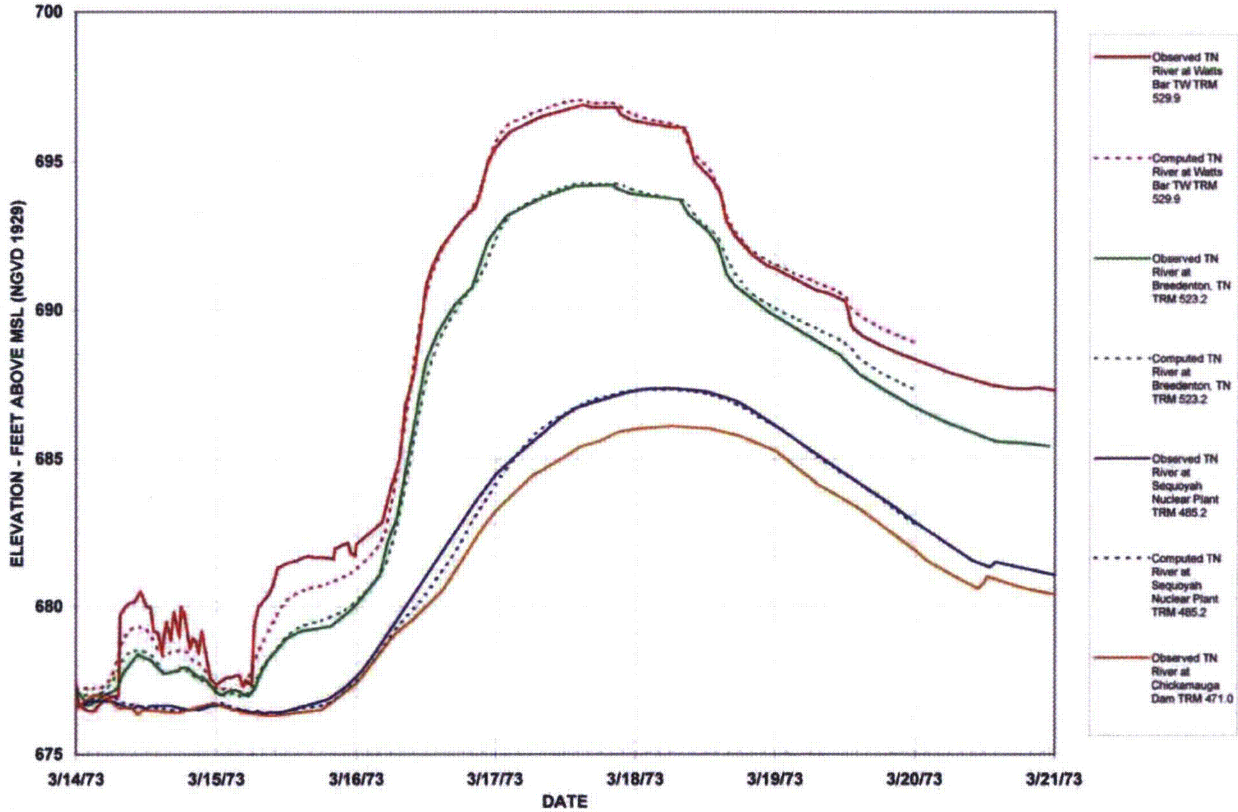
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Watts Bar Reservoir May 2003 Flood Figure 2.4.3-13

Figure 2.4.3-13 Unsteady Flow Model Watts Bar Reservoir May 2003 Flood



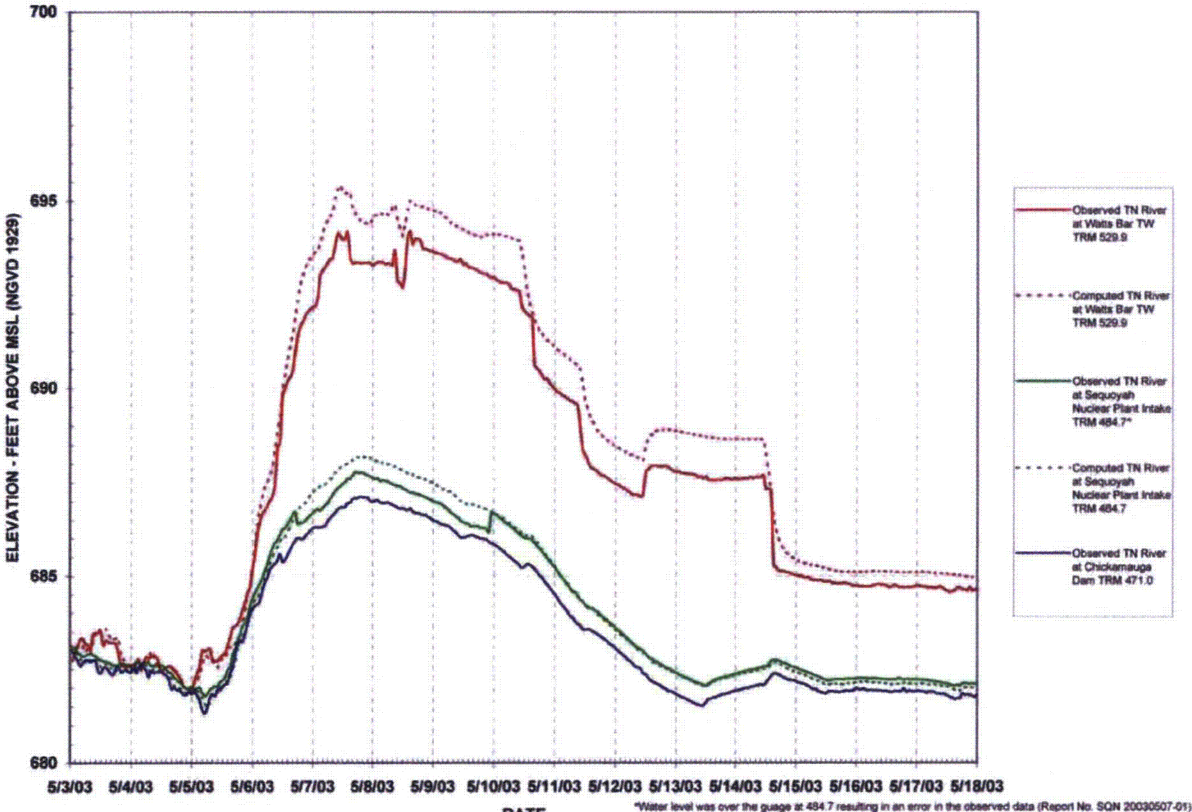
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Chickamauga SOCH Unsteady Flow Model Schematic Figure 2.4.3-14

Figure 2.4.3-14 Chickamauga SOCH Unsteady Flow Model Schematic



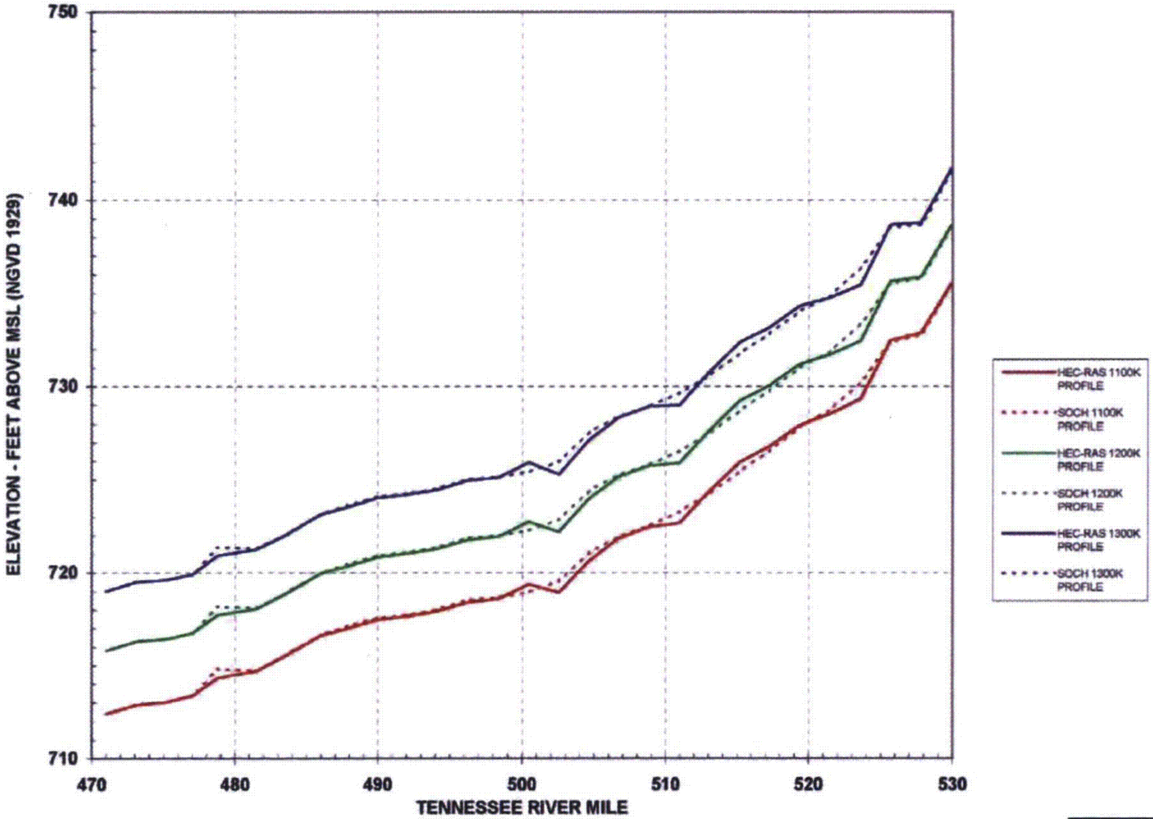
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Chickamauga Reservoir March 1973 Flood Figure 2.4.3-15

Figure 2.4.3-15 Unsteady Flow Model Chickamauga Reservoir March 1973 Flood



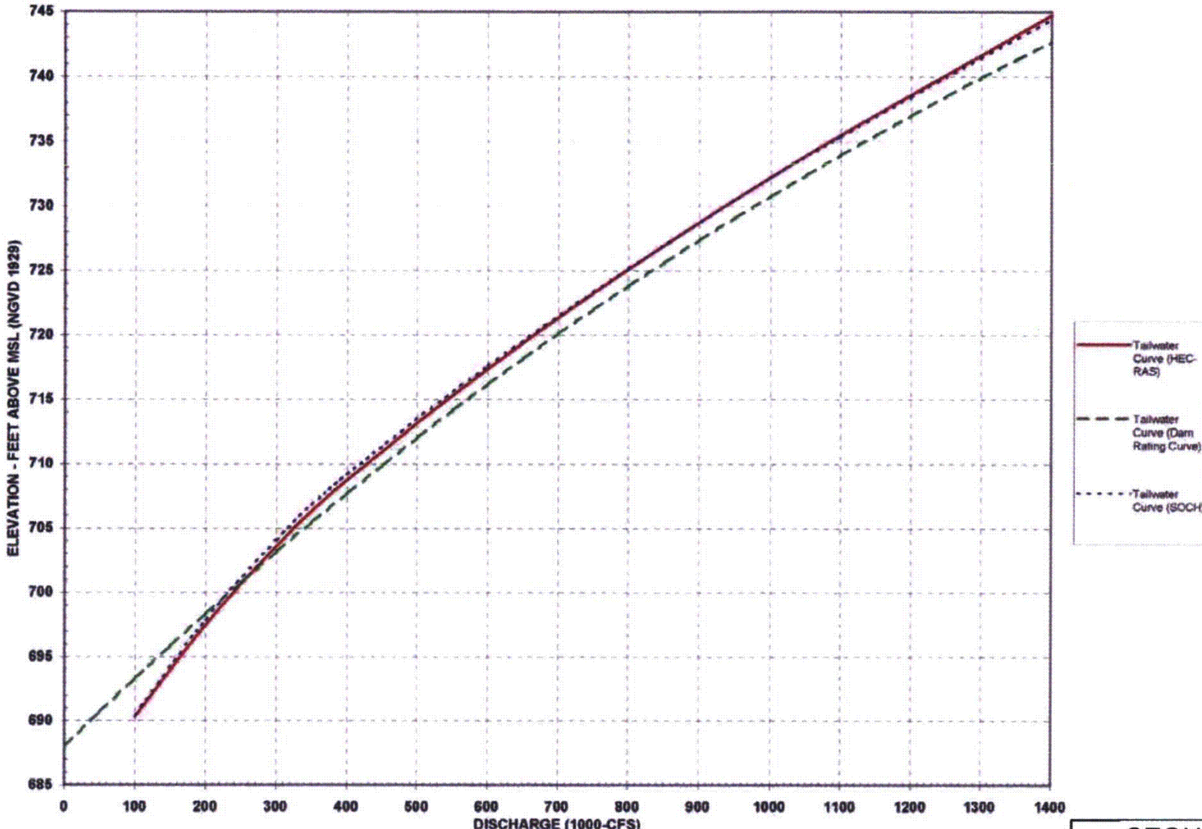
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Chickamauga Reservoir May 2003 Flood Figure 2.4.3-16

Figure 2.4.3-16 Unsteady Flow Model Chickamauga Reservoir May 2003 Flood



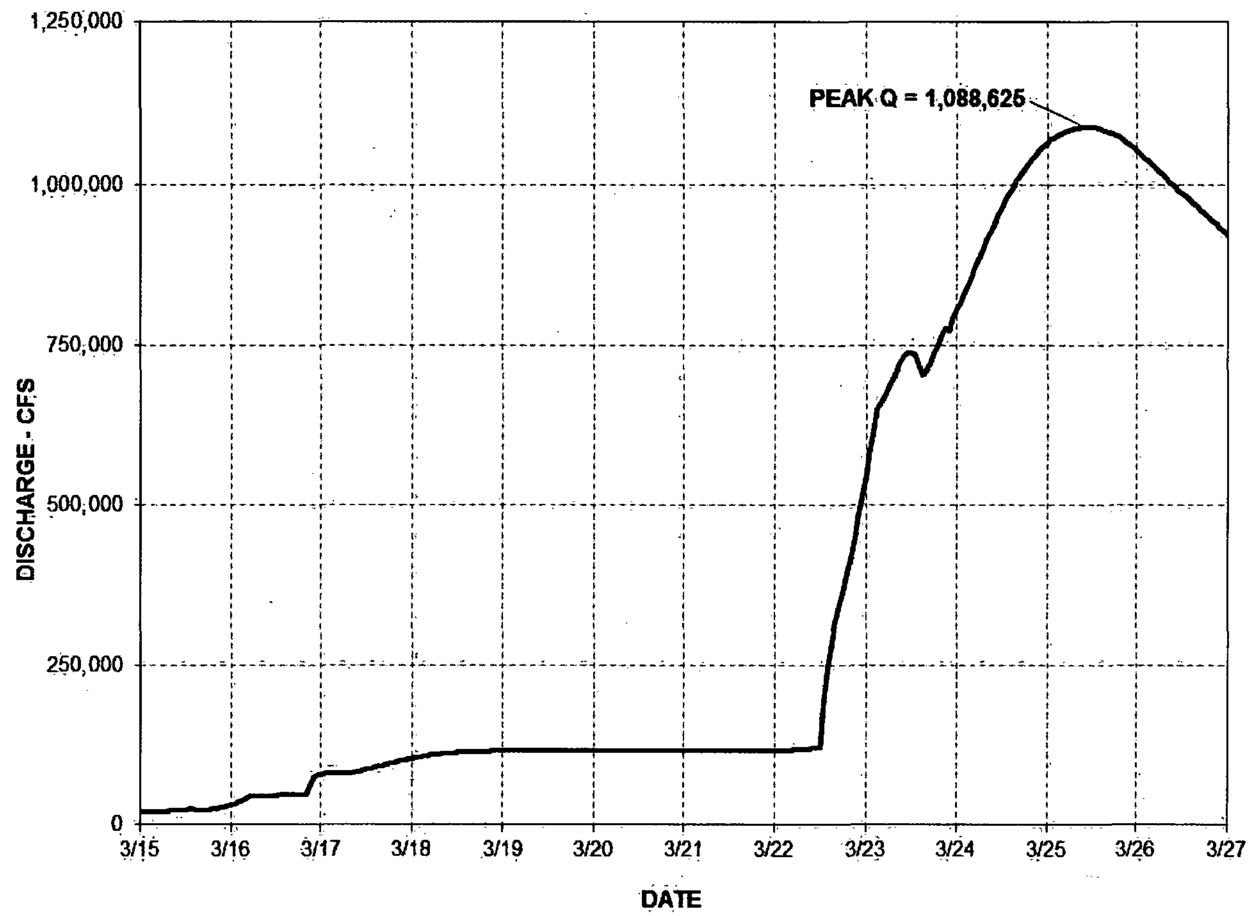
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Chickamauga Steady State Profile Comparisons Figure 2.4.3-17

Figure 2.4.3-17 Chickamauga Steady State Profile Comparisons



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Tailwater Rating Curve, Watts Bar Dam Figure 2.4.3-18

Figure 2.4.3-18 Tailwater Rating Curve, Watts Bar Dam



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
PMF Discharge Hydrograph at Sequoyah Nuclear Plant Figure 2.4.3-19

Figure 2.4.3-19 PMF Discharge Hydrograph at Sequoyah Nuclear Plant

Figure 2.4.3-20 West Saddle Dike Location Plan and Section

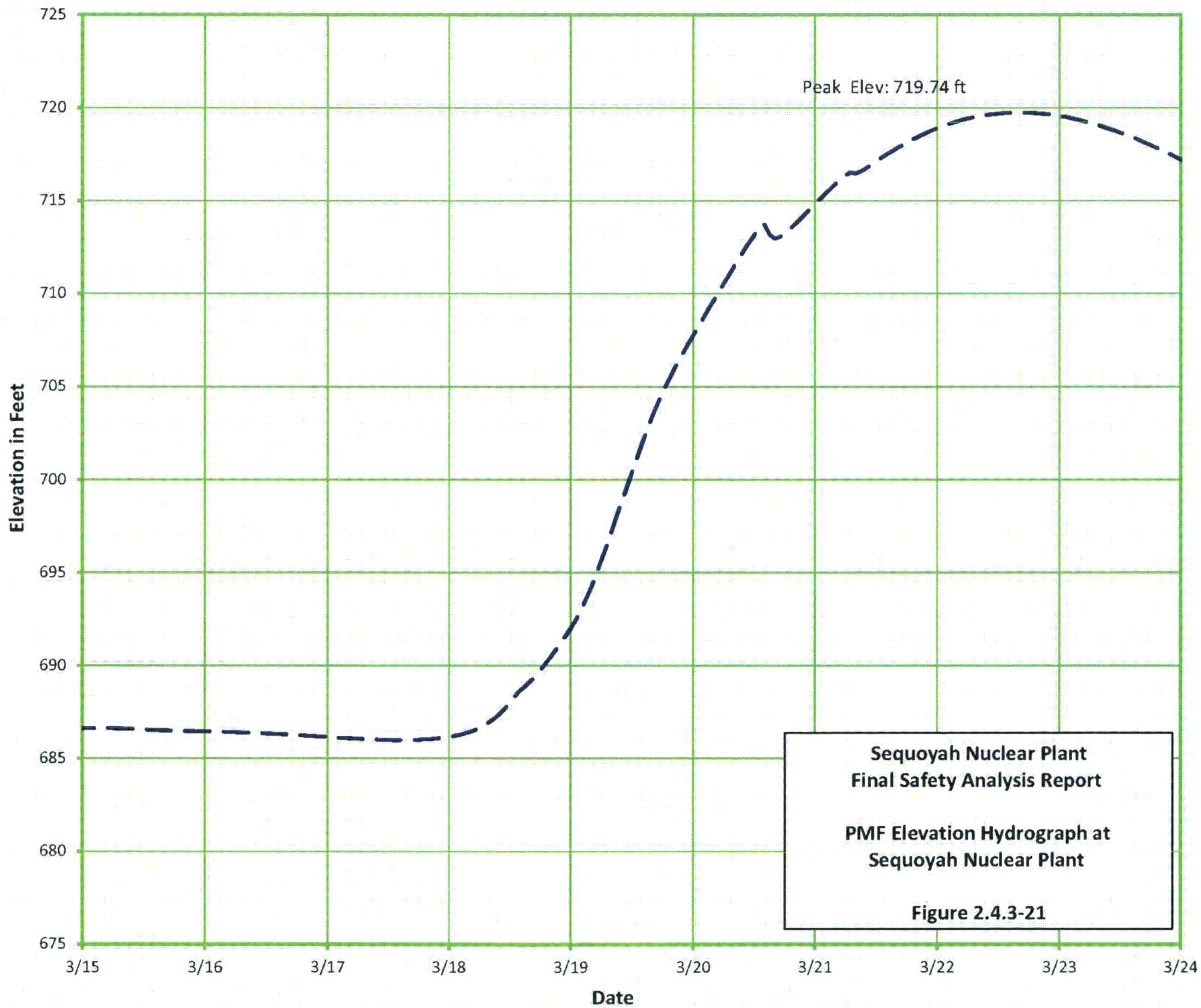


Figure 2.4.3-21 PMF Elevation Hydrograph at Sequoyah Nuclear Plant

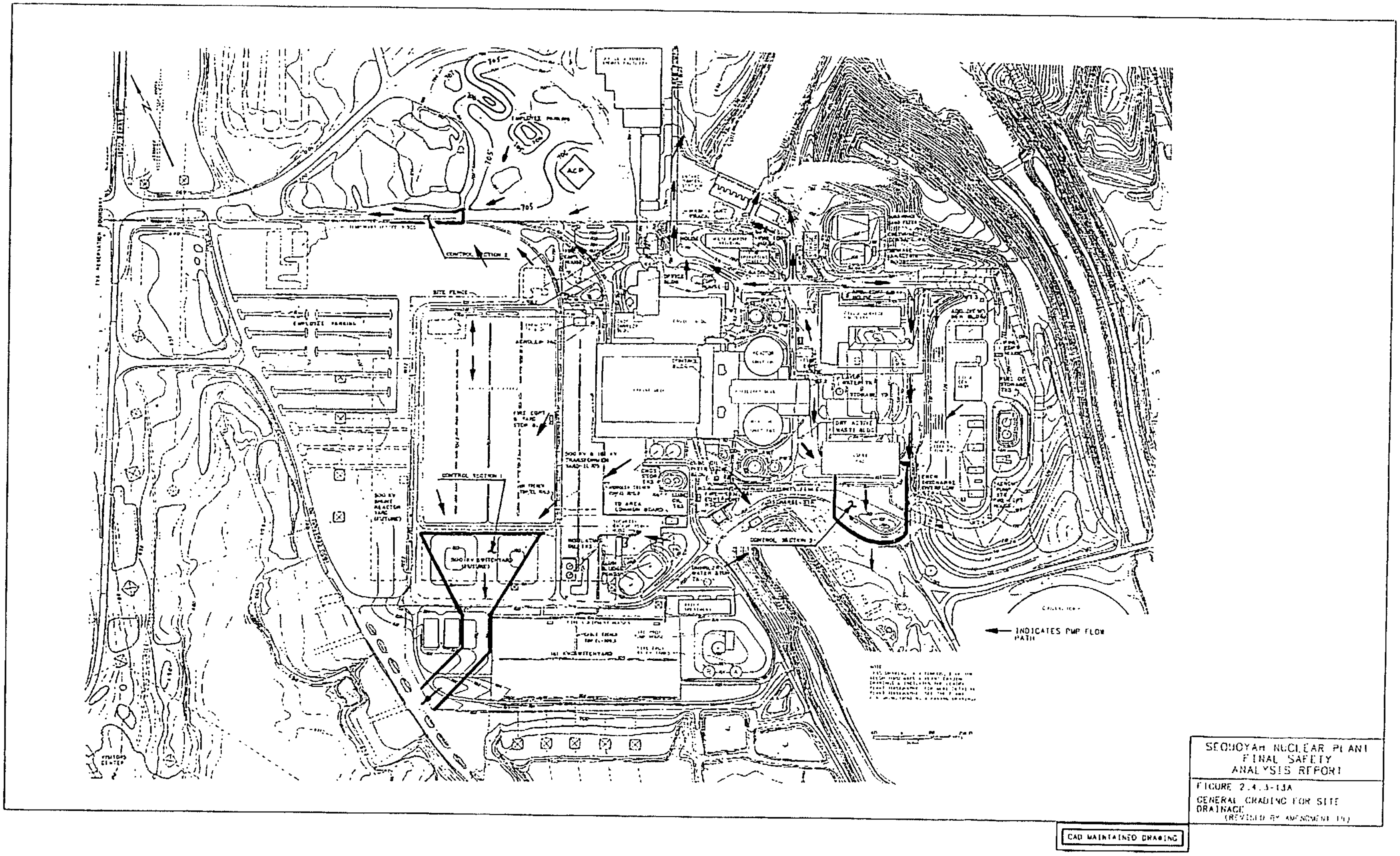


Figure 2.4.3-22 General Grading for Site Drainage

Figure 2.4.3-23 Not Used

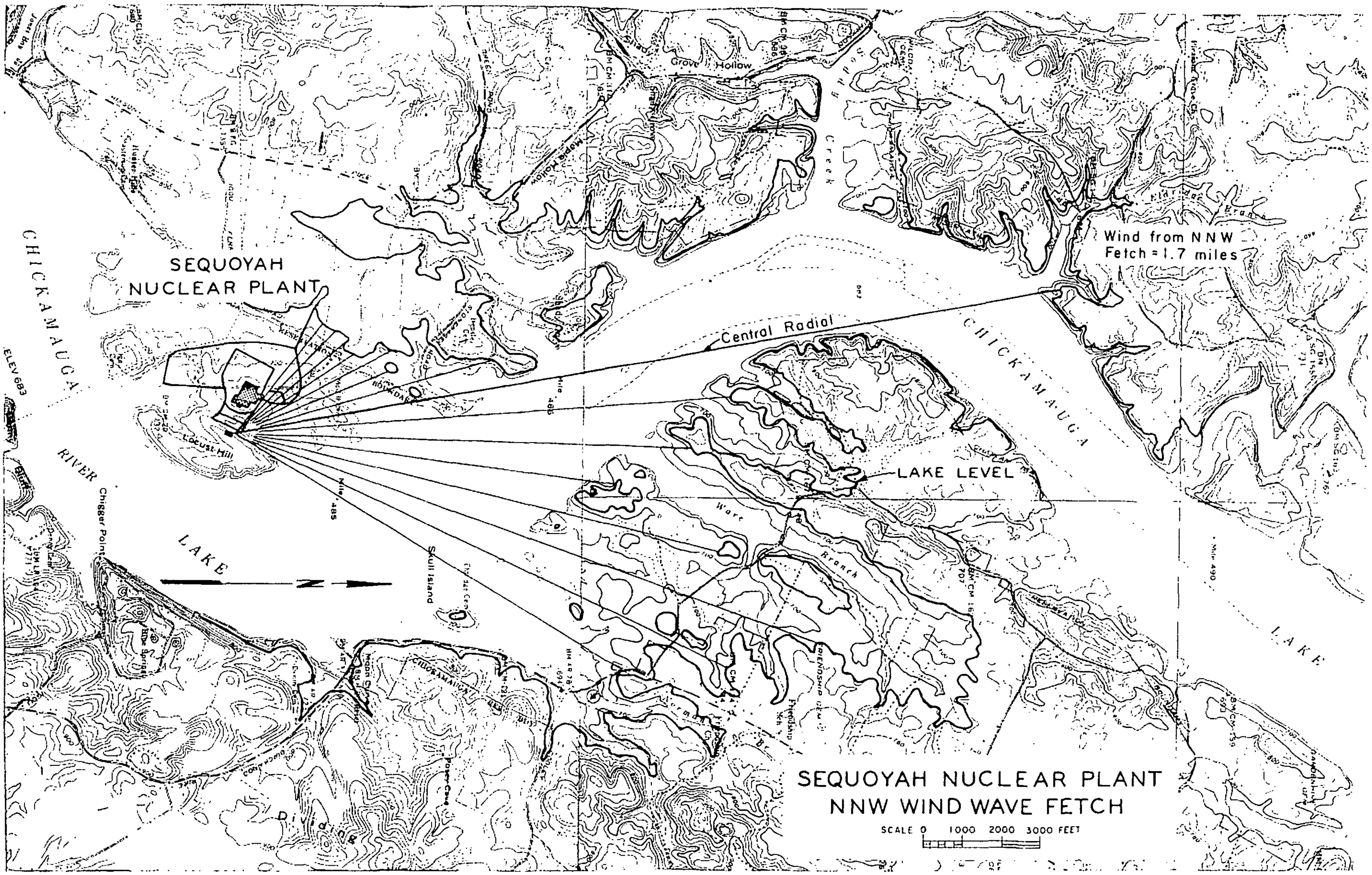


Figure 2.4.3-24 Sequoyah Nuclear Plant NNW Wind Wave Fetch

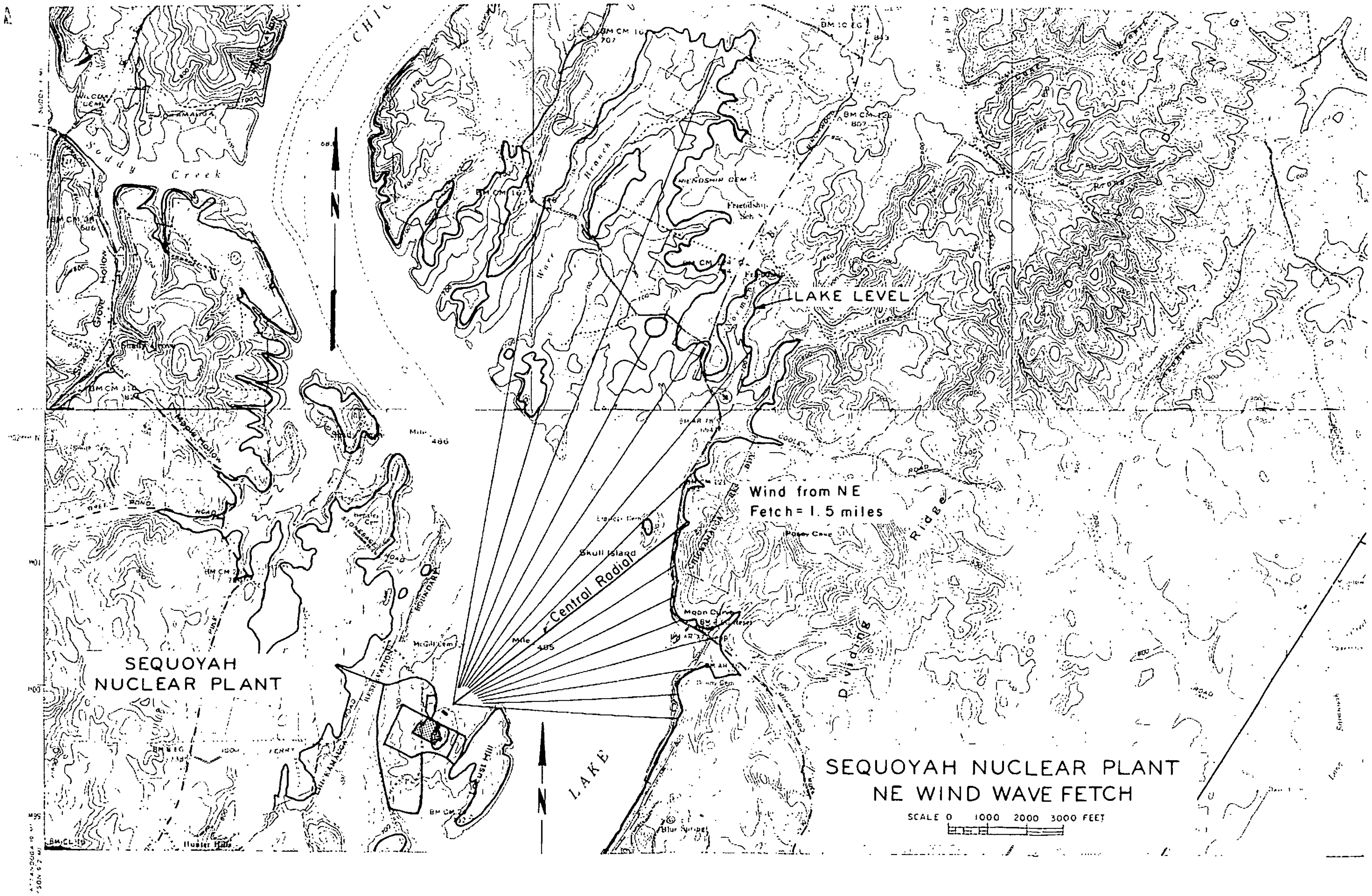


Figure 2.4.3-25 Sequoyah Nuclear Plant NE Wind Wave Fetch

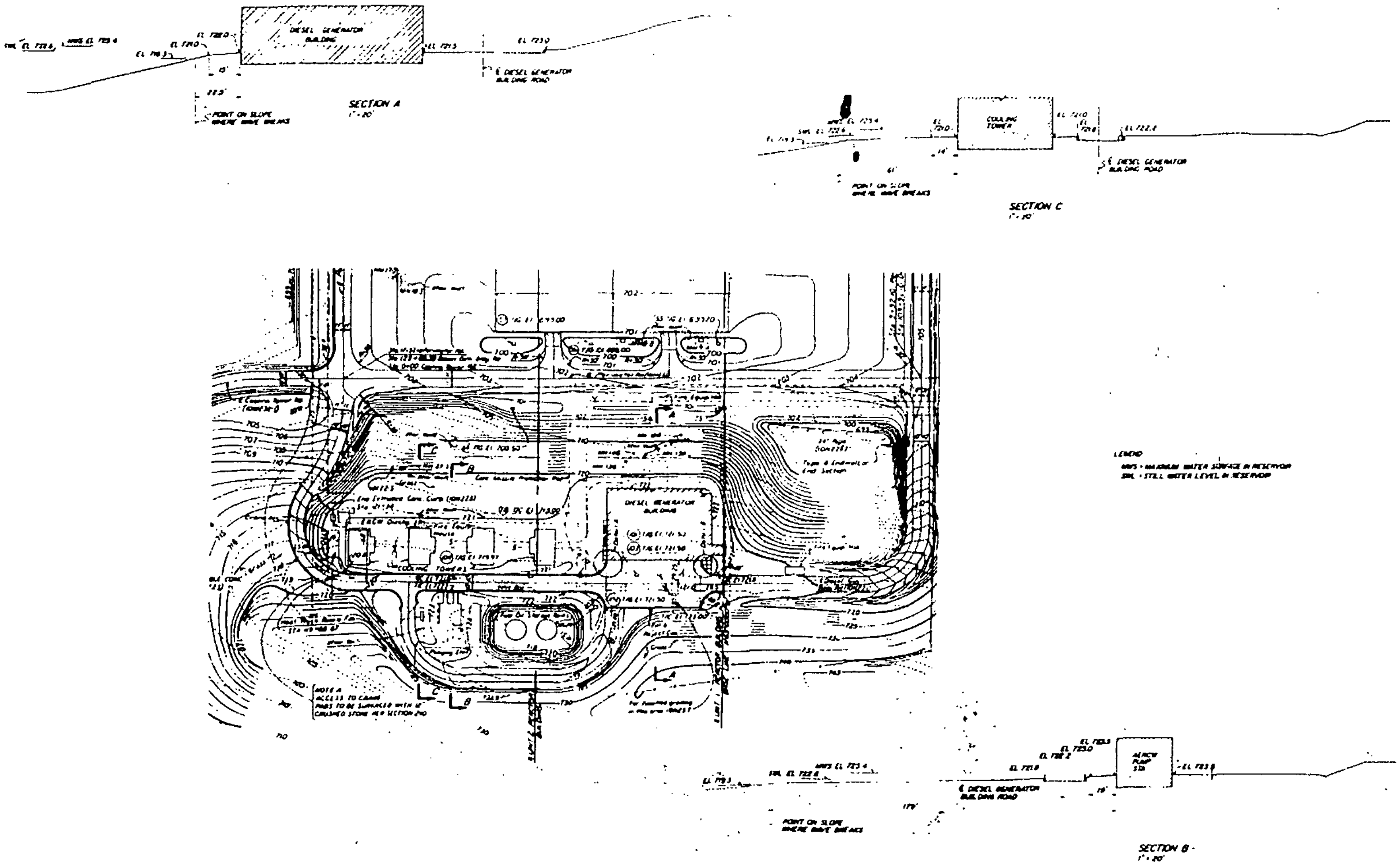
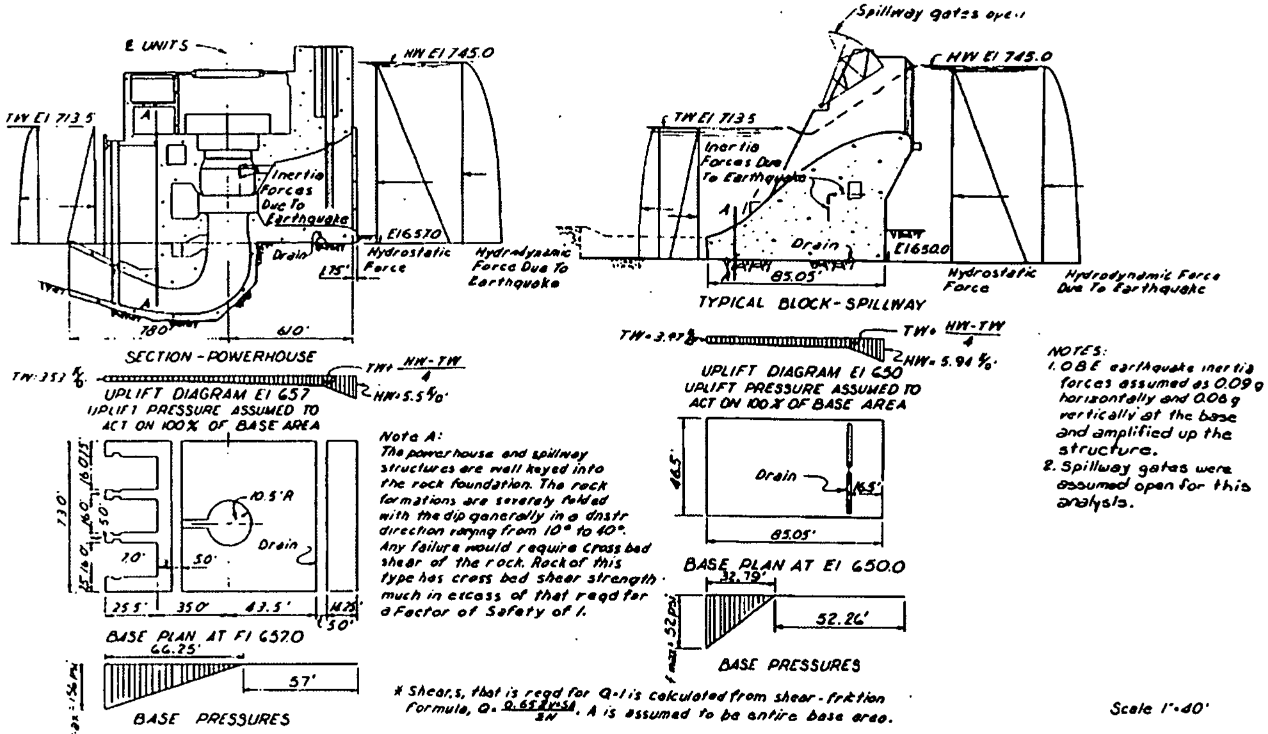


Figure 2.4.3-26 Topography Surrounding Diesel Generator Bldg and Cooling Towers



ΣV	ΣH	ΣH/ΣV	Avg Shear, S	S Reqd For Q=1	f max	FS ΣMR/ΣMO	Vertical Shear on Plane A-A
17,201	26,232	0.96	27.6 psi (entire base)	Note A 9 psi * (21.5 psi)**	156 psi	1.21	47.2 psi

ΣV	ΣH	ΣH/ΣV	Avg. Shear, S	S Reqd for Q=1	f max	FS ΣMR/ΣMO	Vertical Shear on Plane A-A
5686K	13,493K	2.37	24 psi	Note A 17.2 psi * (44.7 psi)**	52 psi	1.04	29 psi

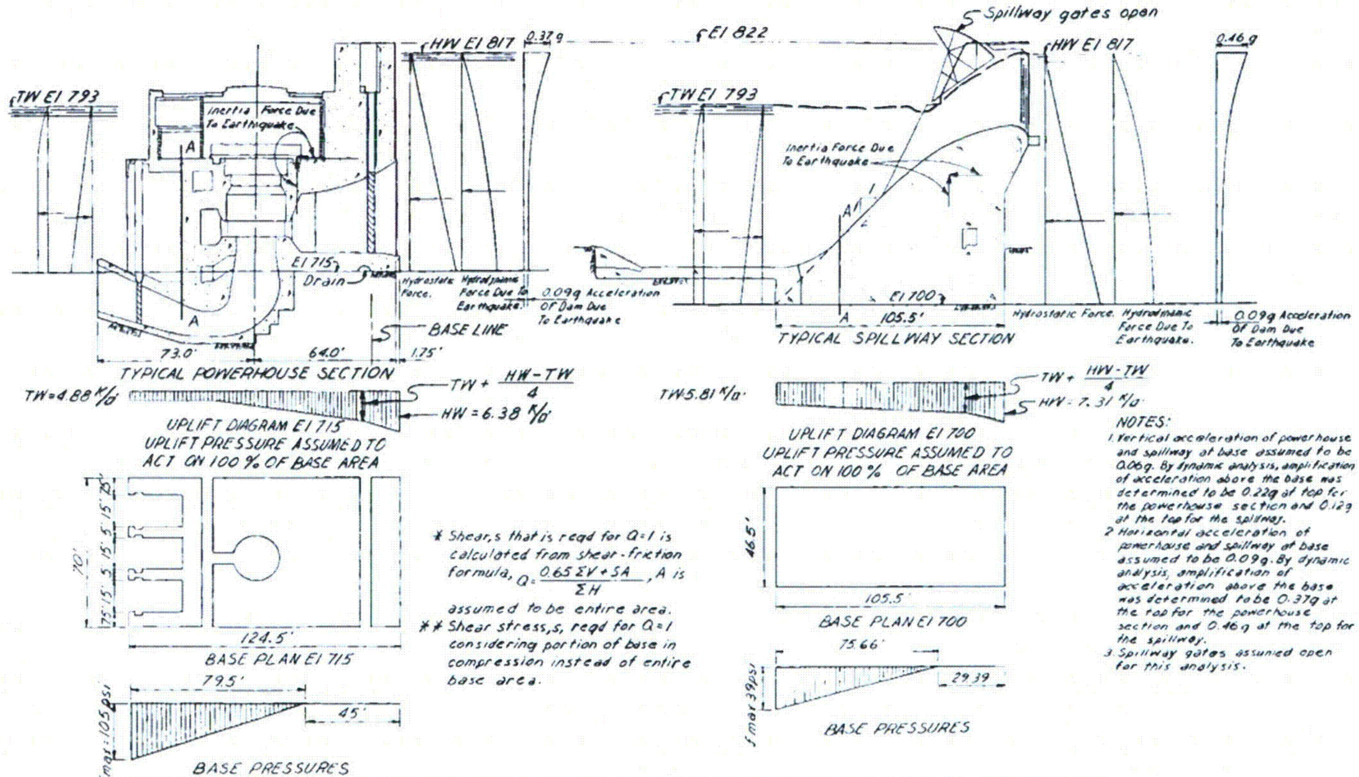
** Shear, S, reqd for Q=1 considering portion of base in compression (no tension), instead of entire base area.

SEQUOYAH NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Results of Analysis For Operating Basis Earthquake- Watts Bar Dam

Figure 2.4.4-1

Figure 2.4.4-1 Powerhouse & Spillway Results of Ananlysis For Operating Basis Earthquake - Watts Bar Dam



ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg shear	S reqd for Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical shear on plane A-A
22,762K	22,541K	0.99	24.2psi (entire base)	8.3psi * (14.6psi) **	105psi	1.18	26psi

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg shear	S reqd for Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical shear on plane A-A
19,488K	14,382K	0.75	20psi (entire base)	2.38psi * (3.30psi) **	39psi	1.26	17psi

SEQUOYAH NUCLEAR PLANT
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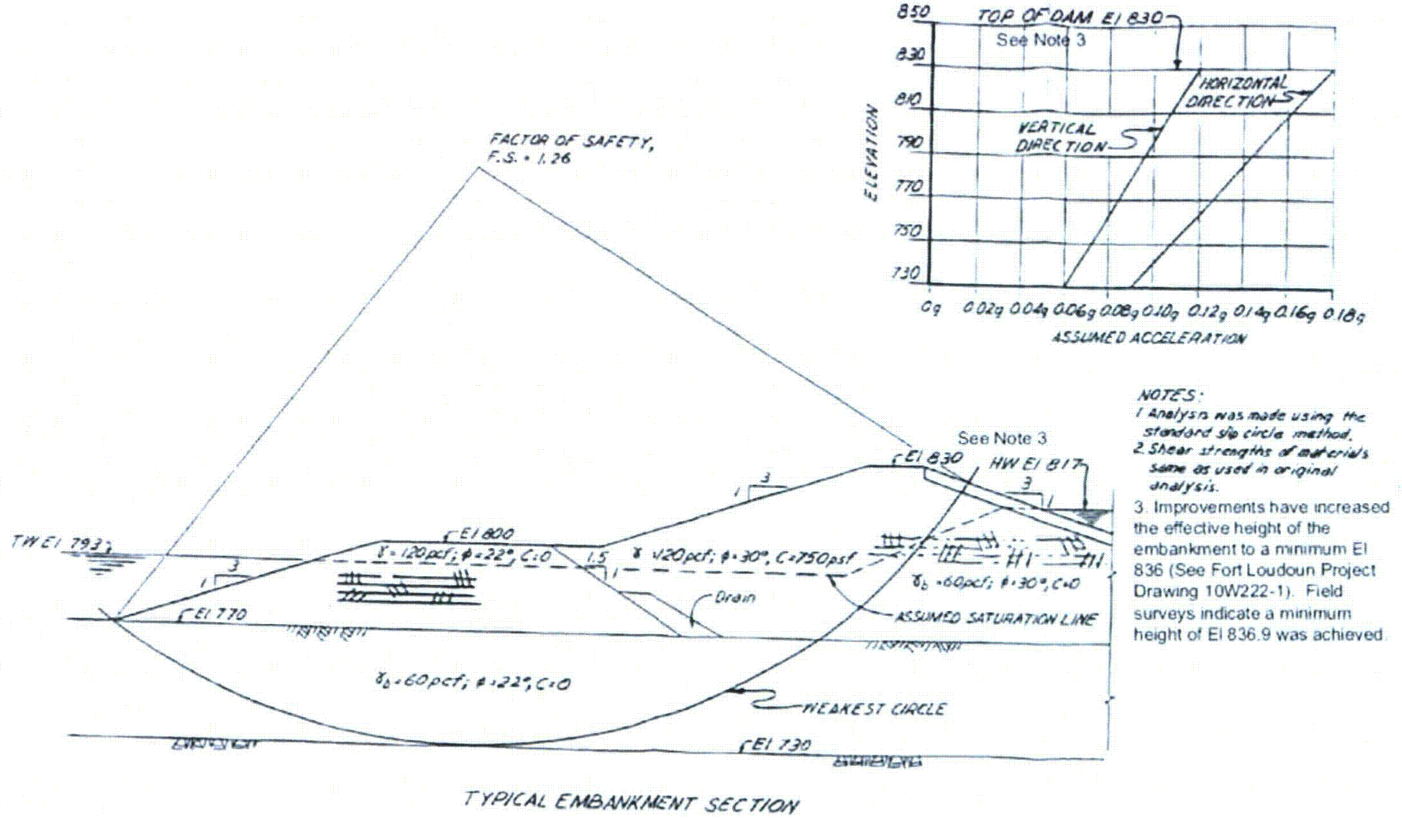
Results of Analysis For Operating
Basis Earthquake- Fort Loudoun Dam

Figure 2.4.4-2

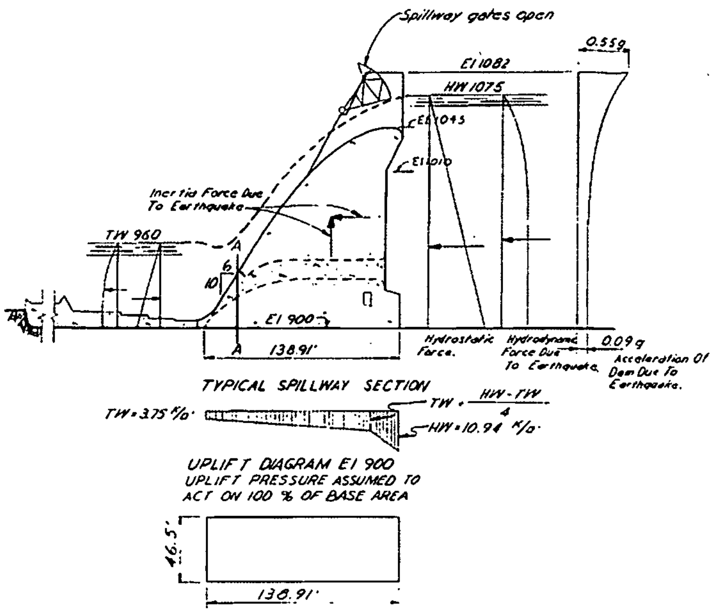
Figure 2.4.4-2 Powerhouse & Spillway Results of Analysis For Operating Basis Earthquake - Fort Loudoun Dam

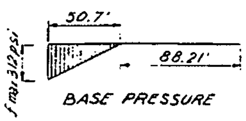


SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Embankment Results For Operating Basis Earthquake, Fort Loudoun Dam Figure 2.4.4-3

Figure 2.4.4-3 Embankment Results Of Analysis For Operating Basis Earthquake - Fort Loudoun Dam

Figure 2.4.4-4 Analysis For OBE & 1/2 PMF Assumed Condition of Dam After Failure of Norris Dam





* Shear, s, that is reqd for Q=1 is calculated from shear-friction formula, $Q = \frac{0.65\Sigma V + SA}{\Sigma H}$, A is assumed to be entire area.

** Shear stress, s, reqd for Q=1 considering portion of base in compression instead of entire base area.

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, S	S Reqd For Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on Plane A-A
53,007K	57,216K	1.08	61 psi (entire base)	25psi * (67psi)**	312psi	1.13	173 psi

NOTES:

1. Vertical acceleration of the spillway at the base assumed to be 0.06g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.11g at the top.
2. Horizontal acceleration of the spillway at the base assumed to be 0.09g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.55g at the top.
3. Spillway gates assumed open for this analysis.

SEQUOYAH NUCLEAR PLANT
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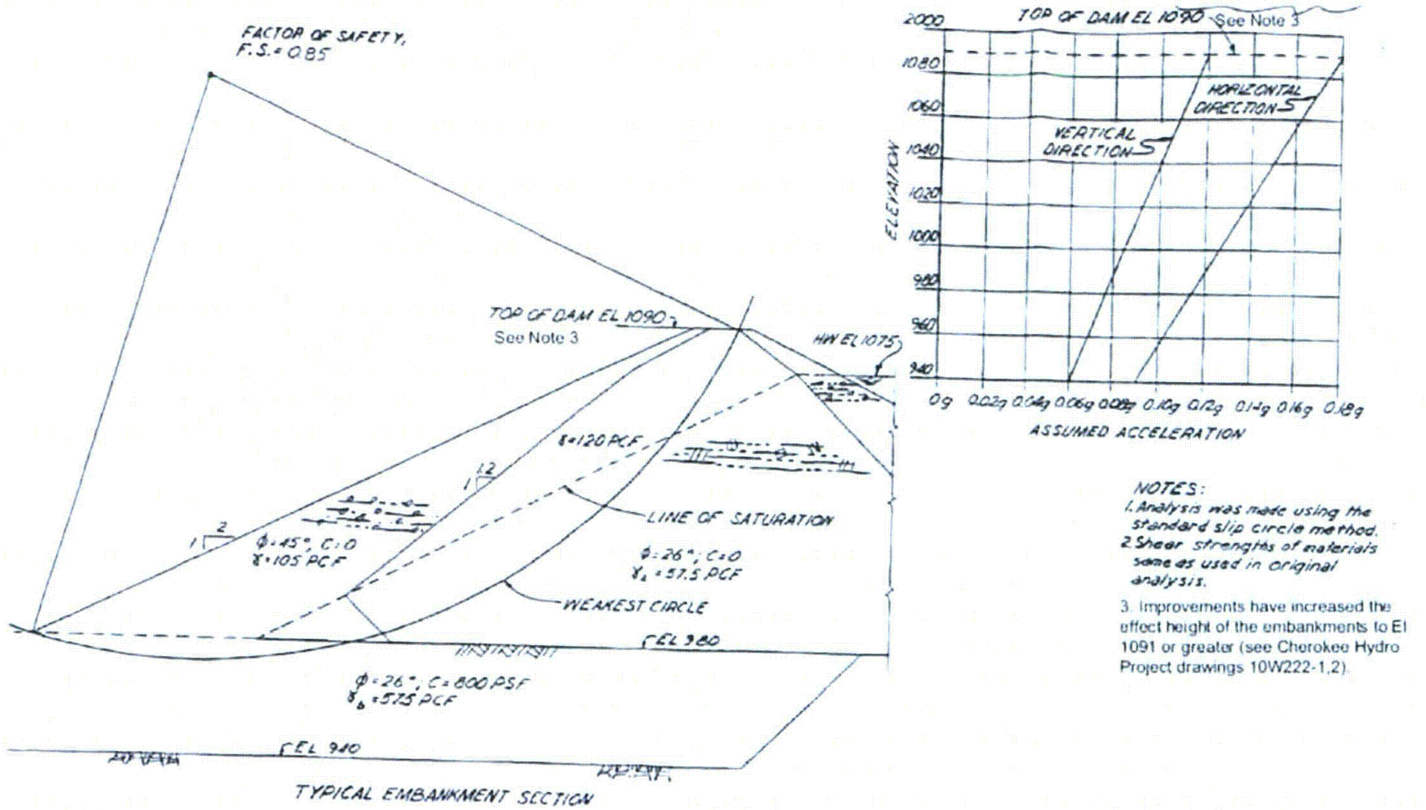
ANALYSIS REPORT

Spillway & Nonoverflow Results Of Analysis For OBE, Cherokee Dam

Figure 2.4.4-5

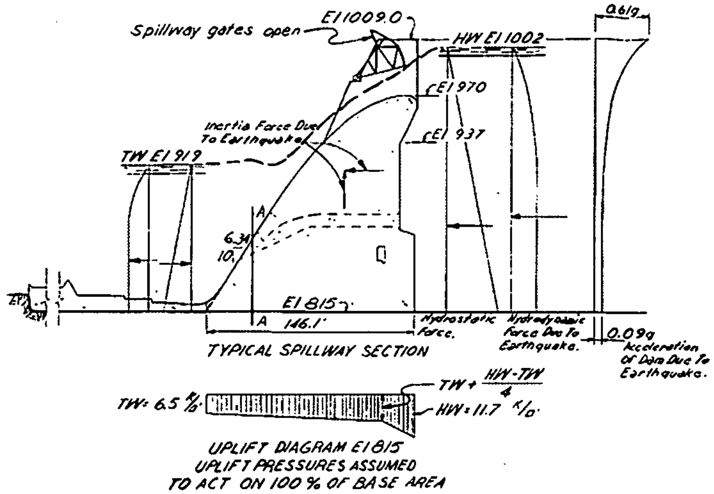
Figure 2.4.4-5 Spillway & Nonoverflow Results of Analysis For Operating Basis Earthquake - Cherokee Dam

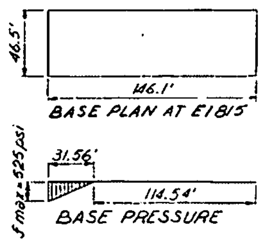


SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Embankment Results Of Analysis For OBE, Cherokee Dam Figure 2.4.4-6

Figure 2.4.4-6 Embankment Results of Analysis For Operating Basis Earthquake - Cherokee Dam

Figure 2.4.4-7 Assumed Condition Of Dam After Failure OBE & 1/2 Probable Max Flood - Cherokee Dam





* Shear, s, that is reqd for Q=1 is calculated from shear-friction formula, $Q = \frac{0.65\Sigma V + SA}{\Sigma H}$. A is assumed to be entire area.

** Shear stress, s, reqd for Q=1 considering portion of base in compression instead of entire base area.

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, S	S Reqd For Q=1	f max	$F.S. = \frac{\Sigma MR}{\Sigma Mo}$	Vertical Shear on Plane AA
55,403K	60,245K	1.09	61.6psi (entire base)	25psi* (170psi)**	525psi	1.06	156psi

NOTES:

1. Vertical acceleration of the spillway at the base assumed to be 0.06 g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.13 g at the top.
2. Horizontal acceleration of the spillway at the base assumed to be 0.09g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.61g at the top.
3. Spillway gates assumed open for this analysis.

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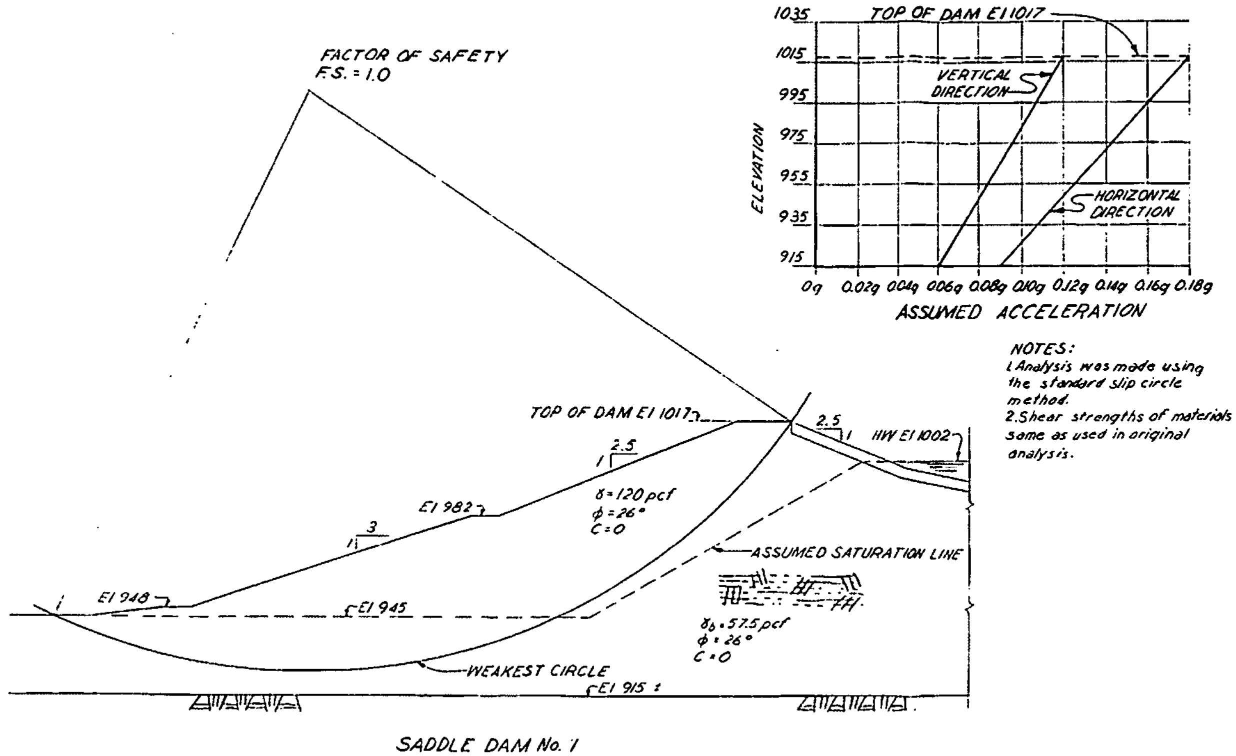
Spillway & Nonoverflow Results Of Analysis For OBE, Douglas Dam

Figure 2.4.4-8

Figure 2.4.4-8 Spillway & Nonoverflow Results of Analysis For Operating Basis Earthquake - Douglas Dam



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Saddle Dam No. 1 Results Of Analysis For OBE, Douglas Dam Figure 2.4.4-9

Figure 2.4.4-9 Saddle Dam No. 1 Results of Analysis For Operating Basis Earthquake - Douglas Dam

Figure 2.4.4-10 Douglas Dam Assumed Condition of Dam After Failure OBE & 1/2 Probable Maximum Flood - Douglas Project

Figure 2.4.4-11 Fontana Dam Assumed Condition of Dam After Failure OBE & 1/2 PMF

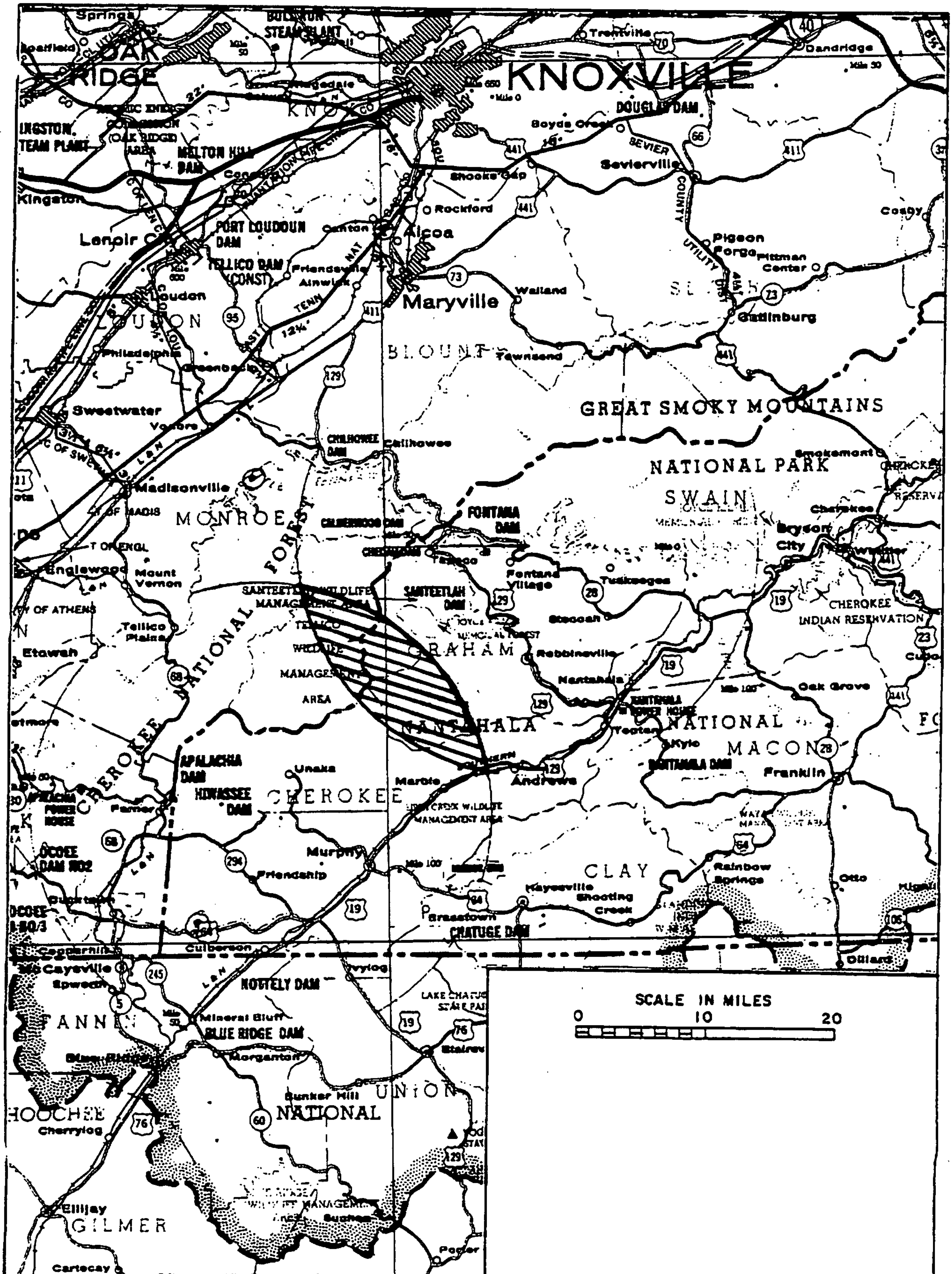
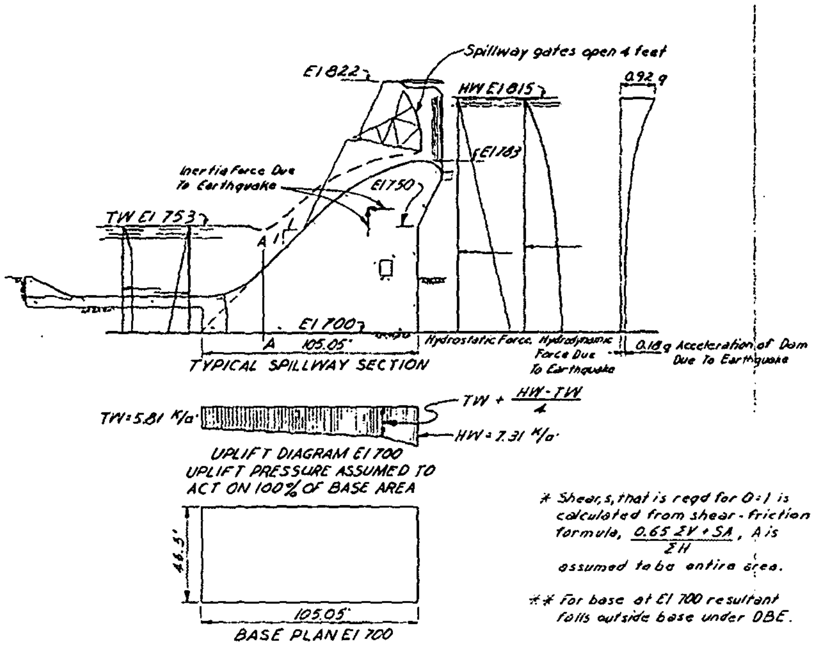


Figure 2.4.4-12 OBE with Epicenter Within Area Shown



NOTES:

1. Vertical acceleration of spillw at base assumed to be 0.12g By dynamic analysis, amplification of acceleration above the base was determined to be 0.24g at to
2. Horizontal acceleration of spillway at base assumed to be 0.18g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.92g at th
3. Spillway gates assumed ope 4 feet for this analysis.

BASE PRESSURE**

ΣV	ΣH	ΣH / ΣV	Avg shear	S reqd for Q=1	f max	FS ΣMR / ΣMo
19,254K	29,534K	1.62	42 psi entire base	25psi*	* *	0.9

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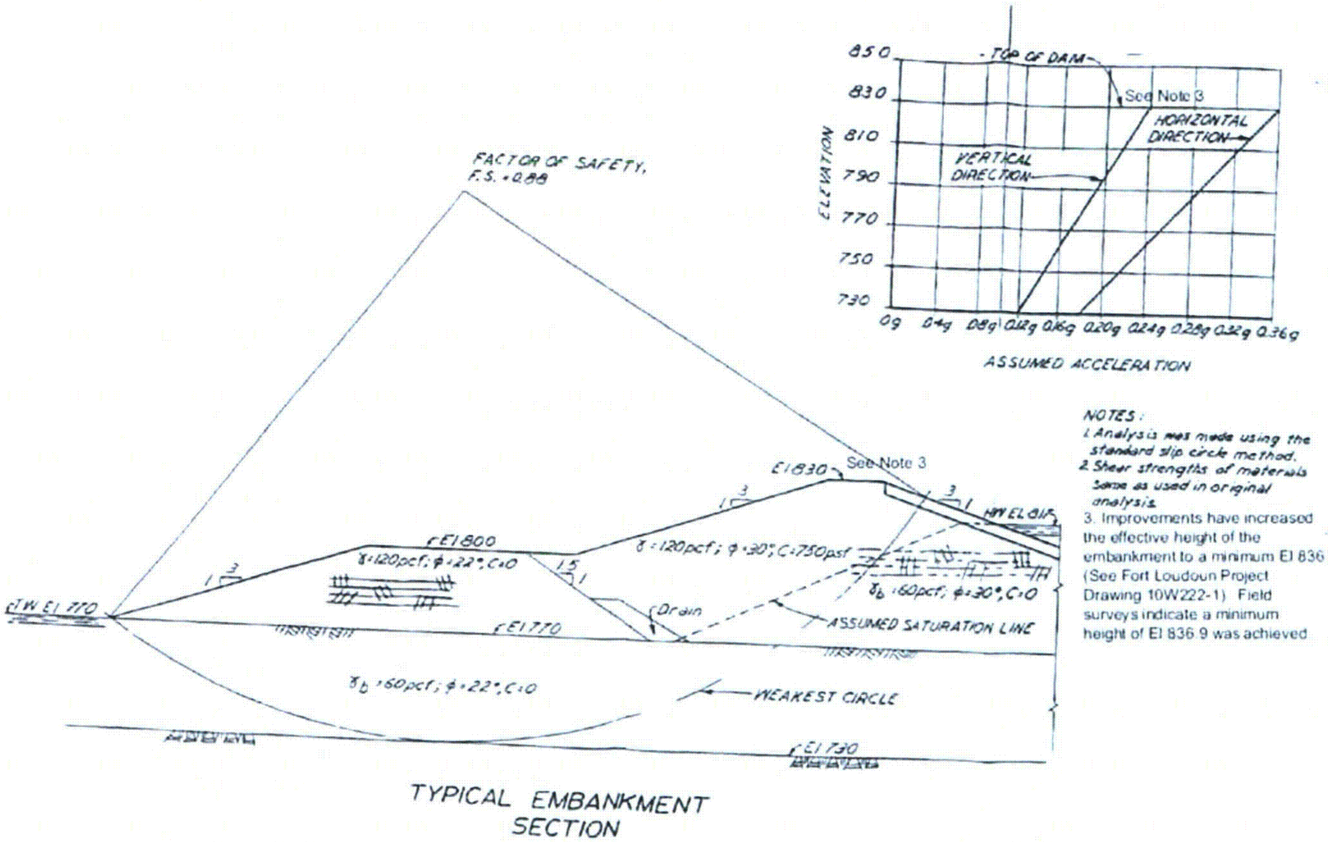
Spillway Results Of Analysis For
SSE Earthquake. Fort Loudoun Dam

Figure 2.4.4-13

Figure 2.4.4-13 Spillway Results of Analysis For SSE Earthquake Fort Loudoun Dam



SEQUOYAH NUCLEAR PLANT
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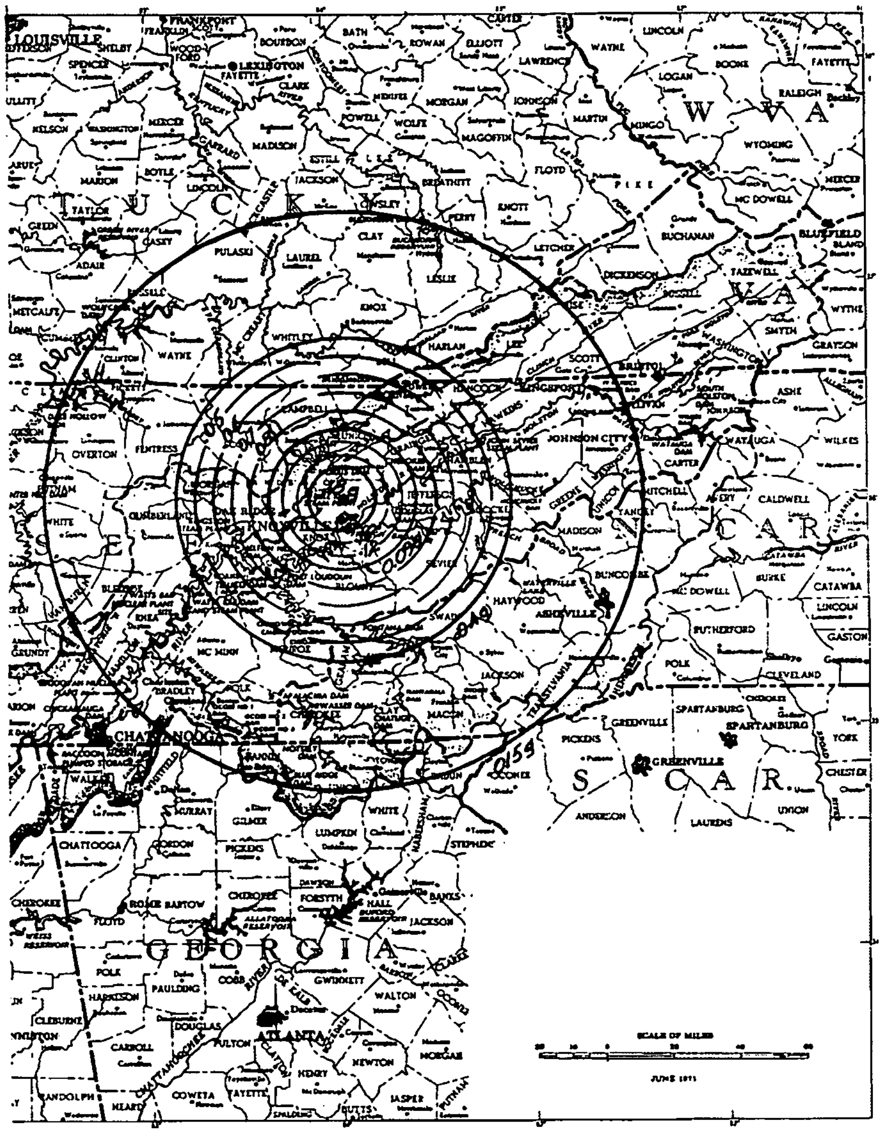
Embankment Results Of Analysis For SSE Earthquake, Fort Loudoun Dam

Figure 2.4.4-14

Figure 2.4.4-14 Embankment Results of Analysis For SSE Earthquake Fort Loudoun Dam

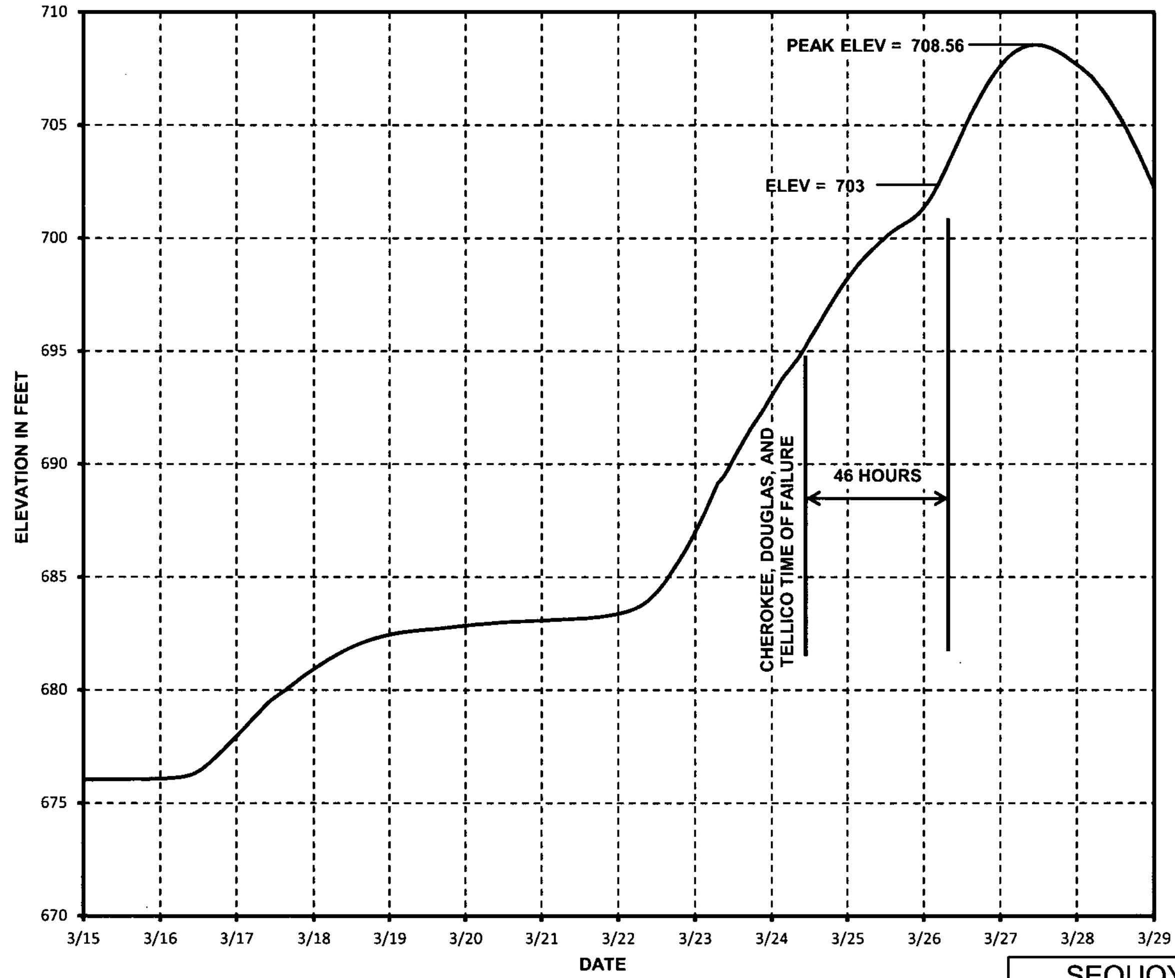
Figure 2.4.4-15 Fort Loudoun Dam Assumed Condition of Dam After Failure SSE Combined with a 25 Year Flood - Fort Loudoun

Figure 2.4.4-16 Norris Dam SSE & 25 Year Flood Judged Condition of Dam After Failure - Norris Dam



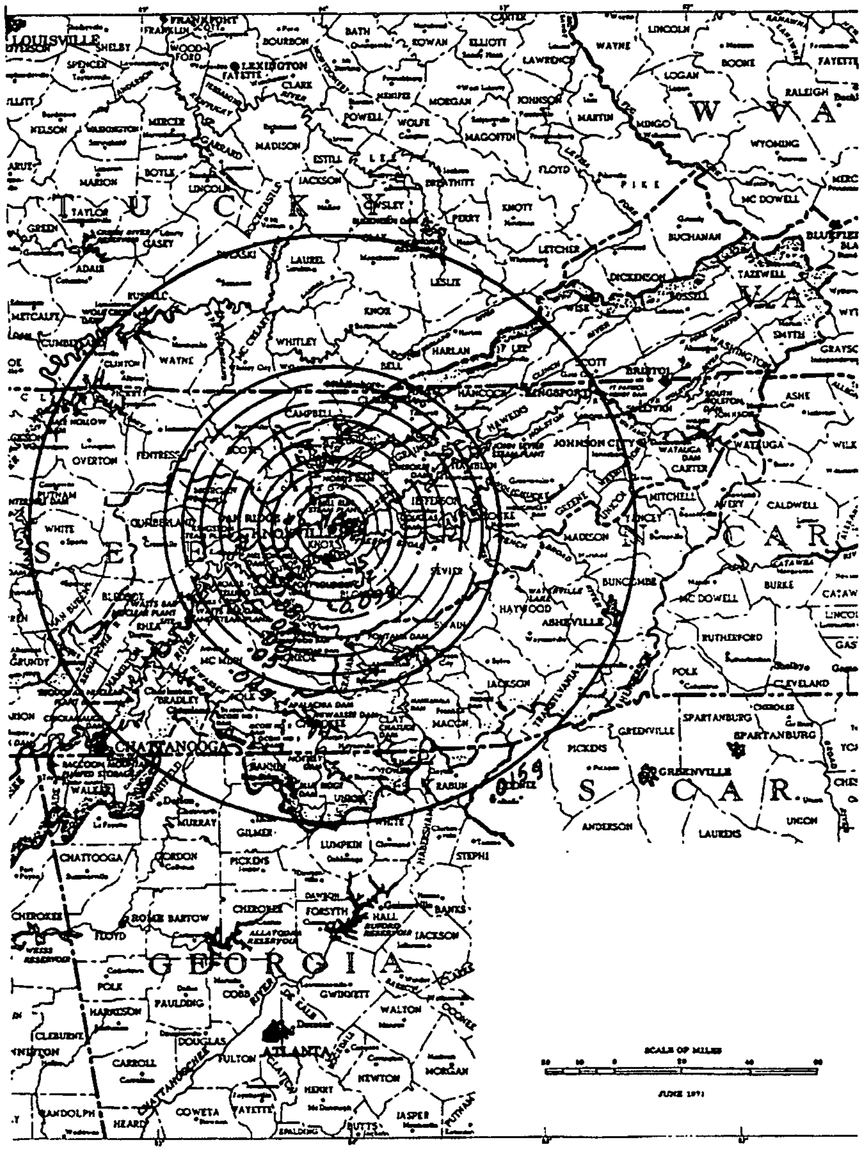
SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
SSE With Epicenter In North Knoxville Vicinity Figure 2.4.4-17

Figure 2.4.4-17 SSE With Epicenter in North Knoxville Vicinity



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seismic Combinatinon With Failure Of Cherokee, Douglas & Tellico In OBE with1/2 PMF Figure 2.4.4-18

Figure 2.4.4-18 OBE Failure Of Cherokee, Douglas & Tellico With 1/2 PMF



SEQUOYAH NUCLEAR PLANT
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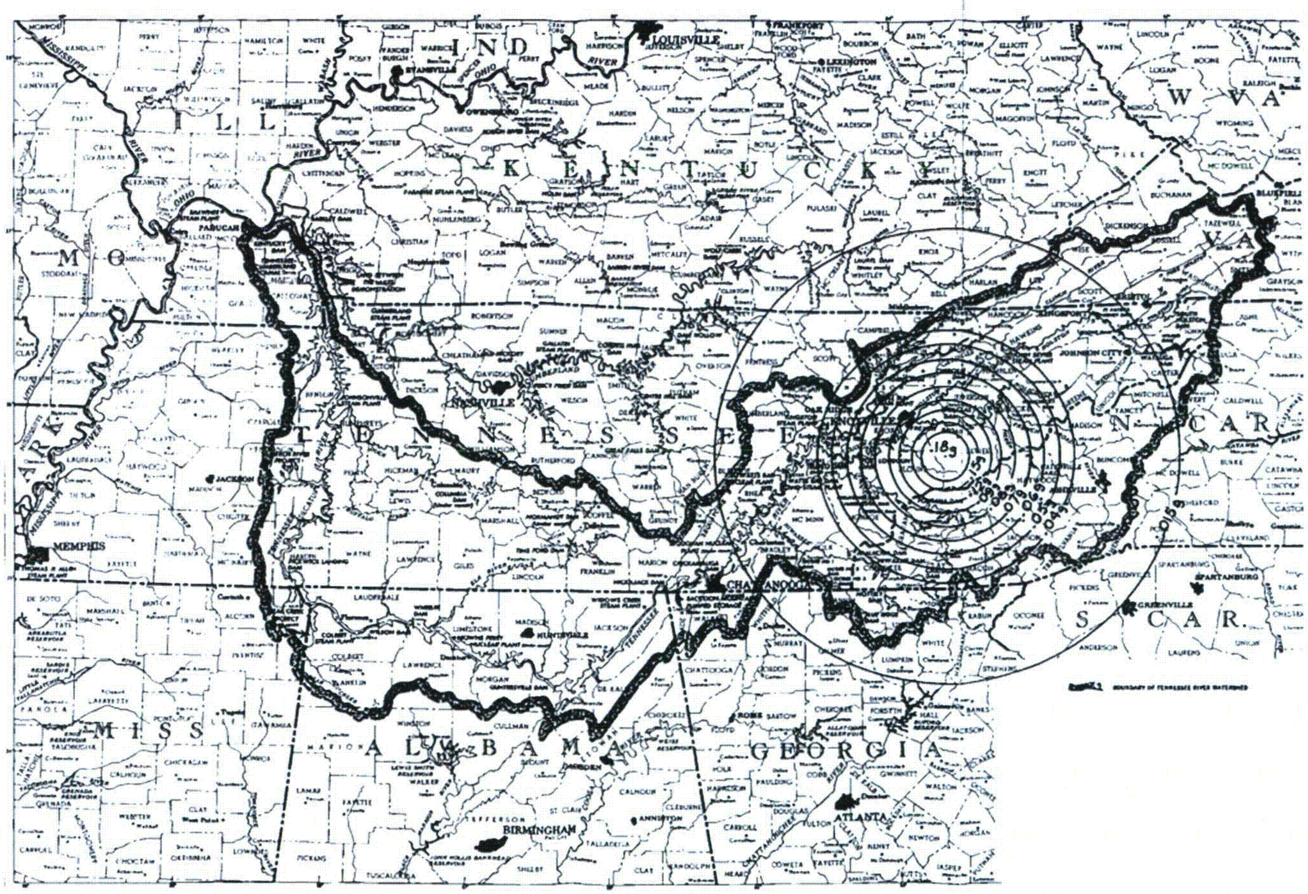
SSE With Epicenter In
West Knoxville Vicinity

Figure 2.4.4-19

Figure 2.4.4-19 SSE With Epicenter In West Knoxville Vicinity

Figure 2.4.4-20 Tellico Dam Assumed Condition of Dam After Failure SSE Combined With a 25 Year Flood Tellico Project

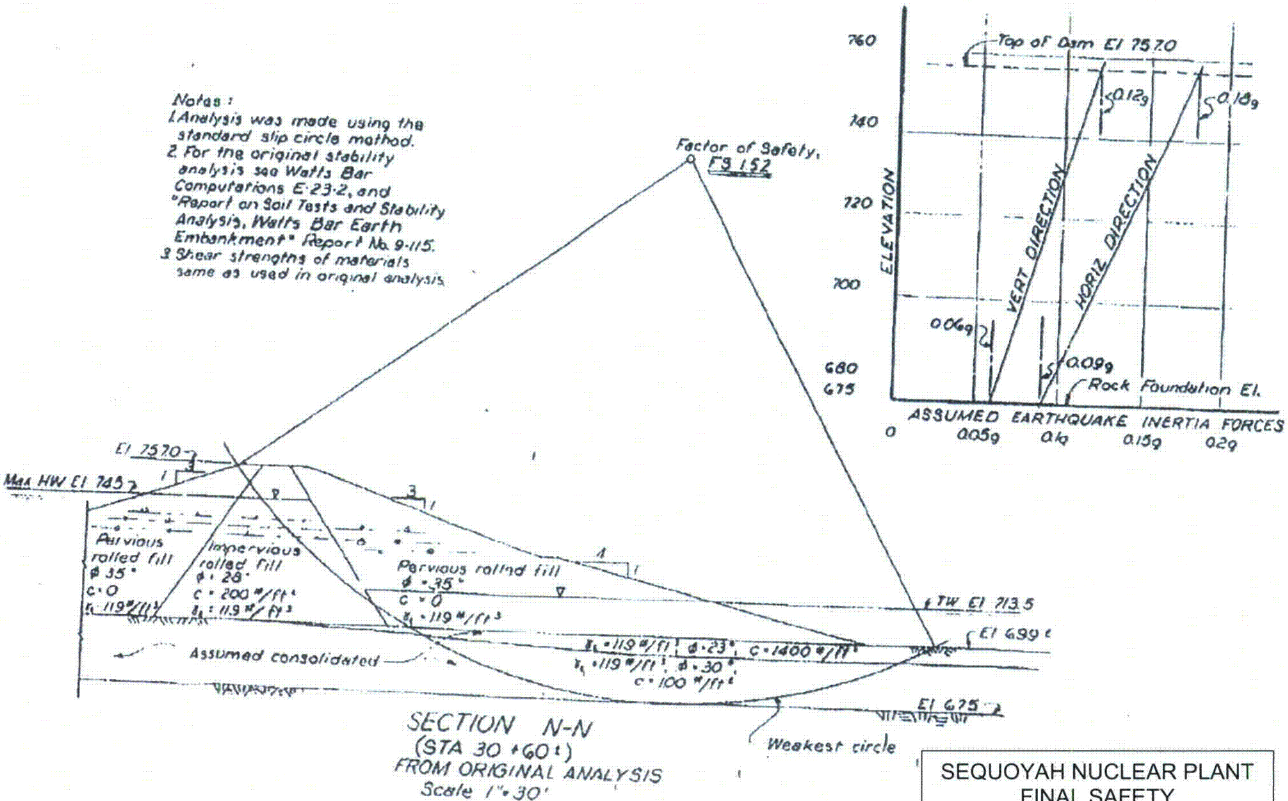


SEQUOYAH NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Location Of The SSE For Simultaneous Failure Of The Douglas & Fontana Dams

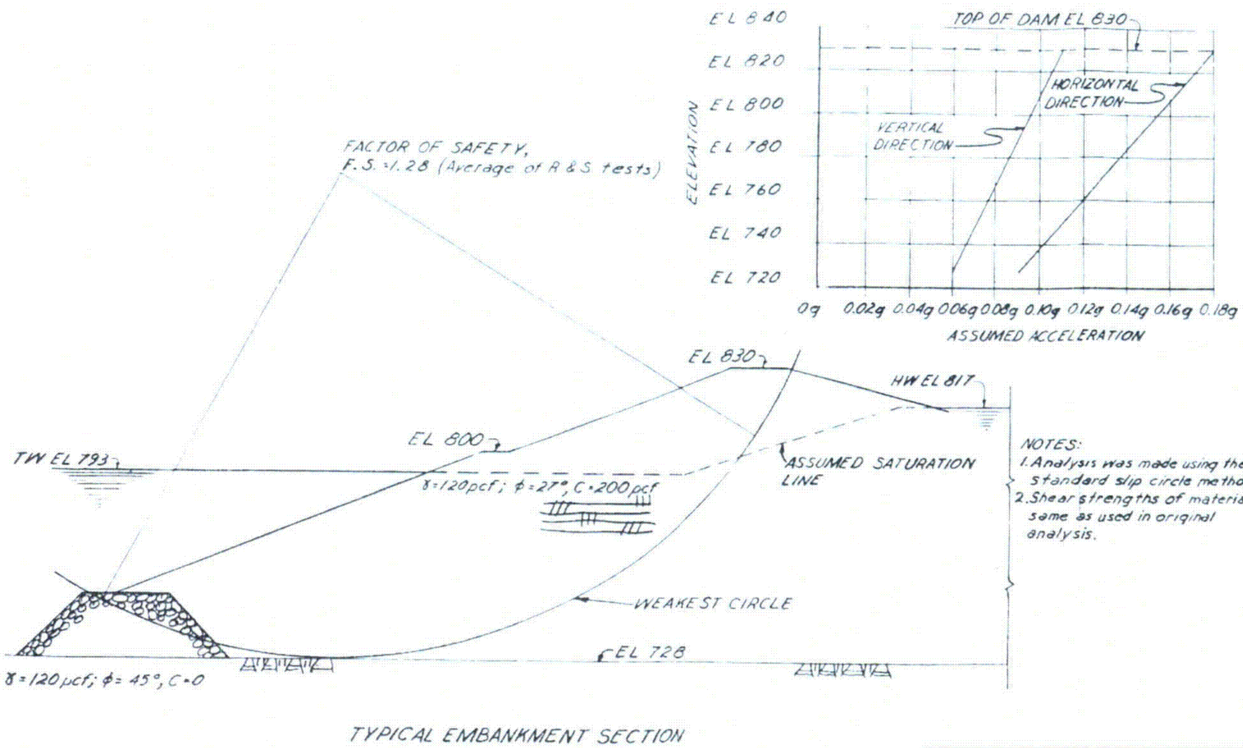
Figure 2.4.4-21

Figure 2.4.4-21 Location Of SSE For Simultaneous Failure Of The Douglas & Fontana Dams



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Embankment Watts Bar Dam, Results of Analysis for OBE Figure 2.4.4-22

Figure 2.4.4-22 Embankment Watts Bar Dam, Results of Analysis for OBE



SEQUOYAH NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Embankment Tellico Dam, Results of Analysis for OBE Figure 2.4.4-23

Figure 2.4.4-23 Embankment Tellico Dam, Results of Analysis for OBE

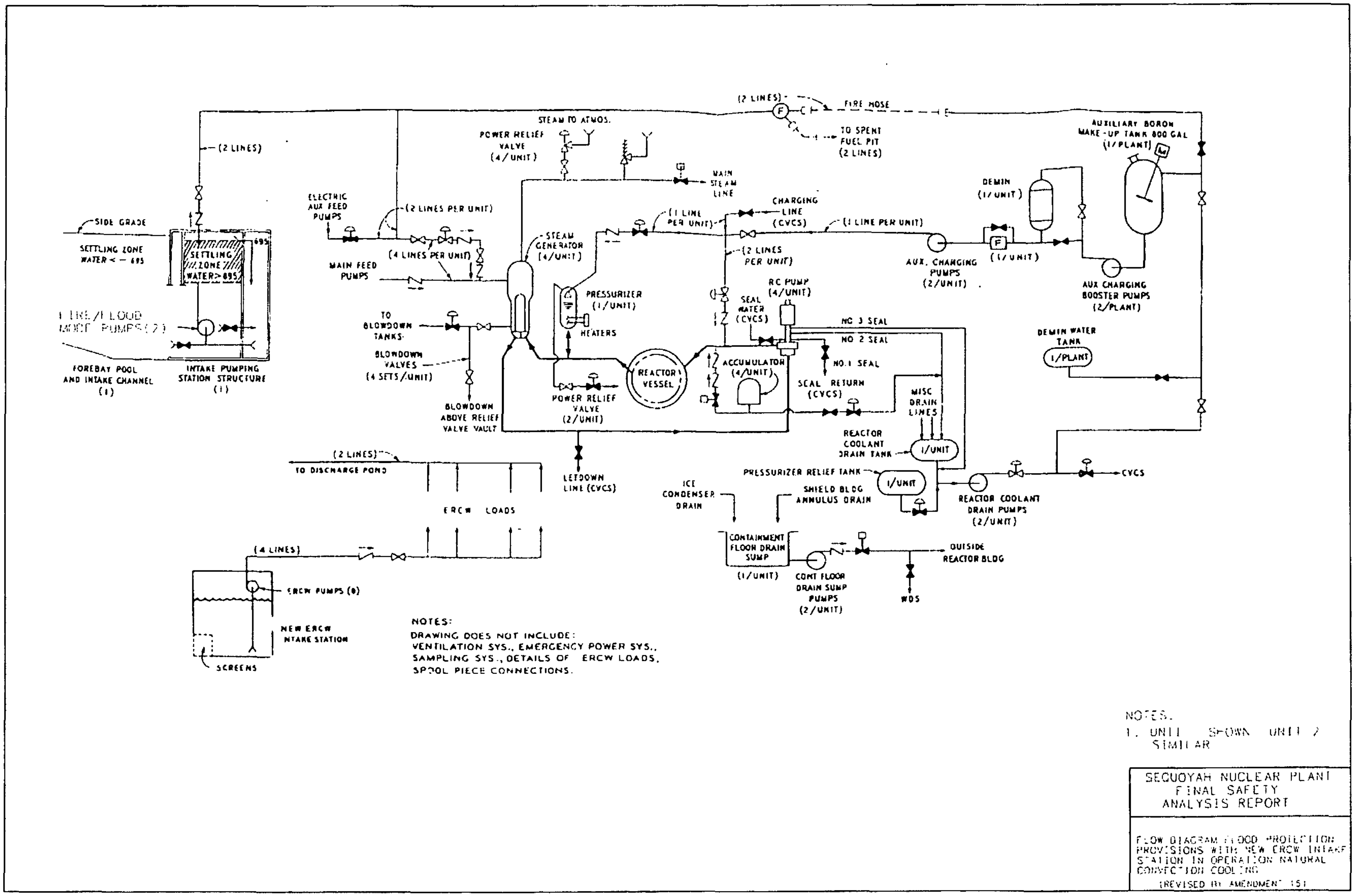


Figure 2.4.14-1 Flow Diagram - Flood Protection Provisions with New ERCW Intake Station in Operation - Natural Convection Cooling

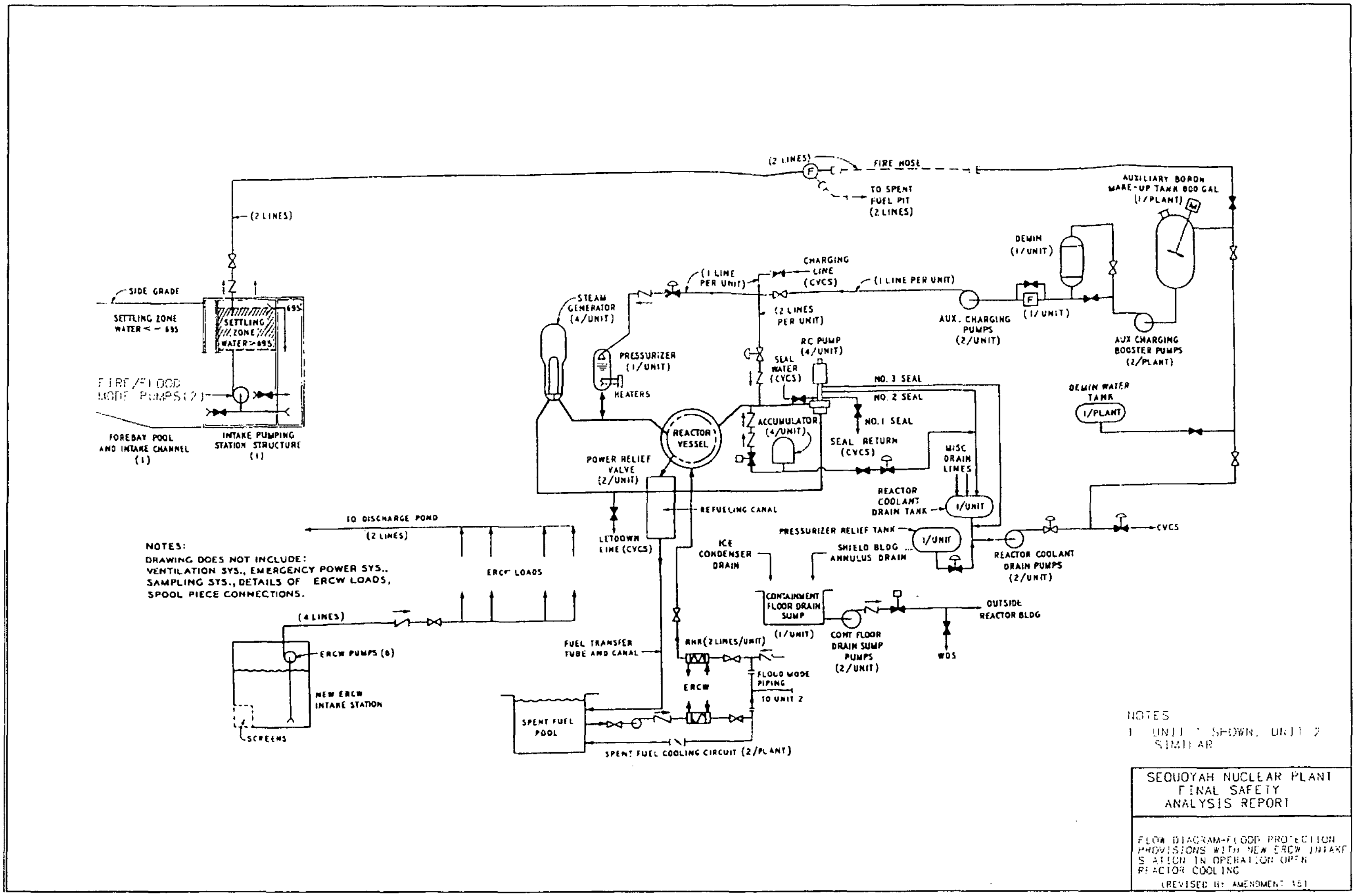
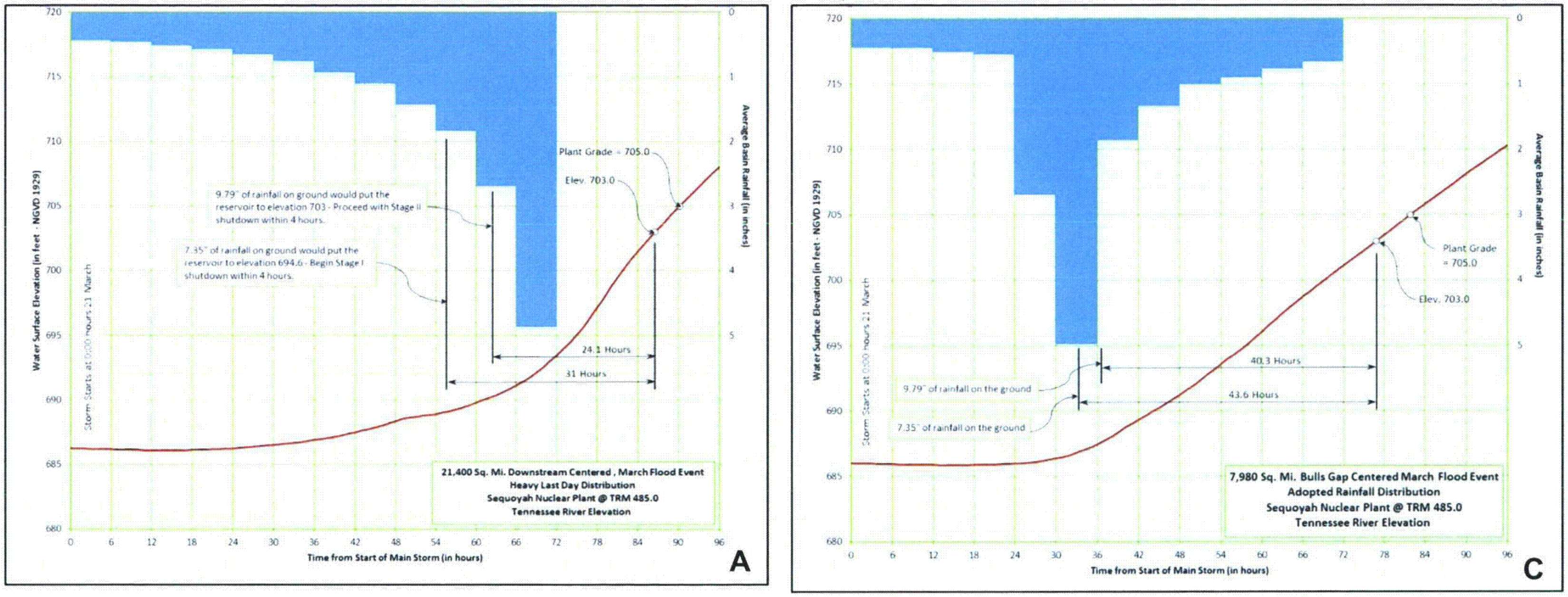


Figure 2.4.14-2 Flow Diagram - Flood Protection Provisions with New ERCW Intake Station in Operation - Open Reactor Cooling



NOTE: Times shown allow 4 hours for communications and forecast computations.

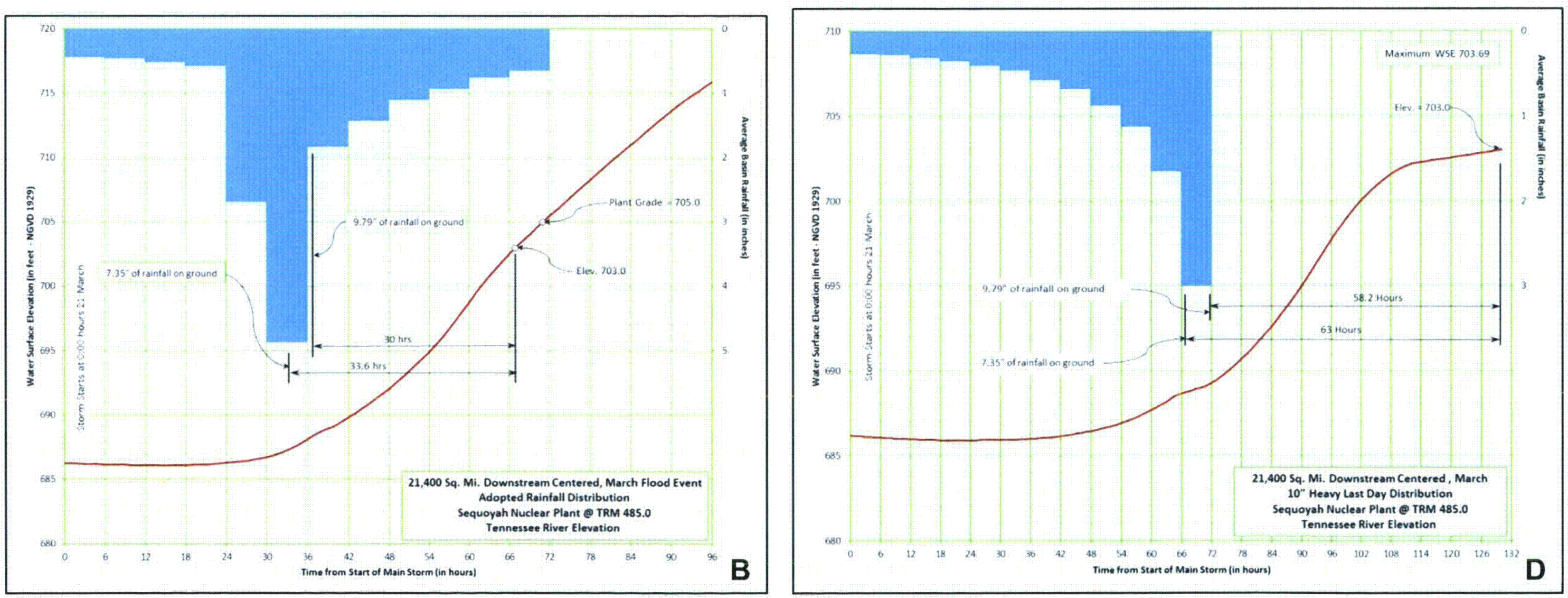
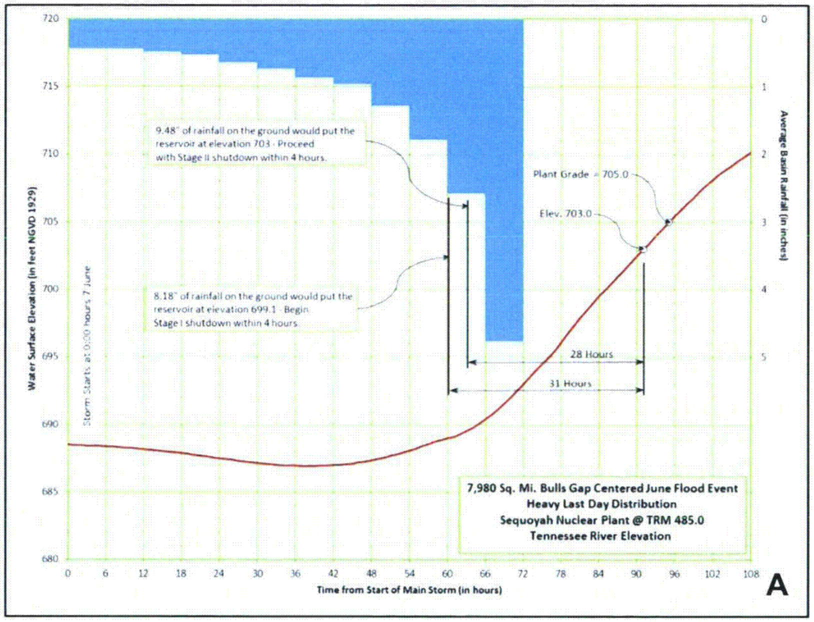
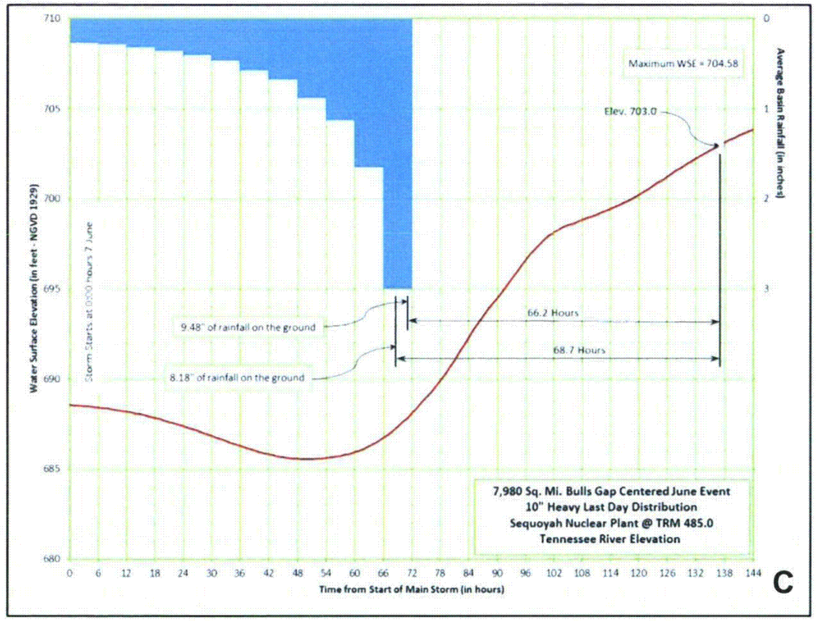


Figure 2.4.14-3 (Sheet 1 of 2) - Sequoyah Nuclear Plant Rainfall Flood Warning Time Basis for Safe Shutdown For Plant Flooding - Winter Events





NOTE: Times shown allow 4 hours for communications and forecast computations.

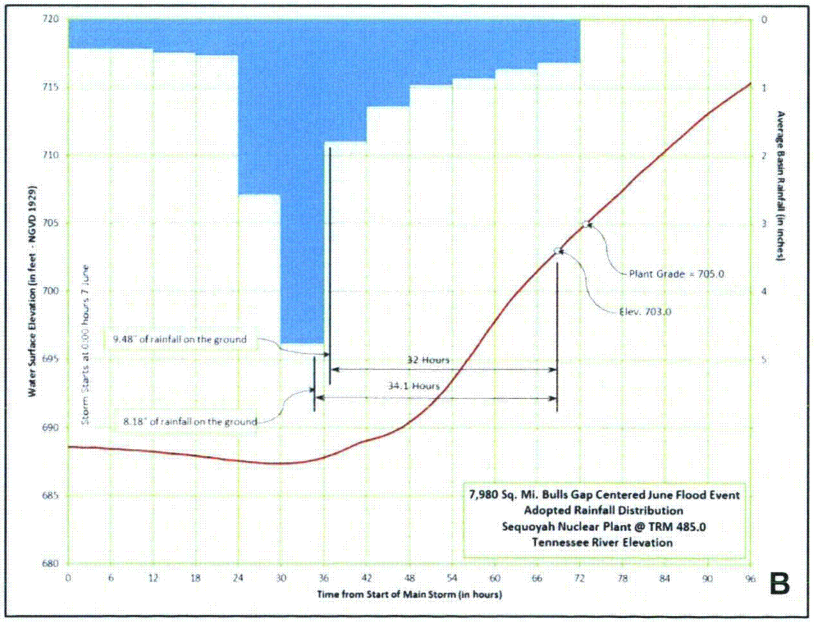


Figure 2.4.14-3 (Sheet 2 of 2) - Sequoyah Nuclear Plant Rainfall Flood Warning Time Basis for Safe Shutdown For Plant Flooding - Summer Events

ENCLOSURE 2

EVALUATION OF ISSUES FROM PRE-APPLICATION MEETING

On March 29, 2012, a Category 1 public meeting was held between the U.S. Nuclear Regulatory Commission (NRC) and representatives of the Tennessee Valley Authority (TVA) at NRC Headquarters, One White Flint North, 11555 Rockville Pike, Rockville, Maryland. The purpose of the meeting was to discuss TVA's planned submittal of a license amendment request to revise the licensing and design basis for hydrologic engineering as described in the Watts Bar Nuclear Plant (WBN), Unit 1 Updated Final Safety Analysis Report (UFSAR).

Following this pre-application meeting, the NRC Staff published a meeting summary, "Summary of March 29, 2012, Pre-Application Meeting with Tennessee Valley Authority on Changing the Licensing Basis for Hydrologic Engineering (TAC No. ME8200)," dated April 11, 2012 (ADAMS Accession No. ML12097A306). In this letter, the NRC Staff recommended that TVA consider addressing the following issues in the submittal. Any issue only related to WBN Unit 1 or for which the response is the same as that for WBN Unit 1 as described in the TVA submittal to the NRC Document Control Desk, "Application to Revise Watts Bar Nuclear Plant Unit 1 Updated Final Safety Analysis Report Regarding Changes to Hydrologic Analysis, TAC No. ME8200 (WBN-UFSAR-12-01)," is noted below.

1. **The chronology and basis for the changes made to the hydrologic engineering design basis from 1995 to 1998 to 2009.**

   This response is the same as WBN Unit 1 except for references to the applicable site, and applies to the hydrologic analysis for Sequoyah Nuclear Plant (SQN) Units 1 and 2.

   The probable maximum flood (PMF) for SQN Units 1 and 2 at the time of Operating License issuance was elevation 722.6 ft, and included assumptions based on the existing understanding of dam structural stability and capability during seismic and extreme flood events in the 1970's. In the 1980's and 1990's, TVA implemented a Dam Safety Program (DSP) that resulted in dam safety modifications that increased dam structural stability and capability Between 1995 and 1998, TVA completed a hydrologic reanalysis to credit the results of the dam safety modifications that had been completed. This reanalysis resulted in lowering the SQN Units 1 and 2 calculated PMF to elevation 719.6 ft, but no physical changes to SQN Units 1 and 2 site flooding protection features were implemented as a result of the decreased design basis flood (DBF) elevations. In 2009, TVA completed a hydrologic reanalysis to address closure of issues involving the hydrologic analysis for the application for a combined operating license (COLA) for the proposed Bellefonte Nuclear Plant (BLN) Units 3 and 4, in accordance with 10 CFR 52. This reanalysis resulted in raising the SQN Units 1 and 2 calculated PMF to elevation 722.0 ft. Although this was not higher than the original PMF but is higher than the earlier revised PMF, no physical changes to SQN Units 1 and 2 site flooding protection features were required based on the changes to PMF alone. However, because of the updates to the Design Basis Flood (DBF) levels based on the most recent wind-wave runup calculations, the Spent Fuel Pit Pump Motors and equipment required for flood mode operation located in the Diesel Generator Building are affected. Temporary compensatory measures are in place and documentation changes and permanent plant modifications are planned to provide adequate flooding protection for this equipment. This is described in Section 1.0 of Enclosure 1, Summary Description.

EVALUATION OF ISSUES FROM PRE-APPLICATION MEETING

2. An update of the status of TVA's resolution of long-term hydrology issues, per the staff's request in the NRC letter dated January 25, 2012.

This response is the same as WBN Unit 1 except for references to the applicable site, and applies to the hydrologic analysis for SQN Units 1 and 2.

On May 31, 2012, a Category 1 public meeting was held between the NRC staff and representatives of the TVA at NRC Headquarters, Two White Flint North, 11545 Rockville Pike, Rockville, Maryland. The purpose of the meeting was to discuss (1) the current licensing basis for flooding at WBN Unit 1 and SQN Units 1 and 2, (2) the status of TVA's current licensing basis reanalysis, (3) flooding protection and flood mode operation at WBN and SQN, (4) modular flood barriers at TVA dams, and (5) TVA's flooding reevaluation plan regarding the NRC's Fukushima 50.54(f) letter dated March 12, 2012.

Following this senior management meeting, the NRC Staff published a meeting summary, "Summary of May 31, 2012, Senior Management Meeting with Tennessee Valley Authority on the Licensing Basis for Flooding/Hydrology," dated June 6, 2012 (ADAMS Accession No. ML12157A457). The TVA slide presentation is provided in ADAMS Accession No. ML12156A076. In the meeting summary, the NRC Staff acknowledged the following related to the status of TVA's resolution of long-term hydrology issues:

a. TVA discussed the challenges faced with the complexities of the revised hydrology modeling used for the licensing basis re-analysis, and TVA acknowledged the lack of timeliness in resolving the flooding issue.

b. TVA discussed the management commitment for regaining safety margin for flooding and updating the current licensing basis through a high quality analysis, ensuring plant operability, and improved timeliness.

c. TVA made a number of commitments at the end of the presentation. These commitments have now been formalized in the TVA submittal to the NRC Document Control Desk, "Commitments Related to Updated Hydrologic Analysis Results for Sequoyah Nuclear Plant, Units 1 and 2, and Watts Bar Nuclear Plant, Unit 1," dated June 13, 2012 (ADAMS Accession No. ML12171A053).

Therefore, the NRC Staff including senior management has been provided an updated status based on the TVA presentation, responses provided by TVA during the presentation, and the commitments provided by TVA regarding future actions to complete the hydrologic analysis and applicable documentation changes and permanent plant and dam embankment modifications. With the exception of implementing the commitments provided to the NRC, there are no other actions required for this issue for SQN Units 1 and 2.

3. The relationship and use of the 25-year flood level versus the May 2003 flood level in TVA's new analysis.

This response is the same as for WBN Unit 1 and applies to the hydrologic analysis for SQN Units 1 and 2.

As described in the second paragraph of Section 3.2 of Enclosure 1, Uncertainties, per NUREG/CR-7046 the only manner to address the uncertainty in the hydrologic analysis is

through calibration of the model to historic flood events or sensitivity analyses. TVA calibrated the model to historic flood events using the two highest recent flood events where data exists. The floods used for calibration are March 1973 and May 2003 storms. The May 2003 flood event was a much larger flood than the 25-year flood. The May 2003 flood reached a maximum elevation of 657.2 feet on May 8, 2003 on the Tennessee River at the Walnut Street gage at Tennessee River Mile (TRM) 464.2. This compares with the March 1973 flood, the maximum flood of record since regulation by the TVA system, which reached a maximum elevation of 658.06 feet on March 18, 1973. Based on the flood frequency elevations at the Walnut Street gage the May 2003 flood was about a 100-year event as shown in the tabulation below.

The flood frequency elevations at the Walnut Street gage TRM 464.2 are as follows:

Flood	Elevation (ft.)[1]
1-year	644.0
2-year	649.2
5-year	650.6
10-year	653.4
50-year	655.9
100-year	657.0
500-year	663.6

[1] National Geodetic Vertical Datum (NGVD) 1929

Based on review of observed elevations at key locations in the vicinity of SQN, the May 2003 flood event was about a 100-year event over the reach of interest with May 2003 maximum elevations exceeding flood of record elevations at some locations. A comparison of the maximum elevations reached during the May 2003 flood at key locations is shown in the tabulation below.

Location	Maximum Elevation (ft.) NGVD 1929	
	Flood of Record	May 2003
Chickamauga Dam Headwater	686.99 5/9/84	687.13 5/7/2003
Watts Bar Dam Tailwater	696.95 3/17/1973	694.17 5/7/2003

Using the calibrated model based upon the two highest recent flood events where data exists (i.e., March 1973 and May 2003), the 25-year flood event specified in RG 1.59 was used for application with the postulated Safe Shutdown Earthquake (SSE) failure of upstream dams as described in Section 2.1 of Enclosure 1, Proposed Changes, under the subheading Section 2.4.4, Potential Dam Failures, Seismically Induced. The 25-year flood magnitude was developed using flood volume frequency relationships. The inflow hydrographs were developed using the March 1973 flood, the flood of record, and a large regional flood, scaled by the ratio of the 25-year volume to the 1973 volume. This provides an estimate of the 25-year flood based on historical watershed experience.

4. **The justification for the proposed combinations of dam failure scenarios used in TVA's new analysis.**

This response is the same as WBN Unit 1 except for references to the applicable site, and applies to the hydrologic analysis for SQN Units 1 and 2.

EVALUATION OF ISSUES FROM PRE-APPLICATION MEETING

The methodology used to develop the controlling seismic/flood condition at SQN is the same as previously followed for the site evaluations described in the SQN Units 1 and 2 UFSAR as follows:

1. A ground motion attenuation function was generated to describe the peak horizontal acceleration of rock at the free surface versus distance from the epicenter.

2. Using the attenuation relationship, the seismic base accelerations for various dams having large stored inventory (reservoir storage) and low spatial separation were determined.

3. The seismic stability of the dams for the seismic event centered at the dam (maximum base acceleration) and seismic events which cause dam failures at adjacent dams (less than maximum base acceleration) were then determined.

4. Based on the predicted seismic stability of the dams (individually and in combination) and reservoir storage, the potential seismic failure/flooding combinations were screened to identify the controlling case for SQN.

5. Hydrological routing for the potential controlling cases was then performed.

The ground motion attenuation functions to permit evaluation of simultaneous failure of two or more dams were based on the attenuation characteristics of an Operating Basis Earthquake (OBE) and a SSE occurring in the geographic area encompassing the Tennessee Valley above Guntersville dam. Utilizing historical earthquake data from locations near the Tennessee Valley, an attenuation curve was developed. Using this OBE/SSE relationship, a representation of the earthquake was developed in the form of concentric circles radiating from a center 0.09g (OBE) or 0.18g (SSE) acceleration with each circle representing decreasing levels of base acceleration as the distance from the epicenter increased. The concentric circles centered at an acceleration of 0.09g/0.18g were then strategically moved around the dams above Guntersville Dam to determine potential multi-site critical base acceleration levels.

The dams above Guntersville were examined for seismic stability based on base acceleration level. During the period from 1970 to 1988, the initial seismic stability analyses were performed on the concrete dam sections and the earth embankments of critical dams. In this evaluation, some of the concrete dams such as Apalachia, Fort Patrick Henry, Melton Hill and Ocoee No. 3 were not analyzed due to their relatively small storage volume and were postulated to fail. In other cases, more detailed seismic evaluations were performed, such as at Norris Dam. The more detailed evaluation of Norris dam concluded that the dam would not fail in OBE (coincident with one-half PMF) or SSE (coincident with 25-year flood). However, for purposes of the seismic failure combinations Norris dam was conservatively postulated to fail with only the resulting debris field impeding flow.

Using the dam base accelerations and seismic stability evaluations (or failure assumptions) as screening criteria, various flood-seismic failure combinations were identified. Cases to be evaluated further were selected based on the potential reservoir flood volume released in seismic failures, the relative timing of those releases, and in some cases results of previous flood routing analysis.

The impact of multiple failures of the large reservoir dams identified in the screening evaluations bound the effects of a single dam failure. Thus, single dam failures were not further evaluated.

Using the earthquake attenuation function, the seismic stability determinations, reservoir volume, flood wave timing, and informal routing methods, the following cases were defined as having the potential to control at SQN for OBE coincident with one-half PMF:

1. Simultaneous failure of Norris and Tellico Dams: Melton Hill Dam located below Norris Dam is not failed with the OBE in this scenario to maximize the downstream impact of the seismic failure wave from Norris Dam that overtops and fails Melton Hill Dam which is judged to be more critical.

2. Simultaneous partial failure of Fontana Dam and complete failure of Hiwassee, Apalachia, Blue Ridge, and Tellico Dams due to an OBE at a location between Hiwassee and Fontana: Fort Loudoun and Watts Bar Dams are seismically stable at OBE base accelerations for this epicenter.

3. Simultaneous partial failure of Fontana Dam and complete failure of Tellico Dam: Fort Loudoun and Watts Bar Dams are seismically stable at base OBE accelerations.

At least three other failure combinations evaluated in the original SQN Units 1 and 2 UFSAR studies and judged not to be controlling were not re-evaluated as a part of the new analysis since they were not controlling in the original analysis.

The following failure combinations for the SSE coincident with the 25-year flood were defined as having the potential to control at SQN using the evaluation criteria:

1. Simultaneous failure of Norris, Cherokee, Douglas and Tellico Dams with SSE epicenter located in the North Knoxville vicinity: For this combination, Fort Loudoun, Watts Bar and Fontana Dams do not fail since the attenuated base acceleration at these dams is less than the base acceleration for which the dams are seismically stable. Melton Hill Dam is not failed seismically to maximize the downstream impact by allowing Melton Hill Dam to overtop and fail due to the Norris Dam failure wave.

2. Simultaneous failure of Norris, Douglas, Fort Loudoun and Tellico Dams: For this combination, Cherokee, Fontana and Watts Bar Dams do not fail since the attenuated base acceleration at these dams is less than the base acceleration for which the dams are seismically stable. Melton Hill Dam is not failed seismically to maximize the downstream impact by allowing Melton Hill to overtop and fail due to the Norris Dam failure wave.

At least seven other failure combinations evaluated in the original SQN Units 1 and 2 UFSAR studies and judged not to be controlling were not re-evaluated as a part of the new analysis.

Flood simulations for the five failure combinations described above were performed to define the maximum bounding elevation at SQN. This is further described in Section 2.1 of

Enclosure 1, Proposed Changes, under the subheading Section 2.4.4, Potential Dam Failures, Seismically Induced.

5. The purpose of the finite element analysis on the Fontana Dam.

This response is the same as WBN Unit 1, and applies to the hydrologic analysis for SQN Units 1 and 2.

As part of TVA's DSP and consistent with the Federal Guidelines for Dam Safety, TVA performed a review of Fontana Dam in the mid-1980s to determine if the dam was capable of withstanding a maximum credible earthquake (MCE) (Reference: Fontana Project Dam Safety Analysis Report, April 1986). The evaluation determined that Fontana Dam was capable of safely passing the PMF but the dam's ability to withstand earthquake loading was not assured. As a result of this finite element analysis, reinforcement of the upper portion of the non-overflow dam was recommended and subsequently implemented to ensure the dam would remain stable for the MCE.

Since this original finite element analysis did not consider the alkali aggregate reaction (AAR) expansion issues at Fontana Dam, additional analyses were performed to evaluate the seismic/hydrostatic stability of the dam and the impacts of stresses associated with AAR expansion in the dam structure.

Patterned cracking was first observed in the dam in 1949. Also, it was noted that the dam was beginning to tilt in the upstream direction at that time. In 1972, cracking was observed in the walls of the drainage gallery in the curved concrete blocks of the dam. A six-inch wide slot with a depth of about 95 feet was cut between November 1975 and July 1976 at the joint of Blocks 32/33 to relieve some of the stress. The slot had completely closed at the top of dam by October 1983. The top third (35 feet) of this slot required re-cutting to a width of five inches between October 1983 and January 1984. Slot closure measurements indicated that the slot closed gradually over time and would require re-cutting in the next several years. The third slot cutting to a width of six inches was performed between February - May 1999 and January – May 2000.

Clearance problems were first detected in the spillway gates of the main spillway in 1967. Pier tilting due to concrete growth was causing binding of the gates when they were being opened. The gates were trimmed four times between 1967 and 1989. In the late 1990's, it was concluded that slot cuts on each end of the spillway would help reduce the tilting of the end piers of the spillway. Two slots with the same width of about 0.6 inches, and with depths of 82 and 57 feet at joint Blocks 34/35 and 41/42 respectively, were cut in January 1999. In November 1999, re-cutting of the spillway slots was undertaken. However, slots 34/35 and 41/42 had closed during the summer season at the top of the slot by 2001.

In summary, three slots have been cut in Fontana Dam (Blocks 32/33, Blocks 34/35, and Blocks 41/42) to address problems associated with AAR. The first slot was cut at Blocks 32/33 in 1975. The slot was required to eliminate the longitudinal force from the long straight portion of the dam. The longitudinal force was tending to push the curved blocks upstream, thus creating the observed cracks. The two spillway slots located at each end of the spillway (Blocks 34/35 and Blocks 41/42) were installed to help control tilting of the piers into the spillway.

A finite element analysis was used to evaluate the existing slots in either the open or closed condition, the effects of cutting deeper slots, the effects of cutting additional slots, and to provide recommendations for long-term slot cutting strategy for best management of the Fontana Dam AAR problem. An August 2006 seismic/hydrostatic stability analysis performed by Acres International which considered the combined impacts of stresses associated with AAR expansion of the dam structure concluded that although the minimum sliding factor of safety is less than 1.0 for the critical section (FS = 0.814) when subjected to a sustained acceleration of 0.26g, the post-earthquake stability of the dam is acceptable.

6. **Discuss whether approvals for the dam and river operations modifications are required from other agencies (e.g., U.S. Army Corps of Engineers).**

This response is the same as WBN Unit 1, and applies to the hydrologic analysis for SQN Units 1 and 2.

TVA was created as a Federal agency by the Tennessee Valley Authority Act of 1933 with specific responsibilities for the unified development of the Tennessee River system. Approval is not required from other agencies for TVA's modifications to its dam and river system operations. However, modifications must be consistent with procedures set forth by the National Environmental Policy Act (NEPA), which is the same requirement for other federal agencies.

As a procedural act, NEPA calls for Federal agencies to make informed decisions, consider alternatives, to have decision-making processes that consider the environmental impacts of their proposed actions, and provide full disclosure of the process as applied. The level of environmental review required for a given action depends on the expected impact on the environment and/or when the proposed action is likely to be controversial.

The most recent environmental reviews that effected modification of the TVA river system were completed as Environmental Impact Statements (EIS) as follows:

1. Tennessee River and Reservoir System Operation and Planning Review, TVA, December 1990. Record of Decision issued February 1991.

2. Reservoir Operations Study, TVA, February 2004. Record of Decision issued May 2004. The U.S. Army Corps of Engineers (USACE) and U.S. Fish and Wildlife Service were cooperating agencies on this EIS.

As a part of the NEPA process, other Federal agencies and the public are invited to participate in the process. Consistent with the NEPA process, the final decision on any action to be taken as a result of the environmental review rests with the initiating Federal agency. In the case of reviews that have a potential impact resulting in modification of Tennessee River system operation, TVA makes the final decision on what actions are adopted for implementation.

The Act further gave TVA the power to construct dams and reservoirs on the Tennessee River and its tributaries to provide for navigation and control floods on the Tennessee and Mississippi River basins. To date, TVA has either acquired or constructed 49 dams located in seven different states as a part of the unified development of the region. The power given

to TVA for construction of dams and reservoirs in the Tennessee River basin is much like the authority given to the USACE on other river systems.

TVA has had a DSP since the first dams were acquired and/or built. Dam safety ensures that the impoundments and dams are designed, constructed, operated and maintained as safely and reliable as is practical. The DSP was formalized in 1982 to ensure consistency with the Federal Guidelines for Dam Safety which was issued in 1979. The guidelines apply to management practices for dam safety of Federal agencies responsible for the planning, design, construction, operation, or regulation of dams. Today, the Dam Safety Governance (DSG) procedures define TVA's dam safety responsibilities to ensure compliance with the Federal guidelines.

Since the DSP was formalized in 1982, TVA has systematically evaluated its dams for hydrologic and seismic adequacy which has resulted in several dams being physically modified. These modifications and operational changes as described above have been completed consistent with NEPA procedures.

The one location on the TVA system where an operational change would require the concurrence of the USACE is at Kentucky Dam. Kentucky Dam, located about 23.0 miles above the confluence of the Tennessee River with the Ohio River, is connected by a navigation canal located just above each dam to Barkley Reservoir, owned by the USACE. Thus, the Kentucky and Barkley Dams have to be operated in tandem. Further, the USACE has the authority to direct the operation of Kentucky reservoir during critical flood operations on the lower Ohio and Mississippi Rivers. The physical location and the large flood storage available allows Kentucky reservoir to provide significant flood reduction benefits on the lower Ohio and Mississippi Rivers.

There have been no operational changes proposed at Kentucky Dam that would require TVA to obtain concurrence from the USACE.

7. Discuss the overall uncertainties in TVA's revised analysis calculations.

This response is the same as WBN Unit 1, and applies to the hydrologic analysis for SQN Units 1 and 2.

The primary standards followed for development of the PMF are American National Standards Institute/American Nuclear Society (ANSI/ANS) 2.8 and RG 1.59. These guidance documents state that the PMF be derived from the combination of circumstances that collectively represent a risk probability that is acceptable for nuclear plant accidents. Each element in the development of the PMF is based on best available data including PMP estimates from the National Weather Service, rain-runoff relationships developed from historical storms, time distribution of PMP consistent with storms in the region, seasonal and areal considerations of rainfall, current reservoir operations, and verification of runoff and stream course models against large historic floods. Per regulatory guidance, the design-basis flood for nuclear power plants is an estimation. The calculations which support the PMF analysis document assumptions and approaches which are consistent with regulatory guidance. The PMF analysis is a best estimate and is consistent with current guidelines. However, it is realized that various elements of the analysis can result in different elevations, some higher and some lower, and those elements are discussed in

further detail in Section 3.2 of Enclosure 1, Uncertainties, in order to explain why the PMF analysis is a reasonable best estimate.

8. Justification for the use of any compensatory measures as a result of TVA's revised analysis.

This response is the same as WBN Unit 1 except for references to the applicable site, and applies to the hydrologic analysis for SQN Units 1 and 2.

The updated DBF analysis for SQN indicated that some upstream dam earth embankments could be overtopped during the PMF. Four dams were identified as having embankments that could be overtopped during the PMF: Cherokee; Fort Loudoun; Tellico; and Watts Bar. Once these earth embankment overtopping events were identified, actions were taken to prevent overtopping to ensure continued SQN operability. An evaluation of temporary flood barriers that could be installed in a short period of time and had a proven performance record for dependability led to the use of HESCO Concertainer units filled with stone. A total of approximately 18,000 feet of temporary flood barriers are installed at Cherokee, Fort Loudoun, Tellico and Watts Bar Dams. This installation was completed by the end of December 2009. The temporary flood barriers are located on the top of the earth embankments and/or on saddle dams as appropriate at each of the four dams. The temporary flood barrier configuration consists of HESCO Concertainer units from three feet in height to HESCO Concertainer units stacked based on manufacture recommendation up to seven feet.

The maintenance of the temporary flood barriers and closure of openings during emergency events is a River Operations (RO) – Asset Owner (AO) responsibility, as defined by Dam Safety procedure RO-SPP-27.0. The purpose of the Dam Safety procedure is to protect upstream and downstream lives and property by ensuring that impoundments and dams are designed, constructed, operated and maintained as safely and reliable as is practical. This procedure describes the methods by which the RO Senior Vice-President (AO) will accomplish compliance with Federal Guidelines for Dam Safety and DSG.

As a part of the RO DSP, the temporary flood barriers are inspected on a regular basis. They are inspected during plant monthly and quarterly inspections and during the 15 month comprehensive site inspections. Any noted damage to the HESCO Concertainer units from these inspections that would compromise the structural integrity or functionality of the temporary flood barriers is repaired promptly. Since completion of installation in December 2009, only minor repairs such as small holes up to three inches in diameter have had to be repaired. Also, as committed to in the TVA submittal to the NRC Document Control Desk, "Commitments Related to Updated Hydrologic Analysis Results for Sequoyah Nuclear Plant, Units 1 and 2, and Watts Bar Nuclear Plant, Unit 1," dated June 13, 2012 (ADAMS Accession No. ML12171A053), TVA's Nuclear Power Group will issue and initially perform procedures for semi-annual inspections of the temporary HESCO flood barriers installed at Cherokee, Fort Loudoun, Tellico, and Watts Bar reservoirs by August 31, 2012. These inspections will:

a. Ensure the temporary HESCO flood barriers remain in place and are not structurally degraded as specified by the manufacturer's written specifications and recommendations;

b. Verify the inventory and staging of the material required to fill the gaps that exist; and

c. Ensure that adequate physical security (e.g., fences and locks) is provided for the staged material against theft.

These inspections will continue until a permanent modification is implemented to prevent overtopping the Cherokee, Fort Loudoun, Tellico, and Watts Bar dams due to the PMF.

For each of the dams, Cherokee; Fort Loudoun; Tellico; and Watts Bar Dams, where the temporary flood barriers have been installed, a supplement to the project Emergency Action Plan (EAP) has been issued which describes the emergency notification responsibilities and procedures. The River Forecast Center has responsibility for identification of events which could exceed critical elevations at each dam consistent with their Emergency Notification procedure and notification to the AO of the flooding condition. The AO declares a Dam Safety emergency which following the Dam Safety procedure (RO-SPP-27.0) implements the Project PMF Barrier Closure Plan. Each of the four dams has openings in the temporary flood barriers which have to be closed. The EAP supplement details the methods to be used by TVA's construction partner GUBMK Constructors for closure of the openings. The closure of the opening can be accomplished by setup of the HESCO Concertainer units linked to the existing HESCO Concertainer units already in place or by overlap of the temporary flood barriers at a given location as appropriate. At each dam where material for closure of the temporary flood barriers is required, the materials (HESCO Concertainer units and stone) are stockpiled in a designated fenced enclosure as described in the supplement to the EAP.

Experience data on the use of the selected temporary flood barriers during historic floods and the vendor documentation on barrier testing were evaluated prior to selection and use. The USACE has also tested the HESCO Concertainer units by performing hydrostatic testing, wave-induced hydrodynamic testing, overtopping testing, and structural debris impact testing with a floating log. The debris impact testing was based on two different log sizes: 12 inch and 17 inch diameter logs (12 feet long) with an impact speed of five mph. The results of the laboratory testing showed that the HESCO Concertainer units were not damaged by the loading conditions used in the testing program.

Stability analysis of the temporary flood barriers was performed for seismic and hydrostatic (PMF) loadings. The analysis showed that the temporary flood barriers are stable under the seismic and PMF loading conditions. This is described in the proposed revision to SQN Units 1 and 2 UFSAR Subsection 2.4.3.4, which states that while the flood barriers are temporary structures, there is a structural analysis for the headwater loading behind the temporary flood barriers that verifies that failure would not occur. Additionally, a seismic evaluation completed on the flood barriers (without headwater behind the barriers) verifies that failure of the temporary flood barriers would not occur.

A potential exists for runaway barges to float downstream and impact the temporary flood barriers at two of the four dams where the barriers are in place. Barges along these reservoirs are typically tied off at barge terminals or mooring cells during high flow events, such as a PMF event. The mooring facilities, however, are not designed for PMF elevations and velocities, so the barges could break loose. There is no barge traffic on Cherokee Reservoir, so no potential for impact exists. The Fort Loudoun Reservoir has limited to moderate barge traffic. Using typical barge dimensions, the barge would have to weigh less

than 70-80% of full load capacity in order to strike the barriers. However, the earthen embankments of the dam where the temporary flood barriers are placed are located at a distance from the main channel. The stream flow during a high flow event is directed toward the concrete overflow portion of the dam, and the barges would be carried by the current away from the temporary flood barriers. At the Tellico Reservoir, there is very infrequent barge traffic. Conservatively assuming there will be a barge on the reservoir, and using typical barge dimensions, the barge would have to weigh less than 40-50% of full load capacity in order to strike the barriers. However, the earthen embankments of the dam where the temporary flood barriers are placed are located at a distance from the main channel. The stream flow during a high flow event is directed toward the concrete overflow portion of the dam, and the barges would be carried by the current away from the temporary flood barriers. There is limited to moderate barge traffic at the Watts Bar Reservoir. An evaluation using typical barge dimensions for the Tennessee River, and conservatively assuming barges are empty (less draft allows for the barge to run closer to the top of the dam), demonstrates that barges are not likely to impact the temporary flood barriers. A spatial analysis shows that the closest edge of the temporary flood barrier would have to be at least 9.0 ft away from the upstream edge of the earthen embankment in order to prevent impact. The temporary flood barriers are located at least this distance from the edge of the earthen embankment, ensuring that there is no potential for barge impact.

As discussed in the NRC letter to TVA, "Tennessee Valley Authority (TVA) Long-Term Hydrology Issues for Operating Nuclear Plants - Browns Ferry Nuclear Plant, Units 1, 2, and 3 (TAC Nos. ME5026, ME5027, and ME5028); Sequoyah Nuclear Plant, Units 1 and 2 (TAC Nos. ME5029 and ME5030); and Watts Bar Nuclear Plant, Unit 1 (TAC No. ME5031)," dated January 25, 2012, Accession No. ML11241A166, the NRC Staff found that the sand baskets [temporary flood barriers] are not capable of resisting debris impact. The NRC Staff further states that "documents [provided by TVA] neither discuss the ability of sand baskets to withstand debris impact, or mention whether the baskets are designed for impact of debris loads. The NRC staff is unable to conclude that these sand baskets were designed to withstand impacts from large debris during a flood. If a design flood were to occur, there is a high likelihood that significant debris would accompany the flood waters which could impact the baskets. There is the potential for this debris to damage the baskets or push the individual baskets apart causing a breach. There would be no time to repair the baskets because the flood would already be in progress. Therefore, sand baskets that are not designed and constructed to withstand impacts from large debris are not acceptable as a long-term solution."

To resolve this issue, as committed to in the TVA submittal to the NRC Document Control Desk, "Commitments Related to Updated Hydrologic Analysis Results for Sequoyah Nuclear Plant, Units 1 and 2, and Watts Bar Nuclear Plant, Unit 1," dated June 13, 2012 (ADAMS Accession No. ML12171A053), TVA will implement permanent modifications to prevent overtopping of the embankments of the Cherokee, Fort Loudoun, Tellico, and Watts Bar Dams due to the PMF. The final solution will be established in an evaluation conducted in compliance with the National Environmental Policy Act (NEPA) Environmental Impact Statement (EIS). Based on the current NEPA EIS schedule, these permanent modifications are scheduled to be installed by October 31, 2015.

Based on TVA RO procedures for the maintenance of the temporary flood barriers and closure of openings during emergency events; TVA RO and TVA's Nuclear Power Group periodic inspections of the temporary flood barriers and additional materials required for

closure of openings; experience data on the use of the HESCO temporary flood barriers during historic floods; stability analysis of the temporary flood barriers for seismic and hydrostatic (PMF) loadings; USACE tests of the HESCO Concertainer units including hydrostatic testing, wave-induced hydrodynamic testing, overtopping testing, and structural debris impact testing with a floating log; and TVA's qualitative assessment of the potential for runaway barges to float downstream and impact the temporary flood barriers; it is concluded that use of the temporary flood barriers for the period of time required to implement the permanent modifications to prevent overtopping of the embankments of the Cherokee, Fort Loudoun, Tellico, and Watts Bar Dams is adequate.

The use of the temporary flood barriers is described in Section 2.1 of Enclosure 1, Proposed Changes, under subheading Subsection 2.4.3, Probable Maximum Flood (PMF) on Streams and Rivers. The credit or lack of credit for the temporary flood barriers in the hydrologic analysis is described in Section 2.1 of Enclosure 1, Proposed Changes, under subheadings Subsection 2.4.3, Runoff and Stream Course Model, and Subsection 2.4.4, Dam Failure Permutations, respectively. In the proposed SQN Units 1 and 2 UFSAR Subsection 2.4.3, the increase in the height of the embankments are included in the discharge rating curves for Cherokee, Fort Loudoun, Tellico, and Watts Bar Dams that are used in the hydrologic analysis for rainfall-induced PMF events. Increasing the height of embankments at these four dams prevents embankment overflow and failure of the embankment. The vendor supplied temporary flood barriers were shown to be stable for the most severe PMF headwater/tailwater conditions using vendor recommended base friction values. In the proposed SQN Units 1 and 2 UFSAR Subsection 2.4.4, the temporary flood barriers are assumed to fail in the hydrologic analysis for seismically-induced dam failures for the cases where reservoir levels would increase to the top of the embankments, and are thus not credited for increasing the height of the embankments.

9. Discuss the temporary modification to the thermal barrier booster pump flood barrier protection in the UFSAR.

This issue was specific to WBN Unit 1. However, the temporary compensatory measures applicable to SQN Units 1 and 2 are discussed in Section 3.3 of Enclosure 1, Margins.

As committed to in the TVA submittal to the NRC Document Control Desk, "Commitments Related to Updated Hydrologic Analysis Results for Sequoyah Nuclear Plant, Units 1 and 2, and Watts Bar Nuclear Plant, Unit 1," dated June 13, 2012 (ADAMS Accession No. ML12171A053), TVA will implement a documentation change to require the Spent Fuel Pit Cooling Pump Enclosure caps as a permanent plant feature for flooding protection, and will install permanent plant modifications to provide adequate flooding protection with respect to the DBF level for the Diesel Generator Building, by March 31, 2013.

10. Discuss any impact on TVA's individual plant examination of external events or final environmental impact statement due to the revised flood analysis.

This issue was specific to WBN Unit 1 and to the initial licensing of WBN Unit 2, and is not applicable to SQN Units 1 and 2.

11. Discuss whether any flood barriers at the plant are impacted by the revised PMF level.

Only two distinct changes to the physical flooding protection features of SQN Units 1 and 2 are required.

As discussed further in Section 3.3 of Enclosure 1, the SQN Units 1 and 2 Spent Fuel Pit Cooling Pump Enclosure caps in the Auxiliary Building are now required to maintain adequate flooding protection of the Spent Fuel Pit Cooling Pump Motors during flood mode. The DBF surge level within flooded structures is elevation 722.5 ft. The Spent Fuel Pit Cooling Pump Motors platform is located at elevation 721.0 ft, but is located in an enclosure that provides flooding protection up to elevation 724.5 ft. However, the Spent Fuel Pit Cooling Pump Enclosure caps were not originally intended to be permanently installed. To restore margin for the Spent Fuel Pit Cooling Pump Motors, installation of the caps at any time prior to or during the event of a Stage I flood warning has been established as a compensatory measure. A documentation change is planned to require the SQN Units 1 and 2 Spent Fuel Pit Cooling Pump Enclosure caps as a permanent plant feature for flooding protection.

As discussed further in Section 3.3 of Enclosure 1, the lowest floor of the common SQN Units 1 and 2 Diesel Generator Building is at elevation 722.0 ft with its doors on the uphill side facing away from the main body of flood water. This elevation is lower than the updated DBF level of elevation 723.2 ft. Therefore, flood levels exceed the floor level at elevation 722.0 ft. The entrances into safety-related areas and mechanical and electrical penetrations into safety-related areas are sealed to prevent major leakage into the building for water up to the grade elevation of 722.0 ft. Additionally, redundant sump pumps are provided within the building to remove minor leakage. As a result of this increase, staged sandbags to be constructed into a berm at the entrances to the Diesel Generator Building at any time prior to or during the event of a Stage I flood warning has been established as a compensatory measure. These sandbags will be constructed into a berm at least three ft in height (elevation 725.0 ft) to prevent water intrusion inside the building. Permanent plant modifications are planned to provide adequate flooding protection features for the common SQN Units 1 and 2 Diesel Generator Building.

As committed to in the TVA submittal to the NRC Document Control Desk, "Commitments Related to Updated Hydrologic Analysis Results for Sequoyah Nuclear Plant, Units 1 and 2, and Watts Bar Nuclear Plant, Unit 1," dated June 13, 2012 (ADAMS Accession No. ML12171A053), TVA will implement a documentation change to require the Spent Fuel Pit Cooling Pump Enclosure caps as a permanent plant feature for flooding protection, and will install permanent plant modifications to provide adequate flooding protection with respect to the DBF level for the Diesel Generator Building, by March 31, 2013.

12. Discuss the use and control of sand baskets (e.g., at the WBN recreational area).

This response is the same as WBN Unit 1, and applies to the hydrologic analysis for SQN Units 1 and 2.

Refer to the response to Issue 8 for more detailed description of use of the HESCO Concertainer units as a temporary flood barrier.

The temporary flood barriers installed in the vicinity of the recreational area at Watts Bar Dam are in place to prevent overtopping of the earth embankment during a PMF. There are three locations where closure of the access openings in the temporary flood barrier would

be required to complete the floodwall in advance of a PMF event. A supplement to the Emergency Action Plan for Watts Bar Dam has been issued to address procedures to be followed during such an event.

The HESCO Concertainer units (20-3'x3'x15' baskets) and stone (approximately 210 tons) needed to complete closure of the floodwall are stored in a designated fenced area near the campground and in proximity to the access points where they would be used. The HESCO Concertainer units are stored on pallets in a folded position.

The TVA River Forecast Center has responsibility for identification of events which could exceed critical elevations at the dam consistent with their Emergency Notification procedure and notification to the RO Senior Vice-President (AO) of the flooding condition. The AO declares a dam safety emergency which following the procedures implements the Watts Bar Dam PMF Barrier Installation Plan. The supplement details the methods, material and equipment to be used by TVA's construction partner GUBMK for closure of the openings through the floodwall. The closure of the opening can be accomplished by setup of the HESCO Concertainer units linked to the existing HESCO Concertainer units already in place or by overlap of the temporary flood barriers at a given location as appropriate.

Similar requirements for the use and control of the HESCO temporary flood barriers exist for Cherokee, Fort Loudoun, and Tellico Dams.

The use of the temporary flood barriers, and credit or lack of credit for the temporary flood barriers in the hydrologic analysis, is discussed further in the response to Issue 8.

As committed to in the TVA submittal to the NRC Document Control Desk, "Commitments Related to Updated Hydrologic Analysis Results for Sequoyah Nuclear Plant, Units 1 and 2, and Watts Bar Nuclear Plant, Unit 1," dated June 13, 2012 (ADAMS Accession No. ML12171A053), TVA will implement permanent modifications to prevent overtopping of the embankments of the Cherokee, Fort Loudoun, Tellico, and Watts Bar Dams due to the PMF. The final solution will be established in an evaluation conducted in compliance with the NEPA EIS. Based on the current NEPA EIS schedule, these permanent modifications are scheduled to be installed by October 31, 2015.

13. Discuss the impact on any safety-related equipment other than the thermal barrier booster pumps.

This issue was specific to WBN Unit 1, and is not applicable to SQN Units 1 and 2.

14. Discuss the impact of TVA's five proposed combinations of dam failure scenarios within its revised flood analysis.

This response is the same as WBN Unit 1, and applies to the hydrologic analysis for SQN Units 1 and 2.

As discussed in the response to Issue 4, the methodology used to develop the controlling seismic/flood condition at SQN is the same as previously followed for the site evaluations described in the SQN Units 1 and 2 UFSAR. This is further described in Section 2.1 of Enclosure 1, Proposed Changes, under the subheading Section 2.4.4, Potential Dam Failures, Seismically Induced.