ENCLOSURE 1

EVALUATION OF PROPOSED CHANGES

ATTACHMENT 1

Proposed SQN Units 1 and 2 UFSAR Text Changes (Markups)

2.4 HYDROLOGIC ENGINEERING

SQN is located on the right bank of Chickamauga Lake at Tennessee River mile (TRM) 484.5 with plant grade at elevation 705.0 ft MSL. The plant has been designed to have the capability for safe shutdown in floods up to the computed maximum water level, in accordance with regulatory position 2 of RG 1.59.

Determination of the maximum flood level included consideration of postulated dam failures from seismic and hydrologic causes. The maximum flood elevation of 722.0 ft would result from an occurrence of the probable maximum storm. Coincident wind wave activity results in wind waves of up to 4.2 ft (crest to trough). Wind wave run up on the Diesel Generator Building reaches elevation 723.2 ft. Wind wave run up on the critical wall of the ERCW Intake Pumping Station and the walls of the Auxiliary, Control and Shield Buildings reaches elevation 726.2 ft.

The nearest surface water user located downstream from SQN is East Side Utility at TRM 473.0, 11.7 miles downstream. All surface water supplies withdrawn from the 98.6 mile reach of the mainstream of the Tennessee River between Dayton, Tennessee (TRM 503.8) and the Mead Corporation in Stevenson, Alabama (TRM 405.2) are listed in Table 2.4.1-1.

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

The location of key plant structures and their relationship to the original site topography are shown on Figure 2.1.2-1. The structures which have safety-related equipment and systems are indicated on this figure and are tabulated below, along with the elevation of major exterior accesses.

Structure	Access	Number of Accesses	Elevation (ft)
Intake ~~pumping~~ Pumping ~~structure~~Structure	(1) Stairwell entrance	1	705.0
	(2) Access hatches	6	705.0
	(3) Cable tunnel	1	690.0
Auxiliary and ~~control buildings~~ Control Buildings	(1) Railroad access opening	1	706.0
	(2) Doors to turbine building	2	706.0
	(3) Doors to turbine building	2	732.0
	(4) Doors to turbine building	2	685.0
	(5) Personnel lock to SB	1	690.0
	(6) General vent or intake	2	714.0
	(7) Doors to AEB and MSVV	4	714.0
Shield ~~building~~Building	(1) Personnel lock (watertight)	1	691.0
	(2) Equipment hatch	1	730.0
	(3) Personnel lock	1	732.0
Diesel ~~generator building~~Generator Building	(1) Equipment access door	4	722.0
	(2) Personnel access door	1	722.0
	(3) Emergency exit	4	722.0
	(4) Emergency exit	1	740.5
ERCW ~~intake~~Intake ~~pumping station~~ Pumping Station	(1) Access door	1	725.0
	(2) Trash sluice	1	723.5
	(3) Deck drainage (sealed for flood)	1	720.0

Exterior accesses are also provided to each of the class IE electrical systems manholes and handholes at elevations varying from 700 ft MSL to 724 f~~ee~~t MSL, depending upon the location of

each structure.

The relationship of the plant site to the surrounding area can be seen in Figures 2.1.2-1 and 2.4.1-1. It can be seen from these figures that significant natural drainage features of the site have not been altered. Local surface runoff drains into the Tennessee River.

2.4.1.2 Hydrosphere

The ~~Sequoyah Nuclear Plant (~~SQN~~)~~ site comprises approximately 525 acres on a peninsula on the western shore of Chickamauga Lake at Tennessee River Mile (TRM) 484.5. As shown by Figure 2.4.1-1, the site is on high ground with the Tennessee River being the only potential source of flooding. SQN is located in the Middle Tennessee Chickamauga watershed, U.S. Geological Survey (USGS) hydrologic unit code 06020001, one of 32 watersheds in the Region 06 – Tennessee River Watershed (Figure 2.4.1-2).

The Tennessee River above SQN site drains 20,650 square miles. The drainage area at Chickamauga Dam, 13.5 miles downstream, is 20,790 square miles. Three major tributaries~~--~~, Hiwassee, Little Tennessee, and French Broad Rivers, ~~--~~rise to the east in the rugged Southern Appalachian Highlands. They flow northwestward through the Appalachian Divide which is essentially defined by the North Carolina-Tennessee border to join the Tennessee River which flows southwestward. The Tennessee River and its Clinch and Holston River tributaries flow southwest through the Valley and ridge physiographic province which, while not as rugged as the Southern Highlands, features a number of mountains including the Clinch and Powell Mountain chains. The drainage pattern is shown on Figure 2.1.1-1. About 20 percent of the watershed rises above elevation 3,000 ft with a maximum elevation of 6,684 ft at Mt. Mitchell, North Carolina. The watershed is about 70 percent forested with much of the mountainous area being 100 percent forested.

The climate of the watershed is humid temperate. ~~Mean annual precipitation for the Tennessee Valley is shown by Figure 2.4.1-2.~~ Above Chickamauga Dam, annual rainfall averages 51 inches and varies from a low of 40 inches at sheltered locations in the mountains to high spots of 85 inches on the southern and eastern divide. Rainfall occurs relatively evenly throughout the year. See Section 2.3 for a discussion of rainfall. The lowest monthly average is 2.9 inches in October. The highest monthly average is 6.8 inches in March, with January a close second with an average of 6.0 inches.

Major flood-producing storms are of two general types; the cool-season, winter type, and the warm-season, hurricane type. Most floods at SQN, however, have been produced by winter-type storms in the main flood-season months of January through early April.

Watershed snowfall is relatively light, averaging only about 14 inches annually above the plant. Snowfall above the 3,000-ft elevation averages 22 inches annually. The ~~maximum~~highest average annual snowfall ~~of~~in the basin is 63 inches ~~occurs~~ at Mt. Mitchell, the highest point east of the Mississippi River. The overall snowfall average above the 3,000-foot elevation, however, is only 22 inches annually. Individual snowfalls are normally light, with an average of 13 snowfalls per year. Snowmelt is not a factor in maximum flood determinations.

The Tennessee River, particularly above Chattanooga, Tennessee, is one of the most highly-regulated rivers in the United States. The TVA reservoir system is operated for flood control, navigation, and power generation with flood control a prime purpose with particular emphasis on protection for Chattanooga, 20 miles downstream from SQN.

Chickamauga Dam, 13.5 miles downstream, affects water surface elevations at SQN. ~~Normal~~Summer full pool elevation is ~~683.0~~682.5 f~~ee~~t. At this elevation the reservoir is 58.9 miles long on the Tennessee River and 32 miles long on the Hiwassee River, covering an area of ~~35,400~~36,050 acres, with a volume of ~~628,000~~622,500 acre-feet. The reservoir has an average width of nearly 1 mile, ranging from 700 f~~ee~~t to 1.7 miles. At ~~SQN,~~the SQN site the reservoir is about 3,000 f~~ee~~t wide with depths ranging between 12 f~~ee~~t and 50 f~~ee~~t at normal pool elevation.

~~The Tennessee River above Chattanooga, Tennessee, is one of the best regulated rivers in the United~~

~~States. A prime purpose of the TVA water control system is flood control with particular emphasis on protection for Chattanooga, 20 miles downstream from SQN.~~

There are ~~20~~17 major ~~reservoirs in the TVA system upstream from the plant, 13 of which have substantial reserved flood detention capacity during the main flood season. Table 2.4.1-1 lists pertinent data for TVA's major dams prior to modifications made by the Dam Safety Program (see Table 2.4.1-5).~~ dams (South Holston, Boone, Fort Patrick Henry, Watauga, Fontana, Norris, Cherokee, Douglas, Tellico, Fort Loudoun, Melton Hill, Blue Ridge, Apalachia, Hiwassee, Chatuge, Nottely, and Watts Bar) in the TVA system upstream from SQN, 14 of which (those previously identified excluding Fort Patrick Henry, Melton Hill, and Apalachia) provide about 4.8 million acre-ft of reserved flood-detention (March 15) capacity during the main flood season. Table 2.4.1-2 lists pertinent data for TVA's dams and reservoirs. Figure 2.4.1-3 presents a simplified flow diagram for the Tennessee River system. Table 2.4.1-3 provides the relative distances in river miles of upstream dams to the SQN site. Details for TVA dam outlet works are provided in Table 2.4.1-4. In addition, there are six major non-TVA dams, previously owned by the Aluminum Company of America (ALCOA). The ALCOA reservoirs often contribute to flood reduction, but ~~were ignored in this analysis because~~ they do not have dependable reserved flood detention capacity. ~~The locations of these dams and the minor dams, Nolichucky and Walters (Waterville Lake), are shown on Figure 2.1.1-1.~~ Table ~~2.4.1-2~~2.4.1-5 lists pertinent data for the ~~major and minor~~ ALCOA dams and Walters Dam (Waterville Lake). The locations of these dams are shown on Figure 2.1.1-1.

~~The flood detention capacity reserved in the TVA system varies seasonally, with the greatest amounts during the flood season. Figure 2.4.1-3, containing 14 sheets, shows tributary and main river reservoir seasonal operating guides for those reservoirs having major influence on SQN flood flows. Table 2.4.1-3 shows the flood control reservations at the multiple-purpose projects above SQN at the beginning and end of the winter flood season and in the summer. Assured system detention capacity above the plant varies from 5.6 inches on January 1 to 4.5 inches on March 15, decreasing to 1.0 inch during the summer and fall. Actual detention capacity may exceed these amounts, depending upon inflows and power demands.~~

Flood control above SQN is provided largely by ~~11~~12 tributary reservoirs. Tellico Dam is counted as a tributary reservoir because it is located on the Little Tennessee River, although, because of canal connection with Fort Loudoun Dam, it also functions as a main river dam. On March 15, near the end of the flood season, these provide a minimum of ~~4,436,000~~4,818,500 acre-feet of detention capacity, equivalent to ~~5.8~~approximately 5 inches on the ~~14,476~~approximately 19,500 square-mile area they control. This is 90 percent of the total available above Chickamauga Reservoir. The two main river reservoirs, Fort Loudoun and Watts Bar, provide 490,000 acre-feet, equivalent to ~~1.5~~approximately 1.2 inches of detention capacity on the remaining area above the ~~plant~~Chickamauga Dam.

The flood detention capacity reserved in the TVA system varies seasonally, with the greatest amounts during the January through March flood season. Figure 2.4.1-4 (16 sheets) shows the reservoir seasonal operating guides for reservoirs above the plant site. Table 2.4.1-6 shows the flood control reservations at the multiple-purpose projects above SQN at the beginning and end of the winter flood season and in the summer. Total assured system detention capacity above Chickamauga Dam varies from approximately 5.5 inches on January 1 to approximately 5 inches on March 15 and decreasing to approximately 1.5 inches during the summer and fall. Actual detention capacity may exceed these amounts, depending upon inflows and power demands.

Chickamauga Dam, the elevation of which affects flood elevations at the plant, has a drainage area of 20,790 square miles, 3,480 square miles more than Watts Bar Dam. There are seven major tributary dams (Chatuge, Nottely, Hiwassee, Apalachia, Blue Ridge, Ocoee No. 1 and Ocoee No. 3) in the 3,480-square-mile intervening watershed, of which four have substantial reserved capacity. On March 15, near the end of the flood season, these provide a minimum of 379,300 acre-ft equivalent to 5.9 inches on the 1,200-square-mile controlled area. Chickamauga Dam contains 345,300 acre-ft of detention capacity on March 15 equivalent to 2.8 inches on the remaining 2,280 square miles. Figure 2.4.1-4 (Sheet 1) shows the seasonal operating guide for Chickamauga.

Elevation-storage relationships for the reservoirs above the site and Chickamauga, downstream, are

shown in Figure 2.4.1-5 (17 sheets).

Daily flow volumes at the plant, for all practical purposes, are represented by discharges from Chickamauga Dam with drainage area of 20,790 square miles, only 140 square miles more than at the plant. Momentary flows at the nuclear plant may vary considerably from daily averages, depending upon turbine operations at Watts Bar Dam upstream and Chickamauga Dam downstream. There may be periods of several hours when there are no releases from either or both Watts Bar and Chickamauga Dams. Rapid turbine shutdown at Chickamauga may sometimes cause periods of ~~up-stream~~reverse flow in Chickamauga Reservoir.

Based upon discharge records since closure of Chickamauga Dam in 1940, the average daily streamflow at the plant is 32,600 cfs. The maximum daily discharge was 223,200 cfs on May 8, 1984. Except for two special operations on March 30 and 31, 1968, when discharge was zero to control milfoil, the minimum daily discharge was 700 cfs on November 1, 1953. Flow data for water years 1951-1972 indicate an average rate of about 27,600 cfs during the summer months (May-October) and about 38,500 cfs during the winter months (November-April). Flow durations based upon Chickamauga Dam discharge records for the period 1951-1972 are tabulated below.

Average Daily Discharge, cfs	Percent of Time Equaled or Exceeded
5,000	99.6
10,000	97.7
15,000	93.3
20,000	84.0
25,000	69.3
30,000	46.8
35,000	31.7

Channel velocities at SQN average about 0.6 fps under normal winter conditions. Because of lower flows and higher reservoir elevations in the summer months, channel velocities average about 0.3 fps.

As listed on Table 2.4.1-~~4~~1, there are 23 surface water users within the 98.6-mile reach of the Tennessee River between Dayton, TN and Stevenson, AL. These include fifteen industrial water supplies and eight public water supplies.

The industrial users exclusive of SQN withdraw about ~~497~~500 million gallons per day from the Tennessee River. Most of this water is returned to the river after use with varying degrees of contamination.

The public surface water supply intake (Savannah Valley Utility District), originally located across Chickamauga Reservoir from the plant site at TRM 483.6, has been removed. Savannah Valley Utility District has been converted to a ground water supply. The nearest public downstream intake is the East Side Utility (formerly referred to as U.S. Army, Volunteer Army Ammunition Plant). This intake is located at TRM 473.0.

Groundwater resources in the immediate SQN site are described in Section 2.4.13.2.

~~2.4.1.3 TVA Dam Safety Program~~

~~Most of the dams upstream from SQN were designed and built before the hydrometerological approach to spillway design had gained its current level of acceptance. Spillway design capacity was generally less than would be provided today. The original FSAR analyses were based on the existing dam system before dam safety modifications were made and included failure of some upstream dams from overtopping.~~

~~In 1982, TVA officially began a safety review of its dams. The TVA Dam Safety Program was designed to be consistent with Federal Guidelines for Dam Safety and similar efforts by other Federal~~

~~agencies. Technical studies and engineering analyses were conducted and physical modifications implemented to ensure the hydrologic and seismic integrity of the TVA dams and demonstrate that TVA's dams can be operated in accordance with Federal Emergency Management Agency (FEMA) guidelines. Table 2.4.1-5 provides the status of TVA Dam Safety hydrologic modifications as of 1998. These modifications enable these projects to safely pass the probable maximum flood. The remaining hydrologic modifications planned for Bear Creek Dam and Chickamauga Dam will not affect SQN in any manner which might invalidate the reanalysis described below.~~

~~In 1997-98, TVA reanalyzed the nuclear plant design basis flood events. The purpose of the reanalysis was to evaluate the effects of the hydrologic dam safety modifications on the flood elevations and response times in the SQN FSAR and to confirm the adequacy of the plant flood plans. The following methods and assumptions were applied to the reanalysis:~~

~~1. The computer programs and modeling methods were the same as previously used and documented in the FSAR.~~

~~2. Probable maximum precipitation, time distribution of precipitation, precipitation losses and reservoir operating procedures were unchanged from the original analysis.~~

~~3. The original stability analyses and postulated seismic dam failure assumptions were conservatively assumed to occur in the same manner and in combination with the same previously postulated rainfall events. No credit was taken for the 1988 post-tensioning of Fontana and Melton Hill Dams to prevent seismic failure. Nor was any credit taken for Dam Safety seismic evaluations of Norris, Cherokee, Douglas, Fort Loudon, Tellico, Hiwassee, Apalachia, and Blue Ridge Dams which demonstrated their structural integrity for a seismic event with a return period of approximately 10,000 years.~~

~~4. The planned modification of Chickamauga Dam (armoring the embankment to permit overtopping) was conservatively assumed to have been implemented for the purpose of calculating flood effects. Under present existing conditions, the Chickamauga embankment would be severely eroded in the overtopping PMF event and the maximum flood elevation at SQN would be lower than that with the planned modification.~~

2.4.2 Floods

2.4.2.1 Flood History ~~(Historical)~~

The nearest location with extensive formal flood records is 20 miles downstream at Chattanooga, Tennessee, where continuous records are available since 1874. Knowledge about significant floods extends back to 1826, based upon newspaper and historical reports. Flood flows and stages at Chattanooga have been altered by TVA's reservoir system beginning with the closure of Norris Dam in 1936 and reaching essentially the present level of control in 1952 with closure of Boone Dam, the last major dam with reserved flood detention capacity constructed above Chattanooga. Tellico Dam provides additional reserved flood detention capacity; however, the percentage increase in total detention capacity above the ~~Watts Bar~~Sequoyah site is small. Thus, for practical purposes, flood records for the period 1952 to date can be considered representative of prevailing conditions. Table 2.4.2-1 provides annual peak flow data at Chattanooga. Figure 2.4.2-1 shows the known flood experience at Chattanooga in diagram form. The maximum known flood under natural conditions occurred in 1867. This flood ~~reached~~was estimated to reach elevation 690.5 ft at SQN site with a discharge of about 450,000 cfs. The maximum flood elevation at the site under present-day regulation reached elevation 687.9 ft at the site on May 9, 1984.

The following table lists the highest floods at SQN site under present-day regulation:

Date	Estimated Elevation~~,~~ at SQN (F~~ee~~t)	Discharge~~,~~ at Chickamauga Dam (cfs)
~~Before Regulation~~		
~~March 11, 1867~~	~~690.5~~	~~450,000~~
~~March 1, 1875~~	~~686.2~~	~~405,000~~
~~April 3, 1886~~	~~684.5~~	~~385,000~~
~~March 7, 1917~~	~~680.0~~	~~335,000~~
~~April 5, 1920~~	~~676.5~~	~~270,000~~
~~Since Present Regulation~~		
February 3, 1957	683.7	180,000
March 13, 1963	684.8	205,000
March 18, 1973	687.0	219,000
April 5, 1977	685.0	150,000
May 9, 1984	687.9	250,000
April 20, 1998	685.9	180,000
May 7, 2003	687.8	225,000

There are no records of flooding from seiches, dam failures, or ice jams. Historic information about icing is provided in Section 2.4.7.

2.4.2.2 Flood Design Considerations

TVA has planned the SQN project to conform with ~~regulatory position 2 of~~ Regulatory Guide 1.59 including position 2.

The types of events evaluated to determine the worst potential flood included (1) Probable Maximum Precipitation (PMP) on the total watershed and critical ~~subwatersheds~~sub-watersheds, including seasonal variations and potential consequent dam failures and (2) dam failures in a postulated ~~Safe Shutdown Earthquake (~~SSE~~)~~ or ~~one-half SSE~~OBE with guide specified concurrent flood conditions.

Specific analysis of Tennessee River flood levels resulting from ocean front surges and tsunamis is not required because of the inland location of the plant. Snow melt and ice jam considerations are also unnecessary because of the temperate zone location of the plant. Flood waves from landslides into upstream reservoirs required no specific analysis, in part because of the absence of major elevation relief in nearby upstream reservoirs and because the prevailing thin soils offer small slide volume potential compared to the available detention space in reservoirs. Seiches pose no flood threats because of the size and configuration of the lake and the elevation difference between normal lake level and plant grade.

The ~~computed maximum stillwater~~maximum PMF plant site flood level ~~in the reservoir at the plant site from any cause~~ is elevation ~~719.6~~722.0 ft. This elevation would result from the PMP critically centered on the watershed as described in Section 2.4.3.

~~Maximum level including wave height is 722.4. This elevation would result from the probable maximum precipitation critically centered on the watershed and a~~ Wind waves based on an overland wind speed of 45~~-mile-per-hour overwater wind, from the most critical direction~~ miles per hour were assumed to occur coincident with the flood peak~~ of the resulting flood~~. This would create maximum wind waves up to 4.2 ft high (trough to crest).

All safety related facilities, systems, and equipment are housed in structures which provide protection from flooding for all flood conditions up to plant grade at elevation 705.0 ft. See Section 2.4.10 for more specific information.

Other rainfall floods will also exceed plant grade~~,~~ elevation 705.0 ft~~,~~ and ~~will necessitate~~require plant shutdown. ~~Flood warning criteria and forecasting techniques have been developed to assure that there will always be adequate time to shut the plant down and be ready for floodwaters above plant~~

~~grade and are described in Subsections 2.4.10 and 2.4.14, and Appendix 2.4A.~~Section 2.4.14 describes emergency protective measures to be taken in seismic events exceeding plant grade.

Seismic and ~~concurrent~~ flood events could ~~create flood levels which would exceed~~cause dam failure surges exceeding plant grade elevation 705.0 ft. ~~The maximum elevation reached in such an event is elevation 707.9, 2.9 feet above plant grade and 11.7 feet below the controlling event probable maximum flood (PMF), excluding wind-wave considerations. In all such events there is adequate time for safe plant shutdown after the seismic event and before plant grade would be crossed. The emergency protective measures and warning criteria are described in Subsections 2.4.10 and 2.4.14, and Appendix 2.4A.~~Section 2.4.14 describes emergency protective measures to be taken in seismic events exceeding plant grade.

~~Most safety-related building accesses are located at elevation 706 or above. The accesses below elevation 706 are within the powerhouse and will not be exposed to floodwater until plant grade is exceeded. Therefore, the structures are protected from flooding prior to the end of the shutdown period.~~

~~Drainage to the Tennessee River has been provided to accommodate runoff from the probable maximum precipitation on the local area of the plant site.~~

~~Specific analysis of Tennessee River flood levels resulting from oceanfront surges and tsunamis is not required because of the inland location of the plant.~~

~~Snowmelt and ice jam considerations are also unnecessary because of the temperate zone location of the plant. Flood waves from landslides into upstream reservoirs required no specific analysis, in part because of the absence of major elevation relief in nearby upstream reservoirs and because the prevailing thin soils offer small slide volume potential compared to the available detention space in reservoirs.~~

~~All safety-related facilities, systems, and equipment are housed in structures which provide protection from flooding for all flood conditions up to plant grade at elevation 705.~~

~~For the condition where flooding exceeds plant grade, as described in Subsections 2.4.3 and 2.4.4, all equipment required to maintain the plant safely during the flood, and for 100 days after the beginning of the flood, is either designed to operate submerged, located above the maximum flood level, or otherwise protected.~~

~~Safety-related~~ For the condition where flooding exceeds plant grade, as described in Sections 2.4.3 and 2.4.4, those safety-related facilities, systems, and equipment located in the containment structure are protected from flooding by the ~~shield building. All~~Shield Building structure with those accesses and penetrations below the maximum flood level ~~in the shield building are~~ designed and constructed as ~~water-tight~~watertight elements.

Wind wave run up during the PMF at the Diesel Generator Building would reach elevation 723.2 ft which is 1.2 ft above the operating floor. Consequently, wind wave run up will impair the safety functions of the Diesel Generator Building. The accesses and penetrations below this elevation in the Diesel Generator Building are designed and constructed to minimize leakage into the building. Redundant sump pumps are provided within the building to remove minor leakage. Protective measures are taken to ensure that all safety-related systems and equipment in the Emergency Raw Cooling Water (ERCW) Intake Pumping Station will remain functional when subjected to the maximum flood level.

Those Class 1E electrical system conduit banks located below the PMF plus wind wave run up flood level are designed to function submerged with either continuous cable runs or qualified, type tested splices.

The ~~turbine, control, and auxiliary building~~Turbine, Control, and Auxiliary Buildings will be allowed to flood. All equipment required to maintain the plant safely during the flood, and for 100 days after the

beginning of the flood, is either designed to operate submerged, is located above the maximum flood level, or is otherwise protected.

~~Wind wave run-up during the PMF at the diesel generator building reaches elevation 721.8 which is 0.2 feet below the operating floor. Consequently, wind wave run-up will not impair the safety function of systems in the diesel generator building.~~

~~The accesses and penetrations below this elevation in the diesel generator building are designed and constructed to minimize leakage into the buildings. Redundant sump pumps are provided within the building to remove minor leakage. Protective measures are taken to ensure that all safety-related systems and equipment in the Emergency Raw Cooling Water (ERCW) pump station will remain functional when subjected to the maximum flood level.~~

~~Class IE electrical cables, located below the Probable Maximum Flood (PMF) plus wind-wave activity and required in a flood, are designed for submerged operation.~~

2.4.2.3 Effects of Local Intense Precipitation

Maximum water levels at buildings expected to result from the local plant PMP were determined using two methods: (1) when flow conditions controlled, standard-step backwater from the control section using peak discharges estimated from rainfall intensities corresponding to the time of concentration of the area above the control section or (2) when ponding or reservoir-type conditions controlled, storage routing the inflow hydrograph equivalent to the PMP hydrograph with 2-minute time intervals.

Structures housing safety-related facilities, systems, and equipment are protected from flooding during a local PMF by the slope of the plant yard. The yard is graded so that the surface runoff will be carried to Chickamauga Reservoir without exceeding the elevation of the external accesses given in Paragraph 2.4.1.1 except those at the intake pumping station whose pumps can operate submerged.

PMP for the plant drainage system and roofs of safety-related structures was determined from Hydrometeorological Report No. 45 [2]. The probable maximum storm used to test the adequacy of the local drainage system would produce 27.5 inches of rainfall in six hours with a maximum one-hour depth of 14 inches. Depths for each of the six hours in sequence were 1.5, 2.3, 5.0, 14.0, 3.0, and 1.7 inches.

The separate watershed subareas and flowpaths are shown on Figure 2.4.3-22.

Runoff from the 24.5 acre western plant site will flow either northwest to a 27-foot channel along the main plant tracks and then across the main access highway or to the south over the swale in Perimeter Road near the 161-kV switchyard and across Patrol Road to the river. Because the 500-kV switchyard and TEACP building areas are essentially level, peak outflows from this subarea were determined using method (2). These peak outflows were then combined with discharge estimates from the remaining areas, using method (1), to establish peak water surface profiles from both the north channel and south swale. The maximum water surface elevation is below critical floor elevation 706 and occurs near the east-west centerline of the Turbine Building.

The 28.9 acre eastern plant site was evaluated as two areas. Area 1 (19.7 acres) including the diesel generator, unit two reactor building, field services/storage buildings and adjacent areas. Runoff from area 1 will flow to the south along the perimeter road and across the pavement with low point elevation 705.0 ft to the discharge channel. Maximum water surface elevations computed using method (1) were less than elevation 706.0 ft. Area 2 (9.2 acres) includes the office/service, unit one reactor building, office/power stores buildings, intake pumping station, and adjacent areas. Runoff from area 2 will flow to the north and west along the ERCW pumping station access road to the intake channel and river. Maximum water surface elevation computed using method (2) is less than elevation 706.

Underground drains were assumed clogged throughout the storm. For fence sections, the Manning's n value was doubled to account for increased resistance to flow and the potential for debris blockage.

2.4.3 Probable Maximum Flood (PMF) on Streams and Rivers

The guidance of Appendix A of Regulatory Guide 1.59 was followed in determining the PMF. ~~Plant surface drainage was evaluated and found capable of passing the local probable maximum storm without reaching or exceeding the critical floor elevation 706, as further described in 2.4.3.5.~~

~~Evaluation of seasonal and areal variations of probable maximum storms showed that the probable maximum Tennessee River flood level at the plant would be caused by a sequence of storms occurring in March centered in the mountains, east of the plant. The flood crest at the plant would be augmented by the failure of the west saddle dike at Watts Bar Dam upstream. The estimated maximum discharge is 1,236,000 cfs. The probable maximum elevation at the plant is 719.6, excluding any wind-wave effects, and excluding any lower flood level due to failure of Chickamauga Dam downstream.~~Two basic storm situations were found to have the potential to produce maximum flood levels at SQN. These are (1) a sequence of storms producing PMP depths on the 21,400-square-mile watershed above Chattanooga and (2) a sequence of storms producing PMP depths in the basin above Chattanooga and below the five major tributary dams (Norris, Cherokee, Douglas, Fontana, and Hiwassee), hereafter called the 7,980-square-mile storm. The maximum flood level at the plant would be caused by the March PMP 21,400-square-mile storm. The flood level for the 7,980-square-mile storm would be slightly less.

In both storms, the West Saddle Dike at Watts Bar Dam would be overtopped and breached. No other failure would occur. Maximum discharge at the plant is 1,331,623 cfs for the 21,400-square-mile storm. The resulting PMF elevation at the plant would be 722.0 ft excluding wind wave effects.

2.4.3.1 Probable Maximum Precipitation

Probable maximum precipitation (PMP) for the Tennessee River watershed above SQN has been defined for TVA by the Hydrometeorological Branch of the National Weather Service in Hydrometeorological Report No. 41 Reference [1]. ~~Two basic storm positions were evaluated. One would produce maximum rainfall over the total watershed. The other would produce maximum rains in the part of the basin downstream from major TVA tributary reservoirs, hereafter referred to as the 7,980-square-mile storm. Snowmelt~~This report defines depth-area-duration characteristics, seasonal variations, and antecedent storm potentials and incorporates orographic effects of the Tennessee River Valley. Due to the temperate climate of the watershed and relatively light snowfall, snowmelt is not a factor in generating maximum floods for the Tennessee River at the plant site.

~~Controlling PMP depths for 21,400-square-mile and 7,980-square-mile areas are tabulated below. These storms would occur in March. Depths for other months would be less.~~

	~~Depth, Inches~~			
	~~72-Hour~~	~~Main Storm~~		
~~Sq. Miles~~	~~Antecedent Storm~~	~~6-Hour~~	~~24-Hour~~	~~72-Hour~~
~~21,400~~	~~6.7~~	~~5.03~~	~~11.18~~	~~16.78~~
~~7,980~~	~~8.1~~	~~7.02~~	~~14.04~~	~~20.36~~

Two basic storms with three possible isohyetal patterns and seasonal variations described in Hydrometeorological Report No. 41 were examined to determine which would produce maximum flood levels at the SQN site. One would produce PMP depths on the 21,400-square-mile watershed above Chattanooga. Two isohyetal patterns are presented in Hydrometeorological Report No. 41 for this storm. The isohyetal pattern with downstream center would produce maximum rainfall on the middle portion of the watershed and is shown in Figure 2.4.3-1.

~~Two possible isohyetal patterns producing the total area depths are presented in Report No. 41.~~The second storm described in Hydrometeorological Report No. 41 would produce PMP depths on the 7,980-square-mile watershed above Chattanooga and below the five major tributary dams. ~~The one critical to this study is the "downstream pattern" shown in Figure 2.4.3-1.~~ The isohyetal pattern for the 7,980-square-mile storm is ~~shown in Figure 2.4.3-2. The pattern is~~ not ~~orographically~~geographically

fixed and can be moved parallel to the long axis, northeast and southwest, along the Tennessee Valley. The isohyetal pattern centered at Bulls Gap, Tennessee, would produce maximum rainfall on the upper part of the watershed and is shown in Figure 2.4.3-2.

~~A 72-hour storm three days antecedent to the main storm was assumed to occur in all PMP situations with storm~~ All PMP storms are nine-day events. A three-day antecedent storm was postulated to occur three days prior to the three-day PMP storm in all PMF determinations. Rainfall depths equivalent to 40 percent of the main storm were used for the antecedent storms with uniform areal distribution as recommended in Report No. 41.

~~Potential storm amounts differing by seasons were analyzed in sufficient number to make certain that the March storms would be controlling. Enough centerings were investigated to assure that a most critical position was used.~~ Seasonal variations were also considered. Table 2.4.3-1 provides the seasonal variations of PMP. The two seasons evaluated were March and June. The March storm was evaluated because the PMP was maximum and surface runoff was also maximum. The June storm was evaluated because the June PMP was maximum for the summer season and reservoir elevations were at their highest levels. Although September PMP is somewhat higher than that in June, less runoff and lower reservoir levels more than compensate for the higher rainfall.

~~Storms producing PMP above upstream tributary dams, whose failure has the potential to create maximum flood levels, were evaluated in the original FSAR analysis. Dam safety modifications at upstream tributary dams have eliminated these potential failures and subsequent plant site flood levels.~~

A standard time distribution pattern was adopted for ~~all~~the storms based upon major observed storms transposable to the Tennessee Valley and in conformance with the usual practice of Federal agencies. The adopted distribution is ~~shown on Figure 2.4.3-3~~ within the limits stipulated in Chapter VII of Hydrometeorological Report No. 41. This places the heaviest precipitation in the middle of the storm. The adopted sequence closely conforms to that used by the U.S. Army Corps of Engineers. A typical distribution mass curve resulting from this approach is shown in Figure 2.4.3-3.

The PMF discharge at SQN was determined to result from the 21,400-square-mile storm producing PMP on the watershed with the downstream storm pattern, as defined in Hydrometeorological Report No. 41. The PMP storm would occur in the month of March and would produce an average of 16.25 inches of rainfall in three days on the watershed above Chickamauga Dam. The storm producing the PMP would be preceded by a three-day antecedent storm producing an average of 6.18 inches of rainfall, which would end three days prior to the start of the PMP storm. Precipitation temporal distribution is determined by applying the mass curve (Figure 2.4.3-3) to the basin rainfall depths in Table 2.4.3-2.

~~The critical probable maximum storm was determined to be a total basin storm with downstream orographically fixed pattern (Figure 2.4.3-1) which would follow an antecedent storm commencing on March 15. Translation of the PMP from Report No. 41 to the basin results in an antecedent storm producing an average precipitation of 6.4 inches in three days, followed by a three-day dry period, and then by the main storm producing an average precipitation of 16.5 inches in three days. Figure 2.4.3-4 is an isohyetal map of the maximum three-day PMP. Basin rainfall depths are given in Table 2.4.3-1.~~

~~PMP for the plant drainage system and roofs of safety-related structures was determined from Hydrometeorological Report No. 45 [2]. The probable maximum storm used to test the adequacy of the local drainage system would produce 27.5 inches of rainfall in six hours with a maximum one-hour depth of 14 inches. Depths for each of the six hours in sequence were 1.5, 2.3, 5.0, 14.0, 3.0, and 1.7 inches.~~

2.4.3.2 Precipitation Losses

~~Precipitation losses in the probable maximum storm are estimated with multivariable relationships used in the day-to-day operation of the TVA system. These relationships, developed from a study of storm and flood records, relate the amount of precipitation excess (and hence the precipitation loss) to~~

~~the week of the year, an antecedent precipitation index (API), and geographic location. The relationships are such that the loss subtraction from rainfall to compute precipitation excess is greatest at the start of the storm and decreases to no subtraction when the storm rainfall totals from 7 to 16 inches. Precipitation losses become zero in the late part of extreme storms.~~

~~For this probable maximum flood analysis, median moisture conditions as determined from past records were used to determine the API at the start of the storm sequence. The antecedent storm is so large, however, that the precipitation excess computed for the later main storm is not sensitive to variations in adopted initial moisture conditions. The precipitation loss in the critical probable maximum storm totals 4.13 inches, 2.30 inches in the antecedent storm amounting to 36 percent of the 3-day 6.44-inch rainfall, and 1.83 inches in the main storm amounting to 11 percent of the 3-day, 16.46 inch rainfall. Table 2.4.3-1 displays the API, rain, and precipitation excess for each of the 45 subwatersheds of the hydrologic model for the SQN probable maximum flood.~~

~~No precipitation loss was applied in the probable maximum storm on the local area used to test the adequacy of the site drainage system and roofs of safety-related structures. Runoff was made equal to rainfall.~~

A multi-variable relationship, used in the day-to-day operation of the TVA reservoir system, has been applied to determine precipitation excess directly. The relationships were developed from observed storm and flood data. They relate precipitation excess to the rainfall, week of the year, geographic location, and antecedent precipitation index (API). In their application, precipitation excess becomes an increasing fraction of rainfall as the storm progresses in time and becomes equal to rainfall in the later part of extreme storms. An API determined from an 11-year period of historical rainfall records (1997-2007) was used at the start of the antecedent storm. The precipitation excess computed for the main storm is not sensitive to variations in adopted initial moisture conditions because of the large antecedent storm.

Basin rainfall, precipitation excess, and API are provided in Table 2.4.3-2. The average precipitation loss for the watershed above Chickamauga Dam is 2.33 inches for the three-day antecedent storm and 1.86 inches for the three-day main storm. The losses are approximately 38% of antecedent rainfall and 11% of the PMP, respectively. The precipitation loss of 2.33 inches in the antecedent storm compares favorably with that of historical flood events shown in Table 2.4.3-3.

2.4.3.3 Runoff Model

The runoff model used to determine Tennessee River flood hydrographs at SQN is divided into ~~45~~40 unit areas and includes the total watershed above Chickamauga Dam. Unit hydrographs are used to compute flows from these areas. The watershed unit areas are shown in Figure 2.4.3-5. The unit area flows are combined with appropriate time sequencing or channel routing procedures to compute inflows into the most upstream tributary reservoirs, which in turn are routed through the reservoirs, using standard routing techniques. Resulting outflows are combined with additional local inflows and carried downstream using appropriate time sequencing or routing procedures, including unsteady flow routing. ~~Figure 2.4.3-5 shows unit areas of the watershed upstream from SQN.~~

~~The runoff model used in this updated FSAR differs from that used previously because of refinements made in some elements of the model during PMF studies for other nuclear plants and those made from information gained from the 1973 flood, the largest that has occurred during present reservoir conditions.~~

~~Changes are identified when appropriate in the text. They include both additional and revised unit hydrographs and additional and revised unsteady flow stream course models.~~

Unit hydrographs were developed for each unit area for which discharge records were available from maximum flood hydrographs either recorded at stream ~~gauging~~gaging stations or estimated from reservoir headwater elevation, inflow, and discharge data using the procedures described by Newton and Vineyard Reference [23]. For non-gaged unit areas synthetic unit graphs were developed from relationships of unit hydrographs from similar watersheds relating the unit hydrograph peak flow to the

drainage area size, time to peak in terms of watershed slope and length, and the shape to the unit hydrograph peak discharge in cfs per square mile. Unit hydrograph plots are provided in Figure 2.4.3-6 (11 Sheets). Table 2.4.3-4 contains essential dimension data for each unit hydrograph.~~The number of unit areas has been increased from 34 used previously to 45. The differences include:~~

~~1. Use of the model developed for the Phipps Bend study which combined the two unit areas for Watauga River (Sugar Grove and Watauga local) into one unit area and divided the Cherokee to Gate City unit area into two unit areas (Surgoinsville local and Cherokee local below Surgoinsville);~~

~~2. Use of the model developed for the Clinch River Breeder Reactor which increased the unit areas on the Clinch River from 3 to 11 and the Watts Bar local from 1 to 2;~~

~~3. Changes to add an unsteady flow model for the Fort Loudoun-Tellico Dam complex which included dividing the lower Little Tennessee River unit area into two unit areas (Fontana to Chilhowee and Chilhowee to Tellico), and the Fort Loudoun local unit area into three unit areas (French Broad River local, Holston River local and Fort Loudoun local); and~~

~~4. Combining the two unit areas above Ocoee No. 1 (Ocoee No. 1 and Ocoee No. 3) into one unit area (Ocoee No. 1 to Blue Ridge).~~

~~In addition, eight of the unit graphs have been revised. Figure 2.4.3-6, which contains 11 sheets, shows the unit hydrographs. Table 2.4.3-2 contains essential dimension data for each unit hydrograph and identification of those hydrographs which are new or revised.~~

Tributary reservoir routings, except for Tellico and Melton Hill, were made using ~~the Goodrich semigraphical method~~standard reservoir routing procedures and flat pool storage conditions. ~~Main~~The main river reservoirs, ~~and~~ Tellico, and Melton Hill routings were made using unsteady flow techniques. ~~This differs from the previous submission in that:~~

~~1. An unsteady flow model has been added for the Fort Loudoun-Tellico complex, and~~

~~2. The Chickamauga unsteady flow model has been revised using the 1973 flood data and results from the HEC-2 backwater computer program.~~

~~In the original study, the failure wave hydrograph of the mouth of the Hiwassee River was approximated for the postulated failures of Hiwassee, Apalachia and Blue Ridge dams as described in section 2.4.4.2.1. In the 1998 reassessment, an unsteady flow model developed during the dam safety studies was used as an adjunct to route the Hiwassee, Apalachia and Blue Ridge failures in the one half SSE. The model was verified by comparing model elevations in a state of steady flow with elevations computed by the standard-step method. This was done for steady flows ranging from 25,000 cfs to 1,000,000 cfs.~~

Unsteady flow routings were computer-solved with ~~a~~the Simulated Open Channel Hydraulics (SOCH) mathematical model based on the equations of unsteady flow, [3]. ~~Boundary conditions prescribed were inflow hydrographs at the upstream boundary, local inflow, and headwater discharge relationships at the downstream boundary based upon normal operating rules, or based upon rated curves when geometry controlled.~~The SOCH model inputs include the reservoir geometry, upstream boundary inflow hydrograph, local inflows, and the downstream boundary headwater discharge relationships based upon operating guides or rating curves when the structure geometry controls. Seasonal operating curves are provided in Figure 2.4.1-4 (16 Sheets).

Discharge rating curves are provided in Figure 2.4.3-7 (17 Sheets) for the reservoirs in the watershed at and above Chickamauga. The discharge rating curve for Chickamauga Dam is for the current lock configuration with all 18 spillway bays available. Above SQN, temporary flood barriers have been installed at four reservoirs (Watts Bar, Fort Loudoun, Tellico and Cherokee Reservoirs) to increase the height of embankments and are included in the discharge rating curves for these four dams. 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. A

single postulated Fort Loudoun Reservoir rim leak north of the Marina Saddle Dam which discharges into the Tennessee River at Tennessee River Mile (TRM) 602.3 was added as an additional discharge component to the Fort Loudoun Dam discharge rating curve. Seven Watts Bar Reservoir rim leaks were added as additional discharge components to the Watts Bar Dam discharge rating curve. Three of the rim leak locations discharge to Yellow Creek, entering the Tennessee River three miles downstream of Watts Bar Dam. The remaining four rim leak locations discharge to Watts Creek, which enters Chickamauga Reservoir just below Watts Bar Dam. A single postulated Nickajack Reservoir rim leak just northeast of Nickajack Dam and back into the Tennessee River below Nickajack Dam was added as an additional discharge component for the Nickajack Dam.

The unsteady flow mathematical model ~~for the 49.9-mile-long Fort Loudoun Reservoir was divided into twenty-four 2.08-mile reaches. The model was verified at three gauged points within Fort Loudoun Reservoir using 1963 and 1973 flood data~~ configuration for the Fort Loudoun-Tellico complex is shown by the schematic in Figure 2.4.3-8. The Fort Loudoun Reservoir portion of the model from TRM 602.3 to TRM 652.22 is described by 29 cross-sections with additional sections being interpolated between the original sections for a total of 59 cross-sections in the SOCH model, with a variable cross-section spacing of about 1 mile. The unsteady flow model was extended upstream on the French Broad and Holston Rivers to Douglas and Cherokee Dams, respectively. The French Broad ~~and Holston River unsteady flow models were verified at one gaged point each at mile 7.4 and 5.5, respectively, using 1963 and 1973 flood data~~ River from the mouth to Douglas Dam at French Broad River mile (FBRM) 32.3 was described by 25 cross-sections with additional sections being interpolated between the original sections for a total of 49 cross-sections in the SOCH model, with a variable cross-section spacing of about 1 mile. The Holston River from the mouth to Cherokee Dam at Holston River mile (HRM) 52.3 was described by 29 cross-sections with one additional cross-section being interpolated between each of the original sections for a total of 57 cross-sections in the SOCH model, with a variable cross-section spacing of about 1 mile.

The Little Tennessee River was modeled from Tellico Dam, ~~mile 0.3, through Tellico Reservoir to Chilhowee Dam at mile 33.6, and upstream to Fontana Dam at mile 61.0. The model for Tellico Reservoir to Chilhowee Dam was tested for adequacy by comparing its results with steady-state profiles at 1,000,000 and 2,000,000 cfs computed by the standard-step method. Minor decreases in conveyance in the unsteady flow model yielded good agreement. The average conveyance correction found necessary in the reach below Chilhowee Dam to make the unsteady flow model agree with the standard-step method was also used in the river reach from Chilhowee to Fontana Dam~~ Little Tennessee River mile (LTRM) 0.3 to Chilhowee Dam at LTRM 33.6. The Little Tennessee River from Tellico Dam to Chilhowee Dam at LTRM 33.6 was described by 23 cross-sections with additional sections being interpolated between the original sections for a total of 49 cross-sections in the SOCH model, with a variable cross-section spacing of up to about 1.8 miles.

~~The~~ Fort Loudoun and Tellico unsteady flow models ~~were~~are joined by ~~a canal unsteady flow model. The canal was modeled with five equally-spaced cross Sections at 525-foot intervals for the 2,100-foot-long canal~~ an interconnecting canal. The canal was modeled using nine cross-sections with an average cross-section spacing of about 0.18 miles.

The Fort Loudoun-Tellico complex was calibrated by two different methods as follows:

(1) Using the available data for the March 1973 flood on Fort Loudoun Reservoir and for the French Broad and Holston rivers. The calibration of the 1973 flood is shown in Figure 2.4.3-9 (2 Sheets). Because there were limited data to verify against on the French Broad and Holston rivers, the steady-state HEC-RAS model was used to replicate the Federal Emergency Management Agency (FEMA) published 100- and 500-year profiles. Tellico Dam was not closed until 1979, thus was not in place during the 1973 flood for calibration.

(2) Using available data for the May 2003 flood for the Fort Loudoun-Tellico complex. The calibration of the May 2003 flood is shown in Figure 2.4.3-10 (3 Sheets). The Tellico Reservoir steady-state HEC-RAS model was also used to replicate the FEMA published 100- and 500-year profiles.

A schematic of the steady-state SOCH model for Watts Bar Reservoir is shown in Figure 2.4.3-11.

The ~~unsteady flow routing~~ model for the 72.4-mile-long Watts Bar Reservoir was ~~divided into thirty-four 2.13-mile reaches. The model was verified at two gauged points within the reservoir using 1963 flood data~~described by 39 cross-sections with two additional sections being added in the upper reach for a total of 41 sections in the SOCH steady state model with a variable cross-section spacing of up to about 2.8 miles. The model also includes a junction with the Clinch River at Tennessee River mile (TRM) 567.7. The Clinch River arm of the model goes from Clinch River mile (CRM) 0.0 to CRM 23.1 at Melton Hill Dam with one additional section being interpolated between each of the original 13 sections and cross-section spaces of up to about 1 mile. Another junction at TRM 601.1 connects the Little Tennessee River arm of the model from the mouth to Tellico Dam at LTRM 0.3 with cross-section spaces of about 0.08 miles. The time step was tested between 5 and 60 seconds which produced stable and comparable results over the full range. A time step of 5 seconds was used for the analysis to allow multiple reservoirs and/or river segments to be coupled together with different cross-section spacing. The verification of Watts Bar Reservoir for the March 1973 and the May 2003 floods are shown in Figure 2.4.3-12 and Figure 2.4.3-13, respectively..

~~The~~A schematic of the unsteady flow mathematical model for Chickamauga Reservoir is shown in Figure 2.4.3-14. The model for the total 58.9-mile-long Chickamauga Reservoir was ~~divided into twenty-eight 2.1-mile reaches providing twenty-nine equally-spaced grid points. The grid point at mile 483.62 is nearest to the plant, mile 484.5. The unsteady flow model was verified at four gauged points within Chickamauga Reservoir using 1973 flood data. This differs from the previous submission in that the 1973 flood was added for verification, replacing the 1963 flood. The 1973 flood occurred during preparation of the FSAR and therefore, was not available for verification. The 1973 flood is the largest which has occurred since closure of South Holston Dam in 1950. Comparisons between observed and computed stages in Chickamauga Reservoir are shown in Figure 2.4.3-7~~described by 29 cross-sections with one additional section being interpolated between each of the original 29 sections for a total of 53 sections in the SOCH model with a variable cross-section spacing of up to about 1 mile. The model also includes a junction with the Dallas Bay embayment at TRM 480.5. The Dallas Bay arm of the model goes from Dallas Bay mile (DB) 5.23 to DB 2.86, the control point for flow out of Chickamauga Reservoir. Another junction at TRM 499.4 connects the Hiwassee River arm of the model from the mouth to the Charleston gage at HRM 18.9. The time step was tested between 5 and 50 seconds producing stable and comparable results over the full range. A time step of 5 seconds was used for the analysis to allow multiple reservoirs and/or river segments to be coupled together with different cross-section spacing. The verification of Chickamauga Reservoir for the March 1973 and the May 2003 floods are shown in Figure 2.4.3-15 and Figure 2.4.3-16, respectively.

~~It is impossible to verify the models~~Verifying the reservoir models with actual data approaching the magnitude of the ~~probable maximum flood. The best remaining alternative was to compare the model elevations in a state of steady flow with elevations computed by the standard step method. This was done for steady flows ranging up to 1,500,000 cfs. An example shown by the rating curve of Figure 2.4.3-8 shows the good agreement~~ PMF is not possible, because no such events have been observed. Therefore, using flows in the magnitude of the PMF (1,200,000 – 1,300,000 cfs), steady-state profiles were computed using the HEC-RAS [24] steady state model and compared to computed elevations from the SOCH model. An example of the comparison between HEC-RAS and SOCH profiles is shown for Chickamauga Reservoir in Figure 2.4.3-17. This approach was applied for each of the SOCH reservoir models. Similarly, the tailwater rating curve was compared at each project as shown for Watts Bar Dam in Figure 2.4.3-18. In this figure, the initial tailwater curve is compared to results from the HEC-RAS or SOCH models.

The reservoir operating guides applied during the SOCH model simulations mimic, to the extent possible, operating policies and are within the current reservoir operating flexibility. In addition to spillway discharge, turbine and sluice discharges were used to release water from the tributary reservoirs. Turbine discharges were also used at the main river reservoirs up to the point where the head differentials are too small and/or the powerhouse would flood. All discharge outlets (spillway gates, sluice gates, and valves) for projects in the reservoir system will remain operable without failure up to the point the operating deck is flooded for the passage of water when and as needed during the flood. A high confidence that all gates/outlets will be operable is provided by periodic inspections by TVA plant personnel, the intermediate and five-year dam safety engineering inspections consistent with Federal Guidelines for Dam Safety, and the significant capability of the emergency response teams to direct and manage resources to address issues potentially impacting gate/outlet functionality.

Median initial reservoir elevations for the appropriate season were used at the start of the PMF storm sequence. Use of median elevations is consistent with statistical experience and avoids unreasonable combinations of extreme events.

The flood from the antecedent storm occupies about 70% of the reserved system detention capacity above Watts Bar Dam at the beginning of the main storm (day 7 of the event). Reservoir levels are at or above guide levels at the beginning of the main storm in all but Apalachia and Fort Patrick Henry Reservoirs, which have no reserved flood detention capacity.

~~The watershed runoff model was verified by using it to reproduce the March 1963 and March 1973 floods; the largest recorded since closure of South Holston Dam. This differs from the previous submission in that the 1973 flood was added for verification, replacing the 1957 flood. Observed volumes of precipitation excess were used in verification. Comparisons between observed and computed outflows from Watts Bar and Chickamauga Dams for the 1973 and 1963 floods are shown in Figures 2.4.3-9 and 2.4.3-10, respectively.~~

~~From a study of the basic units of the predicting system and its response to alterations in various basic elements, it is concluded that the model serves adequately and conservatively to determine maximum flood levels.~~

2.4.3.4 Probable Maximum Flood Flow

The ~~probable maximum flood~~PMF discharge at SQN was determined to be ~~1,236,000~~1,331,623 cfs. This flood would result from the 21,400-square-mile storm in March with a downstream orographically-fixed storm pattern (Figure 2.4.3-1).

The PMF discharge hydrograph ~~of this flood~~ is shown in Figure 2.4.3-~~11~~19. ~~This flood would result from the total basin downstream orographically fixed storm pattern, Figure 2.4.3-4, more completely described in Section 2.4.3.1. The dam safety modification to Fort Loudon, Tellico, and Watts Bar Dams enable them to safely pass the PMF. The west saddle dike~~The West Saddle Dike at Watts Bar Dam (Figure 2.4.3-20) would be overtopped and the earth embankment breached. The discharge from the failed West Saddle Dike flows into Yellow Creek which joins the Tennessee River at mile 526.82, 41.82 miles above SQN.

Chickamauga Dam downstream would be overtopped~~ but was assumed not to fail as a failure would reduce the flood level at the site~~. The dam was postulated to remain in place, and any potential lowering of the flood levels at SQN due to dam failure at Chickamauga Dam was not considered in the resulting water surface elevation.

~~In the original FSAR analysis, the flood would overtop and breach the earth embarkments of Fort Loudon, Tellico, and Watts Bar Dams upstream.~~

~~A second candidate storm is the 7,980-square-mile storm centered at Bulls Gap, Tennessee, 50 miles northeast of Knoxville, shown in Figure 2.4.3-2. The flood from this storm would overtop and breach the west saddle dike at Watts Bar Dam. The flood from the 7,980-square-mile storm is the less critical storm and would produce a probable maximum discharge less than from the total basin storm.~~

~~The previous PMF evaluations considered candidate situations involving upstream tributary dams Douglas and Watauga. These two situations were shown at that time to be non-governing. Dam safety modifications have since eliminated the potential failures of these dams. Therefore, these two candidate situations have been eliminated.~~

~~Reservoir routings started at median observed elevations for the mid-March large area PMP storms. Median levels were reevaluated using operating experience for:~~

~~1. The total project period, or~~

~~2. The five-year period, 1972-1976, for those projects whose operating guides were changed in 1971.~~

~~Because of the wet years of 1972-1975 and the operating guide changes, median elevations were higher for 8 of the 13 tributary reservoirs where routing is involved.~~

~~Normal reservoir operating procedures were used in the antecedent storm. These used turbine and sluice discharge in the tributary reservoirs. Turbine discharges are not used in the main river reservoirs after large flood flows develop because head differentials are too small. Normal operating procedures were used in the principal storm, except that turbine discharge was not used in either the tributary or main river dams.~~

Concrete Section Analysis

For concrete dam sections, factors of safety in sliding were determined by comparison of the existing design headwater/tailwater levels to the headwater/tailwater levels that would occur in the PMF as described in Section 2.4.3. The structures were considered safe against failure if a factor of safety greater than 1.0 for sliding was demonstrated. The dams upstream of SQN passed this test.

~~All gates were determined to be operable without failures during the flood. Gates on main river dams would be fully raised, thus requiring no additional operations by the last day of the storm, which is before the structures and access roads would be inundated.~~

Spillway Gates

During peak PMF conditions, the radial spillway gates of Fort Loudoun and Watts Bar Dams are wide open with flow over the gates and under the gates. For this condition, both the static and dynamic load stresses in the main structural members of the Watts Bar Dam spillway gate are determined to be less than the yield stress and the stress in the trunnion pin is less than the allowable design stress. The open radial spillway gates at other dams upstream of Watts Bar Dam were determined to not fail by comparison to the Watts Bar Dam spillway gate analysis.

~~Median initial reservoir elevations were used at the start of the storm sequence used to define the PMF to be consistent with statistical experience and to avoid unreasonable combinations of extreme events. As a result, 53 percent of the total reserved system flood detention capacity was occupied at the start of the main flood. This is considered to be amply conservative. The statement made in the PSAR and subsequent versions of the FSAR that 67 percent of the reserved system detention capacity was occupied at the start of the main storm was in error. The correct percentage was 33. The remaining reserved system detention capacity was 67 percent. This erroneous statement was first made in the PSAR and was copied in subsequent statements where the routings were the same. In the revised analysis submitted in Amendment 51, all reservoirs are higher or about the same elevation at the beginning of the main storm as a result of the revised starting levels explained in Section 2.4.3.4 of the FSAR. This conservative change results in 53 percent of the total reservoir system detention capacity being occupied at the start of the main flood rather than 33 percent in previous studies.~~

~~Neither the initial reservoir levels nor the operating rules would have significant effect on maximum flood discharges and elevations at the plant site because spillway capacities, and hence, uncontrolled conditions, were reached early in the flood.~~

~~The procedures used to determine if and when an overtopped earth embankment would fail and the procedures for computing the effect of such failures are described in 2.4.4.2 and 2.4.4.3.~~

~~In testing the adequacy of the yard drainage system, to safely pass the site PMP, all underground drains were assumed clogged and the surface drainage to be full.~~

Waterborne Objects

Consideration has been given to the effect of waterborne objects striking the spillway gates and bents supporting the bridge across Watts Bar Dam at peak water level at the dam. The most severe potential for damage is postulated to be by a barge which has been torn loose from its moorings and floats into the dam.

Should the barge approach the spillway portion of the dam end on, one bridge bent could be failed by the barge and two spillway gates could be damaged and possibly swept away. The loss of one bridge bent will likely not collapse the bridge because the bridge girders are continuous members and the stress in the girders is postulated to be less than the ultimate stress for this condition of one support being lost. Should two gates be swept away, the nape of the water surface over the spillway weir would be such that the barge would likely be grounded on the tops of the concrete spillway piers and provide a partial obstruction to flow comparable to un-failed spillway gates. Hence the loss of two gates from this cause will have little effect on the peak flow and elevation.

Should the barge approach the spillway portion broadside, two and possibly three bridge bents may fail. For this condition the bridge would likely collapse on the barge and the barge would be grounded on the tops of the spillway piers. For this condition the barge would likely ground before striking the spillway gates because the gates are about 20 ft downstream from the leg of the upstream bridge bents.

Lock Gates

The lock gates at Fort Loudoun, Watts Bar, and Chickamauga were examined for possible failure with the conclusion that no potential for failure exists. The lock gate structural elements may experience localized yielding and may not function normally following the most severe headwater/tailwater conditions.

2.4.3.5 Water Level Determinations

The controlling PMF elevation at the SQN was determined to be 722.0 ft, produced by the 21,400-square-mile storm in March and coincident with overtopping failure of the West Saddle Dike at Watts Bar Dam. The PMF elevation hydrograph~~ of the controlling PMF, cresting at elevation 719.6,~~ is shown on Figure 2.4.3-~~12~~21. ~~Computation of both the probable maximum discharge hydrograph (Figure 2.4.3-11) and the corresponding elevation hydrograph was accomplished concurrently using the unsteady flow techniques~~ Elevations were computed concurrently with discharges using the SOCH unsteady flow reservoir model described in Section 2.4.3.3.

~~The less critical total area storm-producing PMP depths on the 7,980-square-mile watershed would produce crest elevation 718.9 at the plant site.~~

~~Maximum water levels at buildings expected to result from the local plant PMP were determined using two methods: (1) when flow conditions controlled, standard-step backwater from the control section using peak discharges estimated from rainfall intensities corresponding to the time of concentration of the area above the control section or (2) when ponding or reservoir-type conditions controlled, storage routing the inflow hydrograph equivalent to the PMP hydrograph with 2-minute time intervals.~~

~~The separate watershed subareas and flowpaths are shown on Figure 2.4.3-13a.~~

~~Runoff from the 24.5 acre western plant site will flow either northwest to a 27-foot channel along the main plant tracks and then across the main access highway or to the south over the swale in Perimeter Road near the 161-kV switchyard and across Patrol Road to the river. Because the 500-kV switchyard and TEACP building areas are essentially level, peak outflows from this subarea were determined using method (2). These peak outflows were then combined with discharge estimates from the remaining areas, using method (1), to establish peak water surface profiles from both the north channel and south swale. The maximum water surface elevation is below critical floor elevation 706 and occurs near the east-west centerline of the Turbine Building.~~

~~The 28.9 acre eastern plant site was evaluated as two areas. Area 1 (19.7 acres) including the diesel~~

~~generator, unit two reactor building, field services/storage buildings and adjacent areas. Runoff from area 1 will flow to the south along the perimeter road and across the pavement with low point elevation 705.0 to the discharge channel. Maximum water surface elevations computed using method (1) were less than elevation 706. Area 2 (9.2 acres) includes the office/service, unit one reactor building, office/power stores buildings, intake pumping station, and adjacent areas. Runoff from area 2 will flow to the north and west along the ERCW pumping station access road to the intake channel and river. Maximum water surface elevation computed using method (2) is less than elevation 706.~~

~~Underground drains were assumed clogged throughout the storm. For fence sections, the Manning's n value was doubled to account for increased resistance to flow and the potential for debris blockage.~~

~~The only stream adjacent to SQN is the Tennessee River. There are no streams within the site. The 1-percent-chance floodplain of the Tennessee River at the site is delineated on Figure 2.4.3-14. Details of the analyses used in the computation of the 1-percent-chance flood flow and water elevation are described in a study made by TVA for the Federal Insurance Administration (FIA) and published in February 1979 [5].~~

~~The only structures located in the 1-percent-chance floodplain are transmission towers, the intake pumping station skimmer wall, and the ERCW pump station deck. The ERCW pumps are located on the pump station deck at elevation 720.5, well above the 1-percent-chance flood level. These structures are shown on Figure 2.4.3-14.~~

~~The structures that are located in the floodplain will not alter flood flows or elevations. The 20,650-square-mile drainage area is not altered and the reduction in flow area at the site is infinitesimal and at the fringe of the flooded area. The site will be well maintained and any debris generated from it will be minimal and will present no problem to downstream facilities.~~

2.4.3.6 Coincident Wind-Wave Activity

Some wind waves are likely when the probable maximum flood crests at SQN. The flood would be near its crest for a day beginning about 2-1/2 days after cessation of the probable maximum storm. The day of occurrence would most likely be in the month of March or possibly the first week in April.

A conservatively high velocity of 45 miles per hour over water was adopted to associate with the probable maximum flood crest. A 45-mile- per-hour overwater velocity exceeds maximum March one-hour velocities observed in severe March windstorms of record in a homogeneous region as reported by the Corps of Engineers [6].

That a 45-mile-per-hour overwater wind is conservatively high, is supported also by an analysis of March day maximum winds of record collected at Knoxville and Chattanooga, Tennessee. The records analyzed varied from 30 years at Chattanooga to 26 years at Knoxville, providing samples ranging from 930 to 806 March days. The recorded fastest mile wind on each March day was used rather than hourly data because this information is readily available in National Weather Service publications. Relationships to convert fastest mile winds to winds of other durations were developed from Knoxville and Chattanooga wind data contained in USWB Form 1001 and the maximum storm information contained in Technical Bulletin No. 2 [6]. From the wind frequency analysis it was determined that the 45-mile-per-hour overwater wind for the critical minimum duration of 20 minutes had an 0.1 percent chance of occurrence on any given March day.

~~The probability that this wind might occur on the specific day that the probable maximum flood would crest is extremely remote. Even assuming that the flood was to crest once during the 40-year plant life, the probability of the wind occurring on that particular day is in the order of 1×10^{-6}.~~

~~TVA estimates that the probability of the flood and wind occurring in a given year on the same day to be in the order of 1×10^{-11} to 1×10^{-13}.~~

Computation of wind waves was made using the procedures of the Corps of Engineers [7]. The critical directions were from the north-northwest and northeast with effective fetches of 1.7 and 1.5 miles,

respectively. For the 45-mile-per-hour wind, 99.6 percent of the waves approaching the plant would be less than 4.2- and 4.0-foot-high crest to trough for the 1.7- and 1.5-mile fetches as shown on Figures 2.4.3-~~15~~24 and 2.4.3-~~16~~25. ~~Maximum water surfaces in the reservoir approaching the plant would be 2.8 and 2.7 feet above the maximum computed level or elevations 722.4 and 722.3, respectively.~~Only the most critical fetch length of 1.7 miles is used to determine the design basis flood elevations

The maximum water level attained due to the PMF plus wind-wave activity is elevation ~~723.8~~726.2 ft at the ERCW pump station and the nuclear island structures (~~shield, auxiliary, and control building~~Shield, Auxiliary, and Control Buildings).

The wind waves approaching the Diesel Generator Building and cooling towers break before reaching the structures due to the shallow depth of water. The topography surrounding these structures is such that the wind waves will break on a steeper slope (4H:1V) than the slope immediately adjacent to the structures. This is shown by Figure 2.4.3-~~17~~26.

~~The runup estimates are calculated on the basis that the incoming wind waves break before reaching the structure and then reform for a shallower water depth. This reformed wave then approaches the structure. The runups are lower than the maximum reservoir level due to the small wave height for the reformed wave, the shallow water, and the very shallow slope before reaching the structures.~~

Wind-wave runup coincident with the maximum flood level for the ~~diesel generator building~~Diesel Generator Building and cooling towers (Figure 2.4.3-26) is elevation ~~721.8~~723.2. The level inside structures that are allowed to flood is elevation 720.1. The flood elevations used as design bases are given in Section ~~2.4A.1.1~~2.4.14.1.1.

Dynamic Effect of Waves

1. Nonbreaking Waves

   The dynamic effect of nonbreaking waves on the walls of safety- related structures was investigated using the Rainflow Method [8]. As a result of this investigation, concrete and reinforcing stresses were found to be within allowables.

2. Breaking Waves

   The dynamic effect of breaking waves on the walls of safety-related structures was investigated using a method developed by D. D. Gaillard and D. A. Molitar. The concrete and reinforcing stresses were found to be less than the allowable stresses using this method.

3. Broken Waves

   The dynamic effect of broken waves on the walls of safety-related structures was investigated using a method proposed by the U.S. Army Coastal Engineering Research Center [7]. This method of design yielded concrete and reinforcing stresses within allowable limits.

   All safety-related structures are designed to withstand the static and dynamic effects of the water and waves as stated in Section 2.4.2.2.

2.4.4 Potential Dam Failures, ~~(~~Seismically ~~and Otherwise~~ Induced~~)~~

The procedures described in Appendix A of Regulatory Guide 1.59 were followed when evaluating potential flood levels from seismically induced dam failures.

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

It should be clearly understood that these studies have been made solely to ensure the safety of SQN against failure by floods caused by the assumed failure of dams due to seismic forces. To assure that safe shutdown of the SQN is not impaired by flood waters, TVA has in these studies added conservative assumptions to be able to show that the plant can be safely controlled even in the event that all these unlikely events occur in just the proper sequence.

By furnishing this information TVA does not infer or concede that its dams are inadequate to withstand earthquakes that may be reasonably expected to occur in the TVA region under consideration. TVA believes that multiple dam failures are an extremely unlikely event. The TVA Dam Safety Program (DSP), which is consistent with the Federal Guidelines for Dam Safety [25], conducts technical studies and engineering analyses to assess the hydrologic and seismic integrity of agency dams and verifies that they can be operated in accordance with Federal Emergency Management Agency (FEMA) guidelines. These guidelines were developed to enhance national dam safety such that the potential for loss of life and property damage is minimized. As part of the TVA DSP, inspection and maintenance activities are carried out on a regular schedule to confirm the dams are maintained in a safe condition. Instrumentation to monitor the dams' behavior was installed in many of the dams during original construction and other instrumentation has been added since. Based on the implementation of the DSP, TVA has confidence that its dams are safe against catastrophic destruction by any natural forces that could be expected to occur.

2.4.4.1 Dam Failure Permutations

There are 20 major dams above SQN. Dam locations with respect to the SQN site are shown in Figure 2.4.1-3. These are Watts Bar and Fort Loudoun Dams on the Tennessee River; Watauga, South Holston, Boone, Fort Patrick Henry, Cherokee, and Douglas Dams above Fort Loudoun; Norris, Melton Hill, Fontana, and Tellico Dams between Fort Loudoun and Watts Bar; and Chatuge, Nottely, Hiwassee, Apalachia, Blue Ridge, Ocoee No. 1, Ocoee No. 2, and Ocoee No. 3 emptying into Chickamauga Reservoir. These were examined individually, and in ~~groups~~combinations, to determine if failure might result from a seismic event and. if ~~such~~so, would failure ~~or failures occurring concurrently~~concurrent with storm runoff ~~would~~ create ~~critical~~maximum flood levels at the plant.

~~Two situations were examined: (1) a one-half Safe Shutdown Earthquake (SSE) as defined in Subsection 2.5.2, imposed concurrently with one-half the probable maximum flood and (2) a Safe Shutdown Earthquake (SSE) as defined in Subsection 2.5.2, imposed concurrently with a 25-year flood. Neither of these conditions would create levels greater than the hydrologic probable maximum flood at SQN, described previously in 2.4.3. Details of the dam failure analysis are discussed in Section 2.4.4.2, Dam Failure Permutations.~~ The procedures referred to in Regulatory Guide (RG) 1.59, Appendix A, were followed for evaluating potential flood levels from seismically induced dam failures. In accordance with this guidance, seismic dam failure is examined using the two specified alternatives:

(1) the Safe Shutdown Earthquake (SSE) coincident with the peak of the 25-year flood and a two-year wind speed applied in the critical direction,

(2) the Operating Basis Earthquake (OBE) coincident with the peak of the one-half PMF and a two-year wind speed applied in the critical direction.

The OBE and SSE are defined in Section 2.5.2.4 as having maximum horizontal rock acceleration levels of 0.09 g and 0.18 g respectively. As described in Section 2.5.2.4, TVA agreed to use 0.18 g as the maximum bedrock acceleration level for the SSE.

~~Failure of Chickamauga Dam, downstream, can affect cooling water supplies at the plant. Consequently for conservatism, an arbitrary failure was imposed. This resulting condition would not be critical to plant operation, as discussed in Section 2.4.11.6.~~ From the seismic dam failure analyses made for TVA's operating nuclear plants, it was determined that five separate, combined events have the potential to create flood levels above plant grade at Watts Bar Nuclear Plant. These events are as follows:

(1) The simultaneous failure of Fontana and Tellico Dams in the OBE coincident with one-half PMF.

(2) The simultaneous failure of Fontana, Tellico, Hiwassee, Apalachia, and Blue Ridge Dams in the OBE coincident with one-half PMF.

(3) The simultaneous failure of Norris and Tellico Dams in the OBE coincident with one-half PMF.

(4) The simultaneous failure of Cherokee, Douglas, and Tellico Dams in the OBE coincident with one-half PMF.

(5) The simultaneous failure of Norris, Cherokee, Douglas, and Tellico Dams in the SSE coincident with a 25-year flood.

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

~~2.4.4.1 Reservoir Description~~

~~Characteristics of dams that influence river conditions at SQN are contained in Tables 2.4.1-1 and 2.4.1-2. Their location with respect to the plant is shown on Figure 2.1.1-1. Seismic safety criteria were not incorporated in the design of dams upstream from SQN, except Tellico and Norris. Those projects having a potential to influence plant flooding levels were examined, as described in Section 2.4.4.2.~~

~~Elevation-storage relationships and seasonally varying storage allocations in the major projects are shown on the 14 sheets of Figure 2.4.1-3.~~

~~2.4.4.2 Dam Failure Permutations~~

~~The plant site and upstream reservoirs are located in the Southern Appalachian Tectonic Province and, therefore, subject to moderate earthquake forces with possible attendant failure. All upstream dams, whose failure has the potential to cause flood problems at the plant, were investigated to determine if failure from seismic or hydrologic events would endanger plant safety. Potential failures from both seismic and hydrologic events and the resulting consequences are discussed in this section.~~

~~It should be clearly understood that these studies have been made solely to ensure the safety of SQN against failure by floods caused from excessive rainfall or by the assumed failure of dams due to seismic forces. To assure that safe shutdown of SQN is not impaired by flood waters, TVA has in these studies added conservative assumptions to conservative assumptions to be able to show that the plant can be safely controlled even in the event that all these unlikely events occur in just the proper sequence. TVA is of the strong opinion that the chances of the assumed events occurring approach zero probability.~~

~~By furnishing this information, TVA does not infer or concede that its dams are inadequate to withstand great floods and/or earthquakes that may be reasonably expected to occur in the TVA region under consideration. TVA has a program of inspection and maintenance carried out on a regular schedule to keep its dams safe. Instrumentation of the dams to help keep check on their behavior was installed in many of the dams during original construction. Other instrumentation has been added since and is still being added as the need may appear or as new techniques become available.~~

~~In short, TVA has confidence that its dams are safe against catastrophic destruction by any natural forces that could be expected to occur.~~

~~2.4.4.2.1 Seismic Failure Analysis~~

~~Seismic failure analysis consisted of the following:~~

~~1. Determination of the water level at the plant during one-half the PMF with full reservoirs if its crests were augmented by flood waves from the postulated failure of upstream dams during a one-half SSE.~~

~~2. Determination of the water level at the plant during a 25-year flood with full reservoirs if its crests were augmented by flood waves from the postulated failure of upstream dams during a Safe Shutdown Earthquake (SSE).~~

~~The one-half SSE identified in condition 1 is defined in FSAR Section 2.5.2.4 as having a peak horizontal acceleration value of 0.09 g at the rock foundation. The discussion in Section 2.5.2.4 shows the extreme conservatism contained in the analysis.~~

~~In the 1998 reanalysis all potentially critical seismic events involving dam failure upstream of the plant site were reevaluated. The six events included the postulated one-half SSE failure of (1) Norris, (2) Fontana, (3) Cherokee-Douglas, and (4) Fontana-Hiwassee-Apalachia-Blue Ridge during one-half the PMF; and the postulated SSE failure of (5) Norris-Cherokee-Douglas and (6) Norris-Douglas-Fort Loudoun-Tellico during a 25 year flood.~~

~~Seismic failure of upstream dams during nonflood periods pose no threat to the plant.~~

~~Summary~~

~~A summary of the results of the seismic analysis is given in Table 2.4.4-1. SQN and upstream dams are located as shown on Figure 2.1.1-1. The highest flood level at SQN from different seismic dam failure and flood combinations would be elevation 707.9 from simultaneous failure of Fontana Dam on the Little Tennessee River and Hiwassee, Blue Ridge, and Apalachia Dams on the Hiwassee River during a one-half safe shutdown earthquake coincident with one-half the PMF. This includes improvements resulting from modifications performed for the Dam Safety Program. Wind waves could raise the elevation to 709.6 in the reservoir. Runup could reach elevation 710.4 on a 4:1 slope to elevation 712.8 on a vertical wall in shallow (4.9 feet) water, and to elevation 710.4 on a vertical wall in deep water.~~

~~Only one other seismic dam failure combination with coincident floods could cause elevations above plant grade.~~

~~Plant safety would be assured by shutdown prior to these floods crossing plant grade, elevation 705, using the warning system described in Appendix 2.4A.~~

~~The effect of postulated seismic bridge failure and resulting failure of spillway gate anchors at Watts Bar and Fort Loudoun Dams would not create a safety hazard at SQN.~~

~~Procedures~~

Concrete Structures

The standard method of computing stability is used. The maximum base compressive stress, average base shear stress, the factor of safety against overturning, and the shear strength required for a shear-friction factor of safety of 1 are determined. To find the shear strength required to provide a safety factor of 1, a coefficient of friction of 0.65 is assigned at the elevation of the base under consideration.

~~As stated in Section 2.4.1.2, all of the original stability analyses and postulated dam failure assumptions in the 1998 reanalyses were conservatively assumed to occur in the same manner and in combination with the same postulated rainfall events.~~

The analyses for earthquake are based on the pseudo-static analysis method as given by Hinds [10]

with increased hydrodynamic pressures determined by the method developed by Bustamante and Flores [11]. These analyses include applying masonry inertia forces and increased water pressure to the structure resulting from the acceleration of the structure horizontally in the upstream direction and simultaneously in a downward direction. The masonry inertia forces are determined by a dynamic analysis of the structure which takes into account amplification of the accelerations above the foundation rock.

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

Waves created at the free surface of the reservoir by an earthquake are considered of no importance. Based upon studies by Chopra [12] and Zienkiewicz [13], it is ~~our~~TVA's judgment that before waves of any significant height have time to develop, the earthquake will be over. The duration of earthquake used in this analysis is in the range of 20 to 30 seconds.

Although accumulated silt on the reservoir bottom would dampen vertically traveling waves, the effect of silt on structures is not considered. ~~There is only a small amount of silt now present, and the~~The accumulation rate is slow, as measured by TVA for many years [14].

Embankment

Embankment analysis was made using the standard slip circle method, except for Chatuge and Nottely Dams where the Nemark method for the dynamic analysis of embankment slopes was used. The effect of the earthquake is taken into account by applying the appropriate static inertia force to the dam mass within the assumed slip circle (pseudo-static method).

In the analysis, the embankment design constants used, including the sheer strength of the materials in the dam and the foundation, are the same as those used in the original stability analysis.

Although detailed dynamic soil properties are not available, a value for seismic amplification through the soil has been assumed based on previous studies pertaining to TVA nuclear plants. These studies have indicated maximum amplification values slightly in excess of two for a rather wide range of shear wave velocity to soil height ratios. For these analyses, a straight-line variation is used with an acceleration at the top of the embankment being two times the top of rock acceleration.

As discussed in Section 2.4.3, temporary flood barriers are installed on embankments at Cherokee, Watts Bar, Fort Loudoun and Tellico Reservoirs. However, the temporary flood barriers are not required to be stable following an OBE or SSE and are not assumed to increase the height of the embankments for these loading conditions.

Flood Routing

The runoff model described in Section 2.4.3.3~~, which includes unsteady flow models for critical reservoirs and river reaches, was used to reevaluate plant site flood levels resulting from the postulated SSE and one-half SSE dam failure combinations. The remaining events produced plant site flood levels sufficiently lower than the controlling events and were not evaluated.~~ was used to reevaluate five potentially critical seismic events involving dam failures above the plant. Other events addressed in five earlier studies (the postulated OBE single failures of Watts Bar and Fort Loudoun; the postulated SSE combination failure of Fontana and Douglas, the postulated SSE combination failure of Fontana, Fort Loudoun, and Tellico; the SSE combination failure of Norris, Douglas, Fort Loudoun and Tellico, and the single SSE failure of Norris) produced plant site flood levels sufficiently lower than the controlling events and therefore were not re-evaluated.

The procedures prescribed by Regulatory Guide 1.59 require seismic dam failure to be examined using the SSE coincident with the peak of the 25-year flood, and the OBE coincident with the peak of one-half the PMF.

Reservoir operating procedures used were those applicable to the season and flood inflows.

~~This section was revised with a major rearrangement to locate the controlling events evaluated in the 1998 analysis first and the non-controlling events, which were not re-calculated later. The non-controlling events are left in the SAR for history.~~

One-half SSE Concurrent With One-Half the Probable Maximum Flood

Previous evaluations have been made which determined flood levels at SQN for potentially critical events. Re-evaluations made later using the updated runoff model described in Section 2.4.3.3 and including the Dam Safety Program modifications did not determine flood levels for those events which were previously shown to clearly not be controlling. The 1998 analysis for determining the effects of the Dam Safety Program modifications determined that non-flood related seismic dam failure events clearly pose no threat to the plant. Flood levels were determined for six combined seismic/flood events. Only two of these controlling seismic/flood events would exceed plant grade. These two events consist of multiple dam failures on (1) Little Tennessee/Hiwassee, and (2) Clinch/Upper Tennessee rivers with flood levels at SQN of El. 707.9 and 706, respectively. The following is detailed descriptions of the potentially critical controlling events including reevaluated flood levels, followed by brief descriptions of the non-controlling failure events previously evaluated.

~~Multiple Failures~~

~~Although considered, as discussed in the following paragraphs, TVA believes that multiple dam failures are an extremely unlikely event. TVA's search of the literature reveals no record of failure of concrete dams from earthquake. The postulation of an SSE of 0.18 g acceleration is a very conservative upper limit in itself (as stated in Section 2.5.2). In addition, the SSE must be located in a very precise region to have the potential for multiple dam failures.~~

~~SSE - In order to fail three dams--Norris, Cherokee, and Douglas--the epicenter of a SSE must be confined to a relatively small area, the shape of a football, about 10 miles wide and 20 miles long. In order to fail four dams--Norris, Douglas, Fort Loudoun, and Tellico--the epicenter of an SSE must be confined to a triangular area with sides of approximately 1 mile in length. However, as an extreme upper limit the above two combinations of dams are postulated to fail as well as the combinations of (1) Fort Loudoun, Tellico, and Fontana; (2) Fontana and Douglas; and (3) Fontana and the six Hiwassee River dams. The 1998 re-analysis determined that only the first two combinations are controlling and need to be considered. Only the Norris-Cherokee-Douglas event would exceed plant grade elevation.~~

~~One-half SSE - Attenuation studies of the one-half SSE show that there are three combinations of simultaneous failures of more than one dam which need to be considered with respect to SQN safety which are discussed below. These are (1) Cherokee-Douglas, (2) Fontana-Hiwassee-Apalachia-Blue Ridge, and (3) Hiwassee-Apalachia-Blue Ridge-Ocoee No 1.-Nottely. The 1998 re-analysis determined that only the first two combinations are controlling and need to be considered. Only the Fontana-Hiwasse-Apalachia-Blue Ridge event would exceed plant grade.~~

~~The following descriptions are first for the controlling events for which flood levels were calculated for the 1998 reanalysis, followed by the non-controlling events which were not re-analyzed in 1998.~~

~~One-half SSE~~OBE Concurrent With One-Half the Probable Maximum Flood ~~(Controlling Events)~~

Watts Bar Dam

Stability analyses of Watts Bar Dam powerhouse and spillway sections result in the judgment that these structures will not fail. The analyses show low stresses in the spillway base, and the powerhouse base. Original results are given in Figure 2.4.4-1 and were not updated in the current analysis. Dynamic analysis of the concrete structures resulted in the determination that the base acceleration is amplified at levels above the base. The original slip circle analysis of the earth embankment section results in a factor of safety greater than 1, and the embankment is judged not to fail as shown in Figure 2.4.4-22.

For the condition of peak discharge at the dam for one-half the probable maximum flood the spillway gates are in the wide-open position with the bottom of the gates above the water. This condition was not analyzed because the condition with bridge failure described in the following paragraphs produces the controlling condition.

Analysis of the bridge structure for forces resulting from the OBE, including amplification of acceleration results in the determination that the bridge could fail as a result of shearing the anchor bolts. The downstream bridge girders are assumed to strike the spillway gates. The impact of the girders striking the gates is assumed to fail the bolts that anchor the gate trunnions to the pier anchorages allowing the gates to fall on the spillway crest and be washed into the channel below the dam. The flow over the spillway crest would be the same as that prior to bridge and gate failure, i.e., peak discharge for one-half the probable maximum flood with gates in the wide-open position. Hence, bridge failure will cause no adverse effect on the flood.

Previous evaluations determined that if the dam was postulated to fail from embankment overtopping in the most severe case (gate opening prevented by bridge failure) that the resulting elevations at SQN would be several ft below plant grade elevation 705.0 ft. Therefore, this event was not reevaluated.

Fort Loudoun Dam

Stability analyses of Fort Loudoun Dam powerhouse and spillway sections result in the judgment that these structures will not fail. The analyses show low base stresses, with near two-thirds of the base in compression. The original results given in Figure 2.4.4-2 were not updated for the current analysis.

Slip circle analysis of the earth embankment results in a factor of safety of 1.26, and the embankment is judged not to fail. The original results given in Figure 2.4.4-3 were not updated in the current analysis.

The spillway gates and bridge are of the same design as those at Watts Bar Dam. Conditions of failure during the OBE are the same, and no problems are likely. Coincident failure at Fort Loudoun and Watts Bar does not occur.

For the potentially critical case of Fort Loudoun bridge failure at the onset of the main portion of one-half the probable maximum flood flow into Fort Loudoun Reservoir, in an earlier analysis it was found that the Watts Bar inflows are much less than the condition resulting from simultaneous failure of Cherokee, Douglas, and Tellico Dams as described later.

Tellico Dam

Although, not included in the original analyses for TVA's operating nuclear plants, The concrete portion of Tellico is judged to fail completely because the seismic stability analysis of Tellico is not conclusive. No hydrologic results are given for the single failure of Tellico because the simultaneous failure of Tellico Dam with other dams discussed under multiple failures is more critical. The embankments at Tellico are stable (Figure 2.4.4-23).

~~1.~~ Norris Dam

~~Results of the Norris Dam stability analyses for a typical spillway block and a typical non-overflow section of maximum height are shown on Figure 2.4.4-8. Because only a small percentage of the spillway base is in compression, this structure is judged to fail. The high non-overflow section with a small percentage of the base in compression and with high compressive and shearing stresses is also judged to fail.~~

Although an evaluation made in 1975 by Agbabian Associates concluded that Norris Dam would not fail in an OBE (with one-half PMF) or SSE (with 25-year flood), the original study postulated failure in both seismic events. To be consistent with prior studies, Norris was conservatively postulated to fail.

Figure 2.4.4-~~9~~4 shows the ~~likely~~postulated condition of the dam after OBE failure. ~~Based on stability analyses, the non-overflow blocks remaining in place are judged to withstand the one-half SSE. Blocks 33-44 are judged to fail by overturning.~~

The location of the debris is not based on any calculated procedure of failure because it is believed that this is not possible. It is TVA's judgment, however, that the failure mode shown is one logical assumption; and, although there may be many other logical assumptions, the amount of channel obstruction would probably be about the same.

The discharge rating for this controlling, debris section was developed from a 1:150 scale hydraulic model at the TVA Engineering Laboratory and was verified closely by mathematical analysis.

~~In the hydrologic routing for this failure, Melton Hill Dam was postulated to fail when the flood wave reached headwater elevation 804, based on structural analysis. The headwater at Watts Bar Dam would reach elevation 758.1, 8.9 feet below top of dam. The west saddle dike at Watts Bar Dam would be overtopped and breached. A complete washout of the dike was assumed. The resulting water level at the nuclear plant site is 698.1, 6.9 feet below plant grade 705.~~
No hydrologic results are given for the single failure of Norris Dam because the simultaneous failure of Norris and Tellico Dams, discussed under multiple failures, is more critical.

~~2. Fontana Dam~~

~~Fontana Dam was assumed to fail in the one-half SSE, although no stability analysis was made. Fontana is a high dam constructed with three longitudinal contraction joints in the higher blocks.~~

~~A structural defect in Fontana Dam was found in October of 1972 and consists of a longitudinal crack in three blocks in the curved portion at the left end of the dam (see Figure 2.4.4-16). Strengthening of these blocks by post-tensioning and grouting of the cracks was completed in October 1973 (see Figure 2.4.4-17). Only these three blocks are cracked, and there is no evidence that any other portion of the dam is weakened.~~

~~Studies and tests, undertaken with the concurrence of a board of private consulting engineers, indicate that this cracking was caused by a longitudinal thrust created by a combination of long-time concrete growth and expansion due to temperature rise in the summer months. This thrust tends to push the curved blocks upstream. The studies and tests will continue until there is established a basis for design of permanent measures to control the future behavior of the dam.~~

~~The strengthening work has reestablished the structural integrity of the cracked blocks. Although the joints are keyed and grouted, it is possible that the grouting was not fully effective. Consequently, there is some question as to how this structure will respond to the motion of a severe earthquake. To be conservative, therefore, it is assumed that Fontana Dam will not resist the one-half SSE without failure.~~

~~Figure 2.4.4-16 shows the part of Fontana Dam judged to remain in its original position after failure and the assumed location of the debris of the failed portion. The location of the debris after failure is one logical assumption based on a failure of the dam at the longitudinal contraction joints. There may be other logical assumptions, but the amount of channel obstruction would probably be about the same.~~

~~The higher blocks 9-27, containing either two or three longitudinal joints, are assumed to fail. Right abutment blocks 1-8 and left abutment blocks 28 and beyond were judged to be stable for the following reasons:~~

~~1. Their heights are less than one-half the maximum height of the dam.~~

~~2. None of these blocks have more than one longitudinal contraction joint, and some have no longitudinal joints.~~

~~3. The back slope of Fontana Dam is one on 0.76, which the original stability analysis shows is flatter than that required for stability for the normal static loadings.~~

~~Although not investigated, it was assumed that Nantahala Dam, upstream from Fontana and Santeetlah on a downstream tributary, and the three ALCOA dams, downstream on the Little Tennessee River, Cheoah, Calderwood, and Chilhowee, would fail along with Fontana in the one-half SSE. Instant vanishment was assumed. Tellico and Watts Bar Dam spillway gates would be operable during and after the one-half SSE. Failure of the bridge at Fort Loudoun Dam would render the spillway gates inoperable in the wide-open position.~~

~~The flood wave would overtop Tellico Dam and its saddle dikes. Transfer of water into Fort Loudoun would occur but would not be sufficient to overtop the dam or to prevent failure of Tellico Dam. Tellico was postulated to completely fail. Watts Bar headwater would reach elevation 761.3, 5.7 feet below top of dam. The Watts Bar west saddle dike would be overtopped and breached. A complete washout of the dike was assumed. The elevation at the plant site would be 702.8, 2.2 feet below plant grade.~~

~~3.~~ Cherokee ~~Douglas~~ Dam

~~The simultaneous failure of Cherokee and Douglas Dams could occur when the one-half SSE is located midway between the dams which are just 15 miles apart.~~

Results of the original Cherokee Dam stability analysis for a typical spillway block are shown in Figure 2.4.4-~~10~~5. ~~Based on this analysis, the~~The spillway is judged stable at the foundation base elevation 900.0 ft. Analyses made for other elevations above elevation 900.0 ft, but not shown in Figure 2.4.4-~~10~~5, indicate the resultant of forces falls outside the base at elevation 1010.0 ft. The spillway is assumed to fail at that elevation.

The non-overflow dam is embedded in fill to elevation 981.5 ft and is considered stable below that elevation. However, original stability analysis indicates failure will occur above the fill line.

The powerhouse intake is massive and backed up by the powerhouse. Therefore, it is judged able to withstand the ~~one-half SSE~~OBE without failure.

Results of the original analysis for the highest portion of the south embankment are shown on Figure 2.4.4-~~11~~6. The analysis was made using the same shear strengths of material as were used in the original analysis and shows a factor of safety of 0.85. Therefore, the south embankment is assumed to fail during the one-half SSE. Because the north embankment and ~~saddle dams~~Saddle Dams 1, 2, and 3 are generally about one-half or less as high as the south embankment, they are judged to be stable for the ~~one-half SSE~~OBE.

Figure 2.4.4-~~12~~7 shows the assumed condition of the dam after failure. All debris from the failure of the concrete portion is assumed to be located downstream in the channel at elevations lower than the remaining portions of the dam, and therefore, will not obstruct flow.

No hydrologic results are given for the single failure of Cherokee Dam because the simultaneous failure of Cherokee, Douglas, and Tellico Dams discussed under multiple failures, is more critical.

Douglas Dam

Results of the original Douglas Dam ~~original~~ stability analysis for a typical spillway block are shown in Figure 2.4.4-~~13~~8. The upper part of the Douglas spillway is approximately 12 f~~ee~~t higher than Cherokee, but the amplification of the rock surface acceleration is the same. Therefore, based on the Cherokee analysis, it is judged that the Douglas spillway will fail at elevation 937.0 ft, which corresponds to the assumed failure elevation of the Cherokee spillway.

The Douglas non-overflow dam is similar to that at Cherokee and is embedded in fill to elevation 927.5 ft. It is considered stable below that elevation. However, based on the Cherokee analysis, it is

assumed to fail above the fill line. The abutment non-overflow blocks 1-5 and 29-35, being short blocks, are considered able to resist the ~~one-half SSE~~OBE without failure.

The powerhouse intake is massive and backed up downstream by the powerhouse. Therefore, it is considered able to withstand the ~~one-half SSE~~OBE without failure.

Results of the original analysis of the ~~saddle dam~~Saddle Dam shown on Figure 2.4.4-~~14~~9 indicate a factor of safety of one. Therefore, the ~~saddle dam~~Saddle Dam is considered to be stable for the ~~one-half SSE~~OBE.

Figure 2.4.4-~~15~~10 shows the portions of the dam judged to fail and the portions judged to remain. All debris from the failed portions is assumed to be located downstream in the channel at elevations lower than the remaining portions of the dam and, therefore, will not obstruct flow.

~~These failures, in conjunction with one-half the probable maximum flood, would overtop Fort Loudon for only 6 hours, but would not fail the dam. At Watts Bar the west saddle dike would be overtopped and breached. A complete washout of the dike was assumed. Crest level at SQN would be elevation 701.1, 3.9 feet below plant.~~ No hydrologic results are given for the single failure of Douglas Dam because the simultaneous failure of Cherokee, Douglas, and Tellico Dams as discussed later under multiple failures, is more critical.

Fontana Dam

The original hydrological analysis used a conservative seismic failure condition for Fontana Dam. A subsequent review which takes advantage of later earthquake stability analysis and dam safety modifications performed for the TVA DSP has defined a conservative but less restrictive seismic failure condition at Fontana. This subsequent review used a finite element model for the analysis and considered the maximum credible earthquake expected at the Fontana Dam site. Figure 2.4.4-11 shows the part of Fontana Dam judged to remain in its original position after postulated failure.

No hydrologic results are given for the single failure of Fontana Dam because the simultaneous failure of Fontana and Tellico Dams, as discussed later under multiple failures, is more critical.

Multiple Failures

Previous attenuation studies of the OBE above Watts Bar Dam result in the judgment that the following simultaneous failure combinations require reevaluation:

(1) The Simultaneous Failure of Fontana and Tellico Dams in the OBE Coincident with One-Half PMF

Figure 2.4.4-11 shows the postulated condition of Fontana for the OBE event. Tellico was conservatively postulated to completely fail.

The seismic failure scenario for Fontana and Tellico include postulated simultaneous and complete failure of non-TVA dams on the Little Tennessee River (Cheoah, Calderwood, and Chilhowee) and on its tributaries (Nantahala and Santeetlah). Failure of the bridge at Fort Loudoun Dam would render the spillway gates inoperable in the wide-open position. Watts Bar Dam spillway gates would be operable during and after the OBE.

Watts Bar Dam headwater would reach 756.13 ft, 13.87 ft below the top of the embankment. The West Saddle Dike at Watts Bar Dam with top elevation of 757.0 ft would not be overtopped. The peak discharge at SQN would be 775,899 cfs. The elevation at SQN would be 702.2 ft, 2.8 ft below plant grade elevation 705.0 ft.

(2) The Simultaneous Failure of Fontana, Tellico, Hiwassee, Apalachia, and Blue Ridge Dams in the OBE Coincident with One-Half PMF

Fontana, Tellico, Hiwassee, Apalachia and Blue Ridge Dams could fail when the OBE is located within a flattened oval-shaped area located between Fontana and Hiwassee Dams (Figure 2.4.4-12). Failure scenarios for Fontana, Tellico, Hiwassee, Apalachia, and Blue Ridge Dams include postulated simultaneous failure of non-TVA dams on the Little Tennessee River (Cheoah, Calderwood and Chilhowee) and on its tributaries (Nantahala and Santeetlah).

Based on previous attenuation studies, the OBE event produces maximum ground accelerations of 0.09 g at Fontana, 0.09 g at Hiwassee, 0.07 g at Apalachia, 0.08 g at Chatuge, 0.05 g at Nottely, 0.03 g at Ocoee No.1, 0.04 g at Blue Ridge, 0.04 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. Figure 2.4.4-11 shows the postulated condition of Fontana Dam after failure. Hiwassee, Apalachia, Blue Ridge, and Tellico Dams are postulated to completely fail. Chatuge Dam is judged not to fail in this defined OBE event.

Nottely Dam is a rock-fill dam with large central impervious rolled fill core. The maximum attenuated ground acceleration at Nottely in this event is only 0.05 g. A field exploration boring program and laboratory testing program of samples obtained in a field exploration was conducted. During the field exploration program, standard penetration test blow counts were obtained on both the embankment and its foundation materials. Both static and dynamic (cyclic) triaxial shear tests were made. The Newmark Method of Analysis utilizing the information obtained from the testing program was used to determine the structural stability of Nottely Dam. It is concluded that Nottely Dam can resist the attenuated ground acceleration of 0.054 g with no detrimental damage.

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

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

Fort Loudoun and Watts Bar spillways would remain operable. The Fontana failure wave would transfer water through the canal from Tellico into Fort Loudoun, but it would not be sufficient to overtop Fort Loudoun Dam. The maximum headwater at Fort Loudoun would reach elevation 817.13 ft, 19.87 ft below the top of the dam. Watts Bar headwater would reach elevation 756.13 ft, 13.87 ft below the top of dam. The West Saddle Dike at Watts Bar with a top elevation of 757.00 ft would not be overtopped.

The peak discharge at the SQN site produced by the OBE failure of Fontana, Tellico, Hiwassee, Apalachia, and Blue Ridge coincident with the one-half PMF is 918,880 cfs. The peak elevation is 706.3 ft, 1.3 ft above 705.0 ft plant grade.

(3) The Simultaneous Failure of Norris and Tellico Dams in the OBE Coincident with One-Half PMF

Figure 2.4.4-4 shows the postulated condition of Norris Dam for the OBE event. Tellico was conservatively postulated to completely fail in this event.

In the hydrologic routing for this failure, Melton Hill Dam would be overtopped and was postulated to fail when the flood wave reached headwater elevation 817.0 ft, based on the structural analysis and subsequent structural modifications performed at the dam as a result of the Dam Safety Program.

The headwater at Watts Bar Dam would reach elevation 762.96 ft, 7.04 ft below top of dam. The West Saddle Dike at Watts Bar with top at elevation 757.0 ft would be overtopped and breached. A complete washout of the dike was assumed. Chickamauga headwater would reach 701.05 ft, 4.95 ft below top of dam. The embankments at Nickajack Dam would be overtopped but was postulated not to breach which is conservative.

The peak discharge at the SQN site produced by the OBE failure of Norris and Tellico Dams coincident with the one-half PMF is 912,939 cfs. The peak elevation is 706.3 ft, 1.3 ft above 705.0 ft plant grade.

(4) The Simultaneous Failure of Cherokee, Douglas, and Tellico Dams in the OBE Coincident with One-Half PMF

Figures 2.4.4-7 and 2.4.4-10 show the postulated condition after failure of Cherokee and Douglas Dams, respectively. Tellico was conservatively postulated to completely fail.

In the hydrologic routing for these postulated failures, the headwater at Watts Bar Dam would reach elevation 763.1 ft, 6.9 ft below the top of the dam. The West Saddle Dike at Watts Bar with a top elevation of 757.0 ft would be overtopped and breached. A complete washout of the dike is assumed. Chickamauga Dam headwater would reach 702.95 ft, 3.05 ft below the top of the dam. The embankments at Nickajack Dam would be overtopped but were conservatively postulated not to breach.

The peak discharge at the SQN site produced by the OBE failure of Cherokee, Douglas, and Tellico with the one-half PMF is 930,585 cfs. The peak elevation is 708.6 ft, 3.6 ft above 705.0 ft plant grade. This is the highest flood elevation resulting from any combination of seismic events. The flood elevation hydrograph at the plant site is shown on Figure 2.4.4-18.

~~4. Fontana, Hiwassee, Apalachia, and Blue Ridge Dams~~

~~Fontana, Hiwassee, Apalachia, and Blue Ridge Dams could fail when the one-half SSE is located within the football-shaped area shown in Figure 2.4.4-18.~~

~~This event produces maximum ground accelerations of 0.09 g at Fontana, 0.09 g at Hiwassee, 0.07 g at Apalachia, 0.08 g at Chatuge, 0.05 g at Nottely, 0.03 g at Ocoee No. 1, 0.04 g at Blue Ridge, 0.04 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. Failure is postulated for Fontana and Hiwassee for an earthquake epicenter located anywhere within the football-shaped area shown on Figure 2.4.4-18. Ground accelerations shown for the various dams are maximum that could occur for epicenters located at various points in the described area and would not occur simultaneously. Fort Loudoun, Tellico, and Watts Bar Dams and spillway gates would remain intact. The degree of Fontana failure and likely position of debris are judged to be comparable to that shown for single failure in Figure 2.4.4-16. Hiwassee, Apalachia, and Blue Ridge Dams were assumed to completely disappear. Chatuge was judged not to fail as the acceleration is less than for the one-half SSE centered at the dam.~~

~~Nottely Dam is a rockfill dam with large central impervious rolled fill core. The maximum attenuated ground acceleration at Nottely is only 0.054 g. A field exploration boring program and laboratory testing program of samples obtained in a field exploration was conducted. During the field exploration program, standard penetration tests blow counts were obtained on both the embankment and its foundation materials. Both static and dynamic (cyclic) triaxial shear tests were made. This information was used in the Newmark Method of Analysis. The "Newmark Method of Analysis" (Newmark, N. M., "Effects of Earthquake on Dams of Embankments," Geotechnique 15:140-141, 156, 1965) utilizing the information obtained from the testing program was used to determine the structural stability of Nottely Dam. We conclude Nottely Dam can easily resist the attenuated ground acceleration of 0.054 g with no detrimental damage.~~

~~Ocoee No. 1 Dam is a concrete gravity structure. The maximum attenuated ground acceleration is 0.03 g. The 0.03 g with the proper amplification was used to analyze the structural stability of structures at Ocoee No. 1. The method of analysis used was the same as described previously under "Procedures, Concrete Structures." The analysis shows low stresses with good factors of safety against sliding and overturning. We conclude the dam will not fail.~~

~~In the original analysis, the failure wave hydrograph was approximated for the Hiwassee River at its mouth for the failures of Hiwassee, Apalachia and Blue Ridge Dams. In the 1998 re-analysis an~~

~~unsteady flow model described in Section 2.4.3.3 developed during the dam safety studies was used as an adjunct to route the Hiwassee, Apalachia and Blue Ridge failures.~~

~~In the simultaneous failure of Fontana, Hiwassee, Apalachia, and Blue Ridge Dams, the Fontana failure wave would overtop and fail the Tellico embankments. Transfer of water into Fort Loudoun would occur but would not be sufficient to overtop the dam or to prevent failure of Tellico. Tellico was postulated to completely fail. Watts Bar headwater would reach elevation 761.3, 5.7 feet below top of dam. The west saddle dike at Watts Bar would be overtopped. A complete washout of the dike down to ground elevation was assumed. This flood wave combined with that of Hiwassee, Blue Ridge, and Apalachia Dams would produce a maximum flood level at the plant site of 707.9, 2.9 feet above 705 plant grade. This is the highest flood resulting from any combination of seismic and concurrent flood events. The stage hydrograph at the plant site is shown on Figure 2.4.4-21.~~

SSE Concurrent With 25-Year Flood ~~(Controlling Events)~~

The SSE will produce the same postulated failure of the Fort Loudoun and Watts Bar bridges as described for the OBE described earlier. The resulting flood level at the SQN was not determined because the larger flood during the OBE makes that situation controlling.

Watts Bar Dam

A reevaluation using the revised amplification factors was not made for Watts Bar Dam for SSE conditions. However, even if the dam is arbitrarily removed instantaneously, the level at the SQN based on previous analyses would be below elevation 705.0 ft plant grade.

Fort Loudoun Dam

Results of the original stability analysis for Fort Loudoun Dam are shown on Figure 2.4.4-13. Because the resultant of forces falls outside the base, a portion of the spillway is judged to fail. Based on previous modes of failure for Cherokee and Douglas, the spillway is judged to fail above elevation 750.0 ft as well as the bridge supported by the spillway piers.

The results of the original slip circle analysis for the highest portion of the embankment are shown on Figure 2.4.4-14. Because the factor of safety is less than one, the embankment is assumed to fail.

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

Figure 2.4.4-15 shows the condition of the dam after assumed failure. All debris from the failure of the concrete portions is assumed to be located in the channel below the failure elevations.

No hydrologic routing for the single failure of Fort Loudoun, including the bridge structure, is made because its simultaneous failure with other dams is considered as discussed later in this subparagraph.

Tellico Dam

No hydrologic routing for the single failure of Tellico is made because its simultaneous failure with other dams is more critical as discussed later in this sub-paragraph.

Norris Dam

Although an evaluation made in 1975 by Agbabian Associates concluded that Norris Dam would not fail in the SSE (with 25 year flood), Norris Dam was postulated to fail. The resulting debris downstream would occupy a greater span of the valley cross section than would the debris from the

OBE but with the same top level, elevation 970.0 ft. Figure 2.4.4-16 shows the part of the dam judged to fail and the location and height of the resulting debris.

The discharge rating for this controlling, debris section was developed from a 1:150 scale hydraulic model at the TVA Engineering Laboratory and was verified closely by mathematical analysis. The somewhat more extensive debris in SSE failure restricts discharge slightly compared to OBE failure conditions.

No hydrologic routing for the single failure of Norris was made because the simultaneous failure with Cherokee, Douglas and Tellico Dams, discussed under multiple failures, is more critical.

Cherokee

The SSE is judged to produce the same postulated failure of Cherokee as was described for the OBE. The single failure does not need to be carried downstream because elevations would be lower than the same OBE failure in one-half the probable maximum flood.

Douglas

The SSE is judged to produce the same postulated failure of Douglas as was described for the OBE. The single failure does not need to be carried downstream because elevations would be lower than the same OBE failure in one-half the probable maximum flood.

Multiple Failures

~~5. Norris, Cherokee, and Douglas~~

~~Norris, Cherokee, and Douglas Dams were also postulated to fail simultaneously. Figure 2.4.4-29 shows the location of an SSE, and its attenuation, which produces 0.15 g at Norris, 0.09 g at Cherokee and Douglas, 0.08 g at Fort Loudoun and Tellico, 0.05 g at Fontana, and 0.03 g at Watts Bar. Fort Loudoun, Tellico, and Watts Bar have been judged not to fail for the one-half SSE (acceleration value of 0.09 g) (see following discussion of non-controlling events). The bridge at Fort Loudoun Dam, however, might fail under 0.08 g forces, falling on any open gates and on gate-hoisting machinery. Trunnion anchor bolts of open gates would fail and the gates would be washed downstream, leaving an open spillway. Closed gates could not be opened. The most conservative assumption was used that at the time of the seismic event on the upstream tributary dams, the crest of the 25-year flood would likely have passed Fort Loudoun and flows would have been reduced to turbine capacity. Hence spillway gates would be closed. As stated before, it is believed that multiple dam failure is extremely remote, and it seems reasonable to exclude Fontana on the basis of being the most distant in the cluster of dams under consideration. For the postulated failures of Norris, Cherokee, and Douglas, the portions judged to remain and debris arrangements are as given in Figures 2.4.4-9, 2.4.4-12, and 2.4.4-15, respectively.~~

~~The SSE will produce the same postulated failures of Cherokee and Douglas Dams as were described for the one-half SSE.~~

~~For Norris under SSE conditions, blocks 31-45 (883 feet of length) are judged to fail. The resulting debris downstream would occupy a greater span of the valley cross section than would the debris from the one-half SSE but with the same top level, elevation 970. Figure 2.4.4-28 shows the part of the dam judged to fail and the location and height of the resulting debris. The discharge rating for this controlling debris section was developed from a 1:150 scale hydraulic model at the TVA Engineering Laboratory and was verified closely by mathematical analysis. The somewhat more extensive debris in SSE failure restricts discharge slightly compared to one-half SSE failure conditions.~~

~~The flood for the postulated failure combination would overtop and breach Fort Loudoun Dam. Although transfer of water into Tellico would occur, it would not be sufficient to overtop the dam. At Watts Bar Dam the headwater would reach 764.9, 2.1 feet below the top of the earth embankment of the main dam. However, the west saddle dike at Watts Bar Dam would be overtopped and breached.~~

~~Resulting water surface at SQN would reach elevation 706. This is 1.0 foot higher than plant grade. This is the highest flood resulting from any combination of SSE seismic and flood events. The flood elevation Flow and stage hydrographs at the plant site is shown on Figure 2.4.4-30.~~

~~6. Norris, Douglas, Fort Loudoun, and Tellico~~

~~Norris, Douglas, Fort Loudoun, and Tellico Dams were postulated to fail simultaneously. Figure 2.4.4-31 shows the location of an SSE, and its attenuation, which produces 0.12 g at Norris, 0.08 g at Douglas, 0.12 g at Fort Loudoun and Tellico, 0.07 g at Cherokee, 0.06 g at Fontana, and 0.04 g at Watts Bar. Cherokee is judged not to fail at 0.07 g; Watts Bar has previously been judged not to fail at 0.09 g; and, for the same reasons as given above, it seems reasonable to exclude Fontana in this failure combination. For the postulated failures of Norris, Douglas, Fort Loudoun, and Tellico, the portions judged to remain and the debris arrangements are as given in Figures 2.4.4-9, 2.4.4-15, 2.4.4-26, and 2.4.4-27, respectively. For analysis purposes, Fort Loudoun and Tellico were postulated to fail completely as the portions judged to remain are relatively small.~~

~~The SSE will produce the same postulated failure of Douglas Dam as was described for the one-half SSE.~~

~~Results of the stability analysis for Fort Loudoun Dam are shown on Figure 2.4.4-24. Because the resultant of forces falls outside the base, a portion of the spillway is judged to fail. Based on previous modes of failure for Cherokee and Douglas, the spillway is judged to fail above elevation 750 as well as the bridge supported by the spillway piers.~~

~~The results of the slip circle analysis for the highest portion of the embankment are shown on Figure 2.4.4-25. Because the factor of safety is less than one, the embankment is assumed to fail.~~

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

~~Figure 2.4.4-26 shows the condition of the dam after assumed failure. All debris from the failure of the concrete portions is assumed to be located in the channel below the failure elevations.~~

~~No structural analysis was made for Tellico Dam failure in the SSE. Because of the similarity to Fort Loudoun, the spillway and entire embankment are judged to fail in a manner similar to Fort Loudoun. Figure 2.4.4-27 shows after failure conditions with all debris assumed located in the channel below the failure elevation.~~

~~This postulated failure combination results in Watts Bar headwater elevation 758.9, 8.1 feet below above the top of the embankment of the main dam. The west saddle dike at Watts Bar Dam would be overtopped and breached. A complete washout of the dike was assumed. The resulting water level at SQN would be elevation 699.3, 5.7 feet below plant grade 705.~~

~~One-half SSE Concurrent With One-Half the Probable Maximum Flood (Non-controlling Events-Historical)~~

~~1. Watts Bar Dam~~

~~Stability analyses of Watts Bar Dam powerhouse and spillway sections result in the judgment that these structures will not fail. The analyses show low stresses with about 38 percent of the spillway base in compression and about 42 percent of the powerhouse base in compression. Results are given in Figure 2.4.4-1. Dynamic analysis of the concrete structures resulted in the determination that the base acceleration is amplified at levels above the base.~~

~~The slip circle analysis of the earth embankment section results in a factor of safety of 1.52, and the~~

~~embankment is judged not to fail. Results are given in Figure 2.4.4-2.~~

~~Normally for the condition of peak discharge at the dam for one-half the PMF, the spillway gates would be in the wide open position (Figure 2.4.4-3). But, analysis of the bridge structure for forces resulting from a one-half SSE, including amplification of acceleration results in the determination that the bridge could fail as a result of shearing the anchor bolts. The downstream bridge girders could strike the spillway gates. The impact of the girders striking the gates could fail the bolts which anchor the gate trunnions to the pier anchorages allowing the gates to fall. The flow over the spillway crest would be the same as that prior to bridge and gate failure. Hence, bridge failure will cause no adverse effect on the flood.~~

~~A potentially severe condition is the bridge falling when most spillway gates would be closed. The gate hoisting machinery would be inoperable after being struck by the bridge. As a result, the flood would crest with the gates closed and the bridge deck and girders lying on top of the spillway piers. Analysis of the concrete portions of the dam for the headwater for this condition shows that they will not fail.~~

~~Flood levels at SQN for all the conditions described above is safely below plant grade elevation 705.~~

~~2. Fort Loudoun Dam~~

~~Stability analyses of Fort Loudoun Dam powerhouse and spillway sections result in the judgment that these structures will not fail. The analyses show low base stresses, with near two-thirds of the base in compression. Results are given in Figure 2.4.4-4.~~

~~Slip circle analysis of the earth embankment results in a factor of safety of 1.26, and the embankment is judged not to fail. Results are given in Figure 2.4.4-5.~~

~~The spillway gates and bridge are of the same design as those at Watts Bar Dam. Conditions of failure during a one-half SSE are the same, and no problems are likely. Coincident failure at Fort Loudoun and Watts Bar does not occur.~~

~~For the potentially critical case of Fort Loudoun bridge failure at the onset of the main portion of one-half the probable maximum flood flow into Fort Loudoun Reservoir, it was found that the Watts Bar inflows are much less than the condition resulting from simultaneous failure of Cherokee and Douglas.~~

~~3. Tellico Dam~~

~~No part of Tellico Dam is judged to fail. Results of the stability analyses for a typical non-overflow block and a typical spillway block are shown in Figure 2.4.4-6. The result of the stability analysis of the earth embankment is shown in Figure 2.4.4-7 and indicates a factor of safety of 1.28.~~

~~4. Cherokee Dam~~

~~No hydrologic results are given for the single failure of Cherokee Dam because the simultaneous failure of Cherokee and Douglas is more critical.~~

~~5. Douglas Dam~~

~~No hydrologic results are given for the single failure of Douglas Dam because the simultaneous failure of Cherokee and Douglas is more critical.~~

~~6. Hiwassee River Dams~~

~~Hiwassee Dam was assumed to fail in the one-half SSE. No hydrologic results are given for the single failure of Hiwassee Dam because its simultaneous failure with other dams is more critical.~~

~~7. Apalachia~~

~~Apalachia Dam was assumed to fail in the one-half SSE. No hydrologic results are given for the single failure of Apalachia Dam because its simultaneous failure with other dams is more critical.~~

~~8. Blue Ridge~~

~~Blue Ridge Dam was assumed to fail in the one-half SSE. No hydrologic results are given for the single failure of Blue Ridge Dam because its simultaneous failure with other dams is more critical.~~

~~9. Ocoee No. 1~~

~~Ocoee No. 1 Dam was assumed to fail in the one-half SSE. No hydrologic results are given for the single failure of Ocoee No. 1 Dam because its simultaneous failure with other dams is more critical.~~

~~10. Nottely~~

~~Nottely Dam was assumed to fail in the one-half SSE. No hydrologic results are given for the single failure of Nottely Dam because its simultaneous failure with other dams is more critical.~~

~~11. Chatuge~~

~~Chatuge Dam is a homogeneous, impervious rolled-fill dam. With the epicenter of the one-half SSE located at the dam, the maximum ground acceleration at Chatuge is 0.09 g. Ground accelerations of this magnitude should have no detrimental effects on a well-constructed compacted earthfill embankment. We know of no failures of compacted earth embankment slopes from earthquake motions. Failures to date have been associated with other liquefaction of hydraulic fill embankments of liquefaction of loose granular foundation materials. The rolled embankment materials in Chatuge are not sensitive to liquefaction. To verify these conclusion analysis using the "Newmark Method for the Dynamic Analysis of Embankment Slopes" (Newmark, N. M., "Effects of Earthquake on Dams of Embankments," Geotechnique 15:140-141, 156, 1965) was made to determine the structural stability of Chatuge. We conducted a field exploration boring program and laboratory testing program of samples obtained in the field exploration. During the field exploration program, standard penetration tests blow counts were obtained on both the embankment and its foundation materials. Both static and dynamic (cyclic) triaxial shear tests were made. This information was used in the Newmark Method of Analysis. We concluded from the Analysis that the Chatuge Dam can easily resist the ground acceleration of 0.09 g with no detrimental damage.~~

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

~~Hiwassee, Apalachia, Blue Ridge, Ocoee No.1, and Nottely Dams could fail when the one-half SSE is critically located. All five dams were assumed to completely disappear in this event. Resulting crest level at SQN would be below plant grade 705.~~

~~SSE Concurrent With 25-Year Flood (Non-controlling Events - Historical)~~

~~1. Watts Bar Dam~~

~~A reevaluation was not made for Watts Bar Dam for SSE conditions. A previous evaluation had determined that even if the dam is arbitrarily removed instantaneously, the level at the nuclear plant site would be below plant grade.~~

~~2. Fort Loudoun Dam~~

~~No hydrologic routing for the single failure of Fort Loudoun, including the bridge structure, is made because its simultaneous failure with Tellico and Fontana, as well as with Tellico, Norris, and Douglas, are controlling.~~

~~3. Tellico Dam~~

~~No routing for the single failure of Tellico is made for the reasons given above for Fort Loudoun.~~

~~4. Norris Dam~~

~~This postulated single failure would result in peak headwater at Watts Bar below the top of the earth portions of the dam. Routing was not carried further because it was evident that flood levels at the plant site would be considerably lower than for the Norris failure in the one-half SSE combined with the one-half PMF.~~

~~5. Hiwassee River Dams Considered Separately~~

~~No structural analyses were made for Chatuge, Nottely, Blue Ridge, Ocoee No. 1, Hiwassee, and Apalachia in the SSE. Instead, all six dams were postulated to fail completely.~~

~~No routing for the failure of the six Hiwassee dams alone is made because their simultaneous failure with Fontana is considered as discussed earlier in this subparagraph.~~

~~6. Cherokee, Douglas, and Fontana Considered Separately~~

~~The SSE will produce the same postulated failures of Cherokee, Douglas, and Fontana Dams as were described for the one-half SSE. None of these failures need to be carried downstream, however, because elevations would be lower than the same failures in one-half the probable maximum flood.~~

~~7. Fort Loudoun, Tellico, and Fontana~~

~~An SSE centered between Fontana and the Fort Loudoun-Tellico complex was postulated to fail these three dams. The four ALCOA dams downstream from Fontana and Nantahala, an ALCOA dam, upstream were also postulated to fail completely in this event. Watts Bar Dam and spillway gates would remain intact, but failure of the roadway bridge was postulated which would render the spillway gates inoperable. At the time of seismic failure, discharges would be small in the 25-year flood. For conservatism, Watts Bar gates were assumed inoperable in the closed position after the SSE event. This event would result in a flood level at the nuclear plant site below 705 plant grade.~~

~~8. Douglas and Fontana~~

~~Douglas and Fontana were postulated to fail simultaneously. The location of an SSE required to fail both dams would produce 0.14 g at Douglas, 0.09 g at Fontana, 0.07 g at Cherokee, 0.05 g at Norris, 0.06 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. For the postulated failures of Douglas and Fontana, the portions judged to remain and the debris arrangements are as given in Figures 2.4.4-15 and 2.4.4-16. Fort Loudoun, Tellico, and Watts Bar have previously been judged not to fail for the OBE (0.09 g). The bridge at Fort Loudoun Dam, however, might fail under 0.06 g forces, falling on gates and on gate hoisting machinery. Fort Loudoun gates were assumed inoperable in the closed position following the SSE event. Resulting water surface at SQN would be below plant grade.~~

~~9. Fontana and Hiwassee River Dams~~

~~Fontana and six Hiwassee River dams--Hiwassee, Apalachia, Chatuge, Nottely, Blue Ridge, and Ocoee No. 1--were postulated to fail simultaneously. For the postulated failure of Fontana, the portion judged to remain and the debris arrangements are as given in Figure 2.4.4-16. The six Hiwassee dams were assumed to fail completely. Fort Loudoun, Tellico, and Watts Bar are judged not to fail with all gates operable. The Fontana surge combined with that of the six Hiwassee River dams would reach an elevation at the plant site below the plant grade.~~

~~2.4.4.2.2 Hydrologic Failure Analysis~~

~~All upstream and downstream dams which could have significant influence on flood levels at SQN were examined for potential failure during all flood conditions, which would have the potential to~~

~~produce maximum plant flood levels including the dam PMF at the individual upstream dams. Concrete sections were examined for overturning and horizontal shear and sliding. Spillway gates were examined for stability at potentially critical water levels and against failure from being struck by water borne objects. Locks and lock gates were examined for stability, and earth embankments were examined for erosion due to overtopping.~~

~~During the SQN PMF, the only failure would be the west saddle dike at Watts Bar. Chickamauga Dam would be overtopped but was conservatively assumed not to fail.~~

~~Concrete Section Analysis~~

~~For concrete dam sections, comparisons were made between the original design headwater and tailwater levels and those that would prevail in the PMF. If the overturning moments and horizontal forces were not increased by more than 20 percent, the structures were considered safe against failure. All upstream dams passed this test except Douglas, Fort Loudoun, and Watts Bar. Original designs showed the spillway sections of these dams to be most vulnerable. These spillway sections were examined in further detail and judged to be stable.~~

~~Spillway Gates~~

~~During peak PMF conditions the radial spillway gates of Fort Loudoun and Watts Bar Dams will be wide open with flow over the gates and under the gates. For this condition both the static and dynamic load stresses in the main structural members of the gate will be less than the yield stress by a factor of three. The stress in the trunnion pin is less than the allowable design stress by a factor greater than 10. The trunnion pin is prevented from dislodgment by a key into the gate anchorage assembly and fitting into a slot in the pin.~~

~~The gates were also investigated for the condition when rising headwater level first begins to exceed the bottom of the gates in the wide-open position. This condition produces the largest forces tending to rotate the radial gates upward. In the wide-open position the gates are dogged against steel gate stops anchored to the concrete piers. The stresses in the gate stop members are less than the yield stress of the material by a factor of 2.~~

~~It is concluded that the above-listed margins are sufficient to provide assurance also that the gates will not fail as a result of additional stresses which may result from possible vibrations of the gates acting as orifices.~~

~~Waterborne Objects~~

~~Consideration has been given to the effect of water borne objects striking the spillway gates and bents supporting the bridge across Watts Bar Dam at peak water level at the dam. The most severe potential for damage would be by a barge which has been torn loose from its moorings and floats into the dam.~~

~~Should the barge approach the spillway portion of the dam end on, one bridge bent could be failed by the barge and two spillway gates could be damaged and possibly swept away. The loss of one bridge bent will not collapse the bridge because the bridge girders are continuous members and the stress in the girders will be less than the ultimate stress for this condition of one support being lost. Should two gates be swept away, the nappe of the water surface over the spillway weir would be such that the barge would be grounded on the tops of the concrete spillway weirs and provide a partial obstruction to flow comparable to unfailed spillway gates. Hence the loss of two gates from this cause will have little effect on the peak flow and elevation.~~

~~Should the barge approach the spillway portion broadside, two and possibly three bridge bents could be failed. For this condition, the bridge would collapse on the barge and the barge would be grounded on the tops of the spillway weirs. This would be probable because the approach velocity of the barge would be from 4-to-7 miles per hour and the bottom of the barge would be about six inches above the tops of the weirs. For this condition the barge would be grounded before striking the spillway gates~~

~~because the gates are about 20 feet downstream from the leg of the upstream bridge bents.~~

~~Lock Gates~~

~~The lock gates at Fort Loudoun, Watts Bar, and Chickamauga were examined for possible failure with the conclusion that no potential for failure exists because the gates are designed for a differential hydrostatic head greater than that which exists during the probable maximum flood.~~

~~Embankment Breaching~~

~~In the 1998 reanalysis, the only embankment failure would be the west saddle dike at Watts Bar Dam. Chickamauga Dam, downstream of the plant, would be overtopped but was assumed not to fail. This is conservative as failure of Chickamauga Dam would slightly lower flood elevations at the plant.~~

~~The adopted relationship to compute the rate of erosion in an earth dam failure is that developed and used by the Bureau of Reclamation in connection with its safety of dams program [16]. The expression relates the volume of eroded fill material to the volume of water flowing through the breach. The equation is:~~

$$\frac{Q_{soil}}{Q_{water}} = Ke^{-x} \quad (1)$$

~~where~~

~~Q_{soil} = Volume of soil eroded in each time period~~

~~Q_{water} = Volume of water discharged each time period~~

~~K = Constant of proportionality, 1 for the soil and discharge relationships in this study~~

~~e = Base of natural logarithm system~~

$$X = \frac{b}{H} \tan \phi_d$$

~~Where~~

~~b = Base length of overflow channel at any given time~~
~~H = Hydraulic head at any given time~~
~~ϕ_d = Developed angle of friction of soil material. A conservative value of 13 degrees was adopted for materials in the dams investigated.~~

~~Solving the equation, which was computerized, involves a trial and error procedure over short depth and time increments. In the program, depth changes of 0.1 foot or less are used to keep time increments to less than one second during rapid failure and up to about 350 seconds prior to breaching.~~

~~The solution of an earth embankment breach begins by solving the erosion equation using a headwater elevation hydrograph assuming no failure. Erosion is postulated to occur across the entire earth section and to start at the downstream edge when headwater elevations reached a selected depth above the dam top elevation. Subsequently, when erosion reaches the upstream edge of the embankment, breaching and rapid lowering of the embankment begins. Thereafter, computations include headwater adjustments for increased reservoir outflow resulting from the breach.~~

~~Watts Bar West Saddle Dike Embankment Failure~~

~~Figure 2.4.4-37 is a general plan of Watts Bar showing elevations and sections. Figure 2.4.4-38 is a topographic map of the general vicinity of Watts Bar Dam. Figure 2.4.4-39 is a general plan and section of the west saddle dike.~~

~~The west saddle dike was examined and found subject to failure from overtopping. This failure was assumed to be a complete washout and add to the discharge from Watts Bar Dam.~~

~~Some verification for the breaching computational procedures illustrated above was obtained by comparison with actual failures reported in the literature and in informal discussion with hydrologic engineers. These reports show that overtopped earth embankments do not necessarily fail. Earth embankments have sustained overtopping of several feet for several hours before failure occurred. An extreme example is Oros earth dam in Brazil [17] which was overtopped to a depth of approximately 2.6 feet along a 2,000-foot length for 12 hours before breaching began. Once an earth embankment is breached, failure tends to progress rapidly, however. How rapidly depends upon the material and headwater depths during failure. Complete failures computed in this and other studies have varied from about one-half to six hours after initial breaching. This is consistent with actual failures.~~

~~Chickamauga Embankment Failure~~

~~In the original analysis, the failure of earth embankments at Chickamauga Dam, 13.5 miles downstream from SQN, reduced reduce flood levels at the plant by 0.9 feet. Future embankment improvements are planned for Chickamauga Dam, which if implemented, would prevent failure. Therefore, although overtopped in the PMF, the dam was assumed not to fail in determining flood elevations at the plant. This assumption is conservative.~~

TVA considered the following multiple SSE dam failure combinations.

(5) The Simultaneous Failure of Norris, Cherokee, Douglas and Tellico Dams in the SSE Coincident with 25-year Flood

The SSE must be located in a very precise region to have the potential for multiple dam failures. In order to fail Norris, Cherokee, Douglas, and Tellico Dams, the epicenter of SSE must be confined to a relatively small area the shape of a football, about 10 miles wide and 20 miles long.

Figure 2.4.4-17 shows the location of an SSE, and its attenuation, which produces 0.15 g at Norris, 0.09 g at Cherokee and Douglas, 0.08 g at Fort Loudoun and Tellico, 0.05 g at Fontana, and 0.03 g at Watts Bar. Fort Loudoun and Watts Bar have previously been judged not to fail for the OBE (0.09 g). The bridge at Fort Loudoun Dam, however, might fail under 0.08 g forces, falling on any open gates and on gate hoisting machinery. Trunnion anchor bolts of open gates would fail and the gates would be washed downstream, leaving an open spillway. Closed gates could not be opened. By the time of the seismic event at upstream tributary dams the crest of the 25 year flood would likely have passed Fort Loudoun and flows would have been reduced to turbine capacity. Hence, spillway gates would be closed. As stated before, it is believed that multiple dam failure is extremely remote, and it seems reasonable to exclude Fontana on the basis of being the most distant in the cluster of dams under consideration. For the postulated failures of Norris, Cherokee, and Douglas the portions judged to remain and debris arrangements are as given in Figures 2.4.4-16, 2.4.4-7, and 2.4.4-10, respectively. Tellico is conservatively postulated to completely fail.

As discussed in Section 2.4.3, temporary flood barriers are installed on embankments at Fort Loudoun and Tellico Reservoirs. The temporary flood barriers are assumed to fail in the SSE and are thus not credited for increasing the height of the Fort Loudoun or Tellico Reservoirs embankments. The flood for this postulated failure combination would overtop and breach the south embankment and Marina Saddle Dam at Fort Loudoun. At Watts Bar Dam, the headwater would reach elevation 765.54 ft, 4.46 ft below the top of the earth embankment of the main dam. However, the West Saddle Dike with top at elevation 757.0 ft would be overtopped and breached. The headwater at Chickamauga Dam would reach elevation 701.14 ft, 4.86 ft below top of dam. The embankments at Nickajack Dam would be

overtopped but was conservatively postulated not to breach.

The maximum discharge at SQN would be 974,937 cfs. The elevation at the plant site would be 706.0 ft, 1.0 ft above 705.0 ft plant grade. This is the highest flood elevation resulting from any combination of seismic events.

In addition to the SSE failure combination of Norris, Cherokee, Douglas, and Tellico identified as the critical case, three other combinations were evaluated in earlier studies. These three originally analyzed combinations produced significantly lower elevations and were therefore not reevaluated. These include the following:

1. Norris, Douglas, Fort Loudoun, and Tellico
2. Fontana, Fort Loudoun, and Tellico
3. Fontana and Douglas

In order to fail Norris, Douglas, Fort Loudoun, and Tellico Dams, the epicenter of an SSE must be confined to a triangular area with sides of approximately one mile in length. However, as an extreme upper limit the above combination of dams is postulated to fail as well as the combination of (1) Fontana, Fort Loudoun, and Tellico; and (2) Fontana and Douglas.

Norris, Douglas, Fort Loudoun, and Tellico Dams were postulated to fail simultaneously. Figure 2.4.4-19 shows the location of an SSE, and its attenuation, which produces 0.12 g at Norris, 0.08 g at Douglas, 0.12 g at Fort Loudoun and Tellico, 0.07 g at Cherokee, 0.06 g at Fontana, and 0.04 g at Watts Bar. Cherokee is judged not to fail at 0.07 g; Watts Bar has previously been judged not to fail at 0.09 g; and, for the same reasons as given above, it seems reasonable to exclude Fontana in this failure combination. For the postulated failures of Norris, Douglas, Fort Loudoun, and Tellico, the portions judged to remain and the debris arrangements are as given in Figures 2.4.4-16, 2.4.4-10, 2.4.4-15 and 2.4.4-20 for single dam failure. Fort Loudoun and Tellico were postulated to fail completely as the portions judged to remain are relatively small. This combination was not reevaluated because previous analysis showed it was not controlling.

An SSE centered between Fontana and the Fort Loudoun-Tellico complex was postulated to fail these three dams. The four ALCOA dams downstream from Fontana and Nantahala, a Duke Energy dam (formerly ALCOA) upstream were also postulated to fail completely in this event. Watts Bar Dam would remain intact. This flood level was not reevaluated because previous analysis showed it was not controlling.

Douglas and Fontana Dams were postulated to fail simultaneously. Figure 2.4.4-21 shows the location of an SSE and its attenuation, which produces 0.14 g at Douglas, 0.09 g at Fontana, 0.07 g at Cherokee, 0.05 g at Norris, 0.06 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. For the postulated failures of Douglas and Fontana Dams, the portions judged to remain and the debris arrangements for Douglas Dam are as given in Figures 2.4.4-10 and 2.4.4-11 for Fontana dam failure. Fort Loudoun and Watts Bar Dams have previously been judged not to fail for the OBE (0.09 g). Postulation of Tellico failure in this combination has not been evaluated but is bounded by the SSE failure of Norris, Cherokee, Douglas and Tellico.

2.4.4.~~3~~2 Unsteady Flow Analysis of Potential Dam Failures

Unsteady flow routing techniques [26] were used to evaluate plant site flood levels from postulated seismically induced dam failures wherever their inherent accuracy was needed. ~~For PMF determinations unsteady~~ In addition to the flow models described in Section 2.4.3.3, ~~were used. For routing floods from postulated seismically induced dam failures of tributary dams, additional unsteady flow models were used as adjuncts to those described in Section 2.4.3.3~~models described below were used to develop the outflow hydrographs from the postulated dam failures. The HEC-HMS storage routing was used to compute the outflow hydrograph from the postulated failure of each dam except main river dams. In the case of dams which were postulated to fail completely (Hiwassee, Apalachia and Blue Ridge), HEC-RAS or SOCH was used to develop the outflow hydrograph. For Tellico Dam, the complete failure was analyzed with the SOCH model.

~~Unsteady flow techniques were applied in Norris Reservoir. The Norris Reservoir model was developed in sufficient detail to define the manner in which the reservoir would supply and sustain outflow following postulated dam failure. The model was verified by comparing its routed headwater level in the one-half PMF with those using storage-routing techniques. Headwater level agreed within a foot, and the model was considered adequate for the purpose.~~

~~Unsteady flow techniques were also applied in Cherokee, Douglas, and Fontana Reservoirs. The reservoir models were developed in sufficient detail to define the manner in which the reservoirs would supply and sustain outflow following postulated dam failure.~~ The failure time and initial reservoir elevations for each dam were determined from a pre-failure TRBROUTE analysis. HEC-HMS was used to develop the post failure outflow hydrographs based on the previously determined dam failure rating curves. The outflow hydrographs were validated by comparing the HEC-HMS results with those generated by simulations using TRBROUTE.

2.4.4.~~4~~3 Water Level at Plant Site

~~Maximum water level at the plant from different postulated combinations of seismic dam failures coincident with floods would be elevation 707.9, excluding wind wave effects. It would result from the one-half SSE failure of Fontana, Hiwassee, Apalachia, and Blue Ridge Dams coincident with one-half the probable maximum flood. March wind with one percent exceedance probability over the 1.4-mile effective fetch from the critical north-northwest direction is 26 miles per hour over land. This would cause reservoir waves to reach elevation 709.6. Runup could reach elevation 710.4 on a smooth 4:1 slope, elevation 712.8 on a vertical wall in shallow (4.9 feet) water, and elevation 710.4 on a vertical wall in deep water.~~The unsteady flow analyses of the five postulated combinations of seismic dam failures coincident with floods analyzed yields a maximum elevation of 708.6 ft at SQN excluding wind wave effects. The maximum elevation would result from the OBE failure of Cherokee, Douglas and Tellico Dams coincident with the one-half PMF flood postulated to occur in March. Table 2.4.4-1 provides a summary of flood elevations determined for the five failure combinations analyzed.

Coincident wind wave activity for the PMF is described in Section 2.4.3.6. Wind waves were not computed for the seismic events, but superimposed wind wave activity from guide specified two-year wind speed would result in water surface elevations several ft below the PMF elevation 722.0 ft described in section 2.4.3.

2.4.5 Probable Maximum Surge and Seiche Flooding ~~(HISTORICAL INFORMATION)~~

Chickamauga Lake level during non-flood conditions ~~could be no higher than elevation 685.44, top of gates, and is not likely to~~ would not exceed elevation 682.5 ft, normal ~~summer~~maximum pool level, for any significant time. No conceivable ~~hurricane or cyclonic-type winds~~meteorological conditions could produce ~~the over 20 feet of wave height required to~~a seiche nor reservoir operations a surge which would reach plant grade elevation 705.0 ft, some 22 ft above normal maximum pool level.

2.4.6 Probable Maximum Tsunami Flooding ~~(HISTORICAL INFORMATION)~~

Because of its inland location, SQN is not endangered by tsunami flooding.

2.4.7 Ice ~~Flooding and Landslides (HISTORICAL INFORMATION)~~Effects

Because of the location in a temperate climate~~,~~ significant amounts of ice do not form on ~~the Tennessee Valley rivers and lakes. SQN is in no danger from ice flooding.~~ lakes and rivers in the plant vicinity and ice jams are not a source of major flooding.

Flood waves from landslides into upstream reservoirs pose no danger because of the absence of major elevation relief in nearby upstream reservoirs and because the prevailing thin soils offer small slide volume potential compared to the available detention space in reservoirs.