2.4 HYDROLOGIC ENGINEERING

Watts Bar Nuclear Plant is located on the right bank of Chickamauga Lake at Tennessee River Mile (TRM) 528 with plant grade at elevation 728 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 Regulatory Guide 1.59, Revision 2, August 1977.

Determination of the maximum flood level included consideration of postulated dam failures from seismic and hydrologic causes. The maximum flood Elevation 738.8 would result from an occurrence of the probable maximum storm. Coincident wind wave activity results in wind waves of up to 2.2 ft. (crest to trough). Run up on the 4:1 slopes approaching the Diesel Generator Building reaches Elevation 741.2. Wind wave run up on the critical wall of the Intake Pumping Structure reaches Elevation 741.0 and wind wave run up on the walls of the Auxiliary, Control and Shield Buildings reaches Elevation 740.6.

The nearest surface water user located downstream from Watts Bar Nuclear Plant is Dayton, Tennessee, at TRM 503.8, 24.2 miles downstream. All surface water supplies withdrawn from the 58.9 mile reach of the mainstream of the Tennessee River between Watts Bar Dam (TRM 529.9) and Chickamauga Dam (TRM 471.0) are listed in Table 2.4-1.

The probable minimum flow past the site is estimated to be 2000 cfs, which is more than adequate for plant water requirements.

2.4.1 Hydrological Description

2.4.1.1 Sites and Facilities

The location of key plant structures and their relationship to the original site topography is shown on Figure 2.1-5. The structures which have safety-related equipment and systems are indicated on this figure and are tabulated below along with the elevation of exterior accesses.

Structure	Access	Accesses	Elev.
Intake Pumping	(1) Access Hatches	3	728.0
Structure	(2) Stairwell Entrances	2	741.0
	(3) Access Hatches	6	741.0
Auxiliary and	(1) Door to Turbine Bldg.	1	708.0
Control Bldgs.	(2) Door to Service Bldg.	2	713.0
	(3) Railroad Access Opening	1	729.0
	(4) Door to Turbine Bldg.	2	729.0

Structure	Access	Accesses	Elev.
	(5) Emergency Exit	1	730.0
	(6) Door to Turbine Bldg.	2	755.0
Shield Building			
	(1) Personnel Lock	1	714.0
	(2) Equipment Hatch	1	753.0
	(3) Personnel Lock	1	755.0
Diesel Generator	(1) Equipment Access Doors	4	742.0
Building	(2) Emergency Exits	4	742.0
	(3) Personnel Access Door	1	742.0
	(4) Emergency Exit	1	760.5

Exterior accesses are also provided to each of the Class 1E electrical systems manholes and handholes at elevations varying from 714.5 feet MSL to 728.5 feet MSL, depending upon the location of each structure.

The relationship of the plant site to the surrounding area can be seen in Figures 2.1-4a and 2.1-5. 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 Watts Bar Nuclear Plant site, along with the Watts Bar Dam Reservation, comprises approximately 1770 acres on the west bank of Chickamauga Lake at TRM 528. As shown by Figure 2.1-4a, the site is on high ground with the Tennessee River being the major potential source of flooding. The Watts Bar Nuclear Plant 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).

The Tennessee River above the Watts Bar plant site drains 17,319 square miles. Watts Bar Dam, 1.9 miles upstream, has a drainage area of 17,310 square miles. Chickamauga Dam, the next dam downstream, has a drainage area of 20,790 square miles. Two major tributaries, 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. About 20% of the watershed rises above elevation 3,000 with a maximum elevation of 6684 at Mt. Mitchell, North Carolina. The watershed is about 70% forested with much of the mountainous area being 100% forested.

The climate of the watershed is humid temperate. Above Watts Bar Dam annual rainfall averages 50 inches and varies from a low of 40 inches at sheltered locations within the mountains to high spots of 90 inches on the southern and eastern divide. Rainfall occurs fairly evenly throughout the year. The lowest monthly average is 2.8 inches in October. The highest monthly average is 5.4 inches in July, with March a close second with an average of 5.1 inches.

Major flood-producing storms are of two general types: the cool-season, winter type, and the warm-season, hurricane type. Most floods at Watts Bar Nuclear Plant, 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 about 14 inches annually above the plant. Snowfall above the 3,000-foot elevation averages 22 inches annually. The highest average annual snowfall in the basin is 63 inches at Mt. Mitchell, the highest point east of the Mississippi River. 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, 64 miles downstream from Watts Bar Nuclear Plant.

Chickamauga Dam, 57 miles downstream, affects water surface elevations at Watts Bar Nuclear Plant. Normal full pool elevation is 682.5 feet. 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 36,050 acres, with a volume of 622,500 acre-feet. The reservoir has an average width of nearly 1 mile, ranging from 700 feet to 1.7 miles. At the Watts Bar site the reservoir is about 1100 feet wide with depths ranging between 18 feet and 26 feet at normal pool elevation.

There are 12 major dams (South Holston, Boone, Fort Patrick Henry, Watauga, Fontana, Norris, Cherokee, Douglas, Tellico, Fort Loudoun, Melton Hill, and Watts Bar) in the TVA system upstream from Watts Bar Nuclear Plant, ten of which (those previously identified excluding Fort Partick Henry and Melton Hill) provide about 4.4 million acre-feet of reserved flood-detention (March 15) capacity during the main flood season. Table 2.4-2 lists pertinent data for TVA's dams and reservoirs. Figure 2.4-2 presents a simplified flow diagram for the Tennessee River system. Table 2.4-3 provides the relative distances in river miles of upstream dams to the Watts Bar Nuclear Plant site. Details for TVA dam outlet works are provided in Table 2.4-4. In addition, there are six major dams owned by the Aluminum Company of America (ALCOA). The ALCOA reservoirs often contribute to flood reduction, but they do not have dependable reserved flood detention capacity. Table 2.4-5 lists pertinent data for the ALCOA dams and Walters Dam (Waterville Lake). The locations of these dams are shown on Figure 2.1-1.

Flood control above the plant is provided largely by eight 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 3,937,400 acre-feet of detention capacity equivalent to 5.5 inches on the 13,508-square-mile area they control. This is 89% of the total available above the plant. The two main river reservoirs, Fort Loudoun and Watts Bar, provide 490,000 acre-feet equivalent to 2.4 inches on the remaining 3,802-square-mile area above Watts Bar Dam.

The flood detention capacity reserved in the TVA system varies seasonally, with the greatest amounts during the January through March flood season. Figure 2.4-3 (12 sheets) shows the reservoir seasonal operating guides for reservoirs above the plant site. Table 2.4-6 shows the flood control reservations at the multiple-purpose projects above Watts Bar Nuclear Plant at the beginning and end of the winter flood season and in the summer. Total assured system detention capacity above Watts Bar Dam varies from 4.9 inches on January 1 to 4.8 inches on March 15 and decreasing to 1.5 inches during the summer and fall. Actual detention capacity may exceed these amounts, depending upon inflows and power demands.

Chickamauga Dam, the headwater 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-feet equivalent to 5.9 inches on the 1,200-square-mile controlled area. Chickamauga Dam contains 345,300 acre-feet of detention capacity on March 15 equivalent to 2.8 inches on the remaining 2,280 square miles. Figure 2.4-3 (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 2.4-4 (13 sheets).

Daily flow volumes at the plant, for all practical purposes, are represented by discharges from Watts Bar Dam with a drainage area of 17,310 square miles, only 9 square miles less than at the plant. Momentary flows at the nuclear plant site may vary considerably from daily averages, depending upon turbine operations at Watts Bar and Chickamauga Dams. There may be periods of several hours when no releases from either or both Watts Bar and Chickamauga Dams occur. Rapid turbine shutdown at Chickamauga may sometimes cause periods of reverse flow in Chickamauga Reservoir.

Based upon Watts Bar Dam discharge records since dam closure in 1942, the average daily streamflow at the plant is 27,100 cfs. The maximum daily discharge was 208,400 cfs on May 8, 1984. Daily average releases of zero have been recorded on seven occasions during the past 51 years. Flow data for water years 1960–2010 with regulation essentially equivalent to present conditions indicate an average rate of about 23,000 cfs during the summer months (May-October) and about 31,500 cfs during the winter months (November-April). Flow durations based upon Watts Bar Dam discharge records for the period 1960–2010 are tabulated below:

Average Daily Discharge, cfs	Percent of Time Equalled or Exceeded
5,000	97.4
10,000	87.9
15,000	77.5
20,000	64.2
25,000	48.5
30,000	33.4
35,000	21.4

Channel velocities at the Watts Bar site average about 2.3 fps under normal winter conditions. Because of lower flows and higher reservoir elevations in the summer months, channel velocities average about 1.0 fps.

The Watts Bar plant site is underlain by geologic formations belonging to the lower Conasauga Formation of Middle Cambrian age. The formation consists of interbedded shales and limestones overlain by alluvial material averaging 40 feet in thickness. Ground water yields from this formation are low.

All surface water supplies withdrawn from the 58.9 mile reach of the mainstream of the Tennessee River between Watts Bar Dam (TRM 529.9) and Chickamauga Dam (TRM 471.0) are listed in Table 2.4-1. See Section 2.4.13.2 for description of the ground water users in the vicinity of the Watts Bar site.

2.4.2 Floods

2.4.2.1 Flood History

The nearest location with extensive formal flood records is 64 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 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 prior to construction of Tellico Dam. Tellico Dam provides additional reserved flood detention capacity; however, the percentage increase in the total detention capacity above the Watts Bar site is small. Therefore, flood records for the period 1952 to date can be considered representative of prevailing conditions. Table 2.4-7 provides annual peak flow data at Chattanooga. Figure 2.4-5 shows the known flood experience at Chattanooga in diagram form. The maximum known flood under natural conditions occurred in 1867. This flood was estimated to reach elevation 716.3 at Watts Bar Nuclear Plant site with a discharge of about 440,000 cfs. The maximum flood under present-day regulation reached Elevation 696.95 at the site on March 17, 1973.

The following tabulation lists the highest floods at Watts Bar Dam (TRM 529.9) tailwater located upstream of Watts Bar Nuclear Plant site under present-day regulation:

Date	Elevation, Feet	Discharge, cfs
February 2, 1957	No Record	157,600
November 19, 1957	No Record	151,600
March 13, 1963	694.75	167,700
December 31, 1969	693.28	167,300
March 17, 1973	696.95	184,800
May 28, 1973	695.24	175,200
April 5, 1977	694.79	181,600
May 8, 1984	698.23	214,100
April 20, 1998	694.67	167,500
May 7, 2003	694.17	153,100

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 Watts Bar project to conform with 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 sub-watersheds including seasonal variations and potential consequent dam failures and (2) dam failures in a postulated SSE or 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 maximum PMF plant site flood level is Elevation 738.8. This elevation would result from the PMP critically centered on the watershed as described in Section 2.4.3.

Wind waves based on an overland wind speed of 21 miles per hour were assumed to occur coincident with the flood peak. This would create maximum waves up to 2.2 feet 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 728. See Section 2.4.10 for more specific information.

Other rainfall floods will also exceed plant grade Elevation 728 and require plant shutdown. Section 2.4.14 describes emergency protective measures to be taken in flood events exceeding plant grade.

Seismic and flood events could cause dam failure surges exceeding plant grade Elevation 728. Section 2.4.14 describes emergency protective measures to be taken in seismic events exceeding plant grade.

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 structure with those accesses and penetrations below the maximum flood level designed and constructed as watertight elements. The Diesel Generator Building and Essential Raw Cooling Water (ERCW) pumps are located above this flood level, thereby providing protection from flooding.

Wind wave run up during the PMF at the Diesel Generator Building would reach Elevation 741.2 which is 0.8 foot below the operating floor. Consequently, wind wave run up will not impair the safety functions of the Diesel Generator Building.

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 ERCW pumps are structurally protected from wind waves. Therefore, the safety function of the ERCW pumps will not be affected by floods or flood-related conditions.

The 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.

The electrical equipment room of the intake pumping structure will flood at Elevation 728. However, the design basis flood level for the remaining structure is Elevation 741.0. The Auxiliary and Control Buildings will flood with the water level at Elevation 729. The design basis flood level for the Auxiliary, Control, and Shield Buildings is Elevation 740.6. The Diesel Generator Building is located above the design basis flood level of Elevation 741.2.

2.4.2.3 Effects of Local Intense Precipitation

All streams in the vicinity of the plant shown on Figure 2.1-4a were investigated, including Yellow Creek, with probable maximum flows from a local storm and from breaching of the Watts Bar Dam west saddle dike and were found not to create potential flood problems at the plant. Local drainage which required detailed design is from the plant area itself and from a 150-acre area north of the plant.

The underground storm drainage system is designed for a maximum 1-hour rainfall of 4 inches. The 1-hour rainfall with 1% exceedance frequency is 3.3 inches. 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 accesses given in Section 2.4.1.1. The exterior accesses that are below the grade elevation for that specific structure exit from that structure into another structure and are not exterior in the sense that they exit or are exposed to the environment. For any access exposed to the environment and located at grade elevation, sufficient drainage is provided to prevent water from entering the opening. This is accomplished by sloping away from the opening.

PMP for the plant drainage systems has been defined for TVA by the Hydrometeorological Branch of the National Weather Service and is described in Hydrometeorological Report No. 56.

Ice accumulation would occur only at infrequent intervals because of the temperate climate. Maximum winter precipitation concurrent with ice accumulation would impose less severe conditions on the drainage system than would the PMF.

Figure 2.4-40a (sheet 1) shows the Watts Bar site grading and drainage system and building outlines for the main plant area. Direction of flow for runoff has been indicated by arrows. Figures 2.4-40a (sheets 2 & 3) show paved and unpaved areas. Figure

2.4-40b shows the Watts Bar general plan; Figure 2.4-40c shows the site grading and drainage system for the area north and northwest of the plant along with the outline of the low-level radwaste storage facility. The 150-acre drainage area north of the site has been outlined on Figure 2.4-40b with direction of flow for runoff indicated by arrows.

Figure 2.4-40d (three sheets) shows the plans and profiles for the perimeter roads; Figure 2.4-40e (two sheets) shows the plan and profile for the access highway. Figure 2.4-40f (three sheets) shows the plan, sections, and profiles for the main plant railroad tracks. Figure 2.4-40g (three sheets) shows the yard grading, drainage, and surfacing for the switchyard.

In testing the adequacy of the site drainage system, all underground drains were assumed clogged. Peak discharges were evaluated using storm intensities for the maximum 1-hour rainfall obtained from the PMP mass curve shown on Figure 2.4-40h. Runoff was assumed equal to rainfall. Each watershed was analyzed using the more appropriate of 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 or (2) when ponding or reservoir-type conditions controlled, storage routing the inflow hydrograph equivalent to the PMP hydrograph using 2-minute time intervals. Computed maximum water surface elevations are below critical floor Elevation 729. The separate watershed areas are numbered for identification on Figure 2.4-40a.

Runoff from the employee parking lot and the areas south of the office building and west of the Turbine Building (area 1) will flow along the perimeter road west of the switchyard and drain into the area surrounding the chemical holdup ponds. The control is the drainage ditch and road which acts as a channel between the west end of the switchyard and the embankment to the west. To be conservative it was assumed water would not flow into the switchyard. Maximum water surface elevations at the office and Turbine Buildings computed using method (1) were less than 729.

Flow from the area west of the Service, Auxiliary, Reactor, and Diesel Generator Buildings and north of the office building and gatehouse (area 2) will drain along and then across the perimeter road, flow west through a swale and across the low point in the access road. The swale and the roads have sufficient capacity to keep water surface elevations below 729 at all buildings. Method (1) was used in this analysis.

The area east of the Turbine, Reactor, and Diesel Generator Buildings (area 3) forms a pool bounded by the main and transformer yard railroad tracks with top of rail elevations at 728.00 and 728.25 respectively. Method (2) was used to route the inflow hydrograph through this pool from an initial elevation of 728.00 with outflow over the railroads. Maximum water surface elevations at the Turbine and Reactor Buildings were less than Elevation 729. Use of method (1) starting just downstream of the railroad confirmed this result.

The flow from area 3 over the railroad north of the east-west baseline drains north along a channel between the main railroad and the ERCW maintenance road and east between the ERCW maintenance road and the north cooling tower. Flow from area 3 over the railroad south of the east-west baseline drains south along a channel between the storage yard road and the switchyard past the storage yard to the river. Analysis using method (1) shows that flow over the Diesel Generator Building road controls the elevations at the Turbine and Reactor Buildings. Maximum water surface elevations were computed to be less than Elevation 729.

Flow from the switchyard and transformer yard (area 4) will drain to the east, west, and south. Maximum water surface elevations at the Turbine Building obtained using method (2) were less than Elevation 729.

Table 2.4-8 provides the weir length description and coefficient of discharge used in the analysis for areas 3 and 4.

Flow from the 150-acre drainage area north of the site drains two ways: (1) 50 acres drain east through the double 96-inch culvert under the access railroad shown on Figure 2.4-40c and (2) drainage from the remaining 100 acres is diverted to the west through an 81- by 59-inch pipe arch and, when flows exceed the pipe capacity, south over a swale in the construction access road. The flow over the construction access road drains to the west across the access highway. The following information provides details of our analysis.

The discharge hydrograph for the 100-acre area north of the plant and upstream from the construction access road was determined using a dimensionless unit graph based upon SCS procedures and PMP defined by the National Weather Service.[1] The PMP mass curve used in the determination is shown on Figure 2.4-40h. Runoff was assumed equal to rainfall. The construction access road will act as a dam with the 81- by 59-inch pipe arch acting as a low-level outlet. Flow is prevented from draining to the east above the construction access road by a dike with top elevation at 736.5 (dike location and cross-section shown on Figure 2.4-40c). The profile of the construction access road and the location of the pipe arch are shown on Figure 2.4-40c. The discharge hydrograph was routed using 2-minute time intervals through the pipe arch and over the construction access road using standard storage routing techniques. The rating curve for flow over the construction access road was developed from critical flow relationships with losses assumed equal to 0.5 $V^2/2g$.

The maximum elevation reached at the construction access road was 735.28. The pipe arch is designed for AASHTO H-20 loading which we judge is adequate for the loading expected. In the unlikely event of pipe arch failure and flow blockage, the maximum flood level at the construction access road would increase only 0.12 foot, from Elevation 735.28 to 735.4. The peak flow over the construction road was used in computations.

Flow over the construction access road discharges into the 67-acre area west of the Service, Auxiliary, Reactor, and Diesel Generator Buildings and north of the office building and gatehouse (area 2 of Figure 2.4-40a) before flowing west across the access highway (Figure 2.4-40e). Flow from 60 additional acres to the northwest of the site is also added to this area just upstream of the main access road. Elevations for area 2 were examined to include these additional flows. Backwater was computed from downstream of the access highway, crossing the perimeter road, to the Reactor, Diesel Generator, and Waste Evaporation System Buildings. The elevation at the access highway control was computed conservatively assuming that the peak flows from area 2 and over the construction road added directly. The maximum flood elevation reached in the main plant area was less than Elevation 729.

The discharge hydrograph for the 50-acre area north of the plant was conservatively assumed equivalent to the PMP hydrograph using 2 minute time intervals. This hydrograph was routed using 2-minute time intervals through the double 96-inch culvert using standard storage routing techniques.

The maximum elevation reached at the culvert was 725.67. Flow is prevented from entering the main plant area by site grading as shown on Figure 2.4-40c.

The double 96-inch culvert is designed to carry a Cooper E-80 loading as recommended by the American Railway Engineering Association (AREA). The culvert has already been exposed to the maximum loading (the generator stator with a total load of 792 tons on 22 axles) with no damage to the pipes or tracks. This maximum loading is less than the design load. Loading conditions will not be a problem.

The site will be well maintained and any debris generated from it will be minimal; therefore, debris blockage of the double 96-inch culvert or the 81- by 59-inch pipe arch will not be a problem.

Table 2.4-9 provides a description of drainage area, estimated peak discharge, and computed maximum water surface elevation for each subwatershed investigated in the site drainage analysis.

A local PMF on the holding pond does not pose a threat with respect to flooding of safety-related structures. The top of the holding pond dikes is set at Elevation 714.0, whereas water level must exceed the plant grade at Elevation 728.0 before safety-related structures can be flooded. A wide emergency spillway is cut in original ground at an elevation 2 feet below the top of the dikes. During a local PMF the water trapped by the pond rise will be considerably less than the 14-foot difference between the top of the dikes and plant grade.

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.

The PMF was determined from PMP for the watershed above the plant with consideration given to seasonal and areal variations in rainfall. Two basic storm situations were found to have the potential to produce maximum flood levels at Watts Bar Nuclear Plant. 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, 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 1,065,000 cfs for the 21,400-square-mile storm. The resulting PMF elevation at the plant would be 738.8 excluding wind wave effects.

2.4.3.1 Probable Maximum Precipitation (PMP)

Probable maximum precipitation (PMP) for the watershed above Chickamauga and Watts Bar Dams has been defined for TVA by the Hydrometeorological Report No. 41[4]. Hydrometeorological Report No. 56[35] defines PMP for watersheds above tributary dams. These reports define depth-area-duration characteristics, seasonal variations, and antecedent storm potentials and incorporate orographic effects of the Tennessee River Valley. Hydrometeorological Report No. 56[35], the most recent report covering the watershed, applies to watershed basins up to 3,000 square miles. Hydrometeorological Report No. 41[4] addresses the larger basins. 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 site.

Two basic storms with three possible isohyetal patterns and seasonal variations described in Hydrometeorological Report No. 41[4] were examined to determine which would produce maximum flood levels at the Watts Bar plant site. One storm would produce PMP depths on the 21,400-square-mile watershed above Chattanooga. Two isohyetal patterns are presented in Hydrometeorological Report No. 41[4] 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 2.4-6.

The second storm described in Hydrometeorological Report No. 41[4] would produce PMP depths on the 7,980-square-mile watershed above Chattanooga and below the five major tributary dams. The isohyetal pattern for the 7,980-square-mile storm is not 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 2.4-7.

Seasonal variations were also considered. Table 2.4-10 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.

All PMP storms are 9-day events. A 3-day antecedent storm was postulated to occur 3 days prior to the 3-day PMP storm in all PMF determinations. Rainfall depths equivalent to 40% of the main storm were used for the antecedent storms with uniform areal distribution as recommended in Hydrometeorological Report No. 41[4].

A standard time distribution pattern was adopted for 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 within the limits stipulated in Chapter VII of Hydrometeorological Report No. 41[4]. 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 2.4-8.

The PMF discharge at the Watts Bar Nuclear Plant was determined to result from the 21,400 square mile storm producing PMP on the watershed with the downstream orographically fixed storm pattern, as defined in Hydrometeorological Report No. 41[4]. The PMP storm would occur in the month of March and would produce 16.25 inches of rainfall in 3 days on the watershed above Chickamauga Dam. The storm producing the PMP would be preceded by a 3-day antecedent storm producing 6.18 inches of rainfall, which would end 3 days prior to the start of the PMP storm. Precipitation temporal distribution is determined by applying the mass curve (Figure 2.4-8) to the basin rainfall depths in Table 2.4-11.

2.4.3.2 Precipitation Losses

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 2.4-11. The average precipitation loss for the watershed above Chickamauga Dam is 2.33 inches for the 3-day antecedent storm and 1.86 inches for the 3-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 2.4-12.

2.4.3.3 Runoff and Stream Course Model

The runoff model used to determine Tennessee River flood hydrographs at Watts Bar Nuclear Plant is divided into 40 unit areas and includes the total watershed above Chickamauga Dam. Unit hydrographs are used to compute flows from the unit areas. The watershed unit areas are shown in Figure 2.4-9. 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.

Unit hydrographs were developed for each unit area for which discharge records were available from maximum flood hydrographs either recorded at stream gaging stations or estimated from reservoir headwater elevation, inflow, and discharge data using the procedures described by Newton and Vineyard, Reference [5]. 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 2.4-10 (11 Sheets). Table 2.4-13 contains essential dimension data for each unit hydrograph.

Tributary reservoir routings, except for Tellico and Melton Hill, were made using standard reservoir routing procedures and flat pool storage conditions. The main river reservoirs, Tellico, and Melton Hill reservoirs were routed using unsteady flow techniques.

Unsteady flow routings were computer-solved with the Simulated Open Channel Hydraulics (SOCH) mathematical model, based on the equations of unsteady flow, Reference [6]. 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 2.4-3 (12 Sheets).

Discharge rating curves are provided in Figure 2.4-11 (13 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 as of Amendment 98.

The unsteady flow mathematical model configuration for the Fort Loudoun-Tellico complex is shown by the schematic in Figure 2.4-12. 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 from the mouth to Douglas Dam at French Broad River mile (FBRM) 32.3 was described by 33 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, Little Tennessee River mile (LTRM) 0.3 to Chilhowee Dam at Little Tennessee River mile (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.

Fort Loudoun and Tellico unsteady flow models are joined by an interconnecting canal. The canal was modeled using 9 cross-sections with cross-section spacing of about 0.25 mile.

The Fort Loudoun-Tellico complex was verified 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 verification of the 1973 flood is shown in Figure 2.4-13 (2 Sheets). Because there were limited data to verify against on the French Broad and Holston rivers, the unsteady flow 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 verification.

(2) Using available data for the May 2003 flood for the Fort Loudoun-Tellico complex. The verification of the May 2003 flood is shown in Figure 2.4-14 (3 Sheets). The Tellico Reservoir SOCH model was also used to replicate the FEMA published 100- and 500-year profiles.

A schematic of the unsteady flow model for Watts Bar Reservoir is shown in Figure 2.4-15. The model for the 72.4 mile long reservoir was described by 39 cross-sections with two additional sections being added in the upper reach for a total of 41 sections in the SOCH 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 2.4-16 and Figure 2.4-17, respectively.

A schematic of the unsteady flow model for Chickamauga Reservoir is shown in Figure 2.4-18. The model for the 58.9 mile long reservoir was described by 29 cross-sections with one additional section being interpolated between each of the original 29 sections for a total of 57 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 2.4-19 and Figure 2.4-20, respectively.

Verifying the reservoir models with actual data approaching the magnitude of the 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[34] 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 2.4-21. 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 2.4-22. In this figure, the initial tailwater curve is compared to results from the HEC-RAS 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 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 5-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.

2.4.3.4 Probable Maximum Flood Flow

The PMF discharge at the Watts Bar Nuclear Plant was determined to be 1,065,000 cfs. This flood would result from the March 21,400-square-mile storm with a downstream orographically-fixed storm pattern (Figure 2.4-6).

The PMF discharge hydrograph is shown in Figure 2.4-23. The west saddle dike at Watts Bar Dam (Figure 2.4-24) upstream of Watts Bar Nuclear Plant 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, 1.18 miles below Watts Bar Nuclear Plant.

Chickamauga Dam, downstream, would be overtopped. The dam was postulated to remain in place, and any potential lowering of the flood levels at Watts Bar Nuclear Plant due to dam failure at Chickamauga Dam was not considered in the resulting water surface elevation.

Concrete Section Analysis

For concrete dam sections, comparisons were made between the original design headwater and tailwater levels and those that would occur in the PMF. If the overturning moments and horizontal forces were not increased by more than 20%, the structures were considered safe against failure. The 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 further and are concluded to be stable.

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 gate are less than the yield stress by a factor of 3. The stress in the trunnion pin is less than the allowable design stress by a factor greater than 10.

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 gates will not fail as a result of additional stresses which may result from possible vibrations of the gates acting as orifices.

2.4.3.5 Water Level Determinations

The controlling PMF elevation at the Watts Bar Nuclear Plant was determined to be 738.8, produced by the March 21,400 square mile storm and coincident with overtopping failure of the west saddle dike at Watts Bar Dam. The PMF elevation hydrograph is shown in Figure 2.4-25. Elevations were computed concurrently with discharges using the SOCH unsteady flow reservoir model described in Section 2.4.3.3. The PMF profile, together with the regulated maximum known flood, median summer elevation, and bottom profiles along a 4-mile reach of the Chickamauga Reservoir, which encompasses the plant location, is shown in Figure 2.4-26.

2.4.3.6 Coincident Wind Wave Activity

Some wind waves are likely when the probable maximum flood crests at Watts Bar Nuclear Plant. The flood would be near its crest for a day beginning about 2 days after cessation of the probable maximum storm (Figure 2.4-25). The day of occurrence would be in the month of March or possibly the first week in April.

Figure 2.4-27 shows the main plant general grading plan. The diesel generator buildings to the north and the pumping station to the southeast of the main building complex must be protected from flooding to assure plant safety. The diesel generator buildings operating floors are at elevation 742 which are above the maximum computed elevation including wind wave runup. The electrical equipment room of the intake pumping station will flood at elevation 728. The auxiliary and control buildings are allowed to flood. All equipment required to maintain the plant safely during the flood is either designed to operate submerged, is located above the maximum flood level, or is otherwise protected. Those safety-related facilities, systems, and equipment located in the containment structure are protected from flooding by the shield building structure with those accesses and penetrations below the maximum flood level designed and constructed as watertight elements.

The maximum effective fetches for the structures are shown on Figure 2.4-28. Effective fetch accounts for the sheltering effect of several hills on the south riverbank which become islands at maximum flood levels. The maximum effective fetch in all cases, except for the west face of the intake pumping structure occurs from the northeast or east northeast direction. The maximum effective fetch for the west face of the intake pumping structure occurs from the west direction. The diesel generator building maximum effective fetch is 1.1 miles, and the critical west face of the intake pumping structure maximum effective fetch is 1.3 miles. The maximum effective fetch for the auxiliary, control, and shield buildings is 0.8 miles.

For the Watts Bar FSAR, the two-year extreme wind for the season in which the PMF could occur was adopted to associate with the PMF crest as specified in Regulatory Guide 1.59. The storm studies on which the PMF determination is based[4] show that the season of maximum rain depth is the month of March. Wind velocity was determined from a statistical analysis of maximum March winds observed at Chattanooga, Tennessee.

Records of daily maximum average hourly winds for each direction are available at the Watts Bar site for the period May 23, 1973, through April 30, 1978. This record, however, is too short to use in a statistical analysis to determine the 2-year extreme wind, as specified in ANSI Standard N170-1976, an appendix to Regulatory Guide 1.59. Further, the necessary 30-minute wind data are not available. To determine applicability of Chattanooga winds at the Watts Bar plant, a Kolmogorov-Smirnov (K-S) statistical test was applied to cumulative frequency distributions of daily maximum hourly winds for each direction at Chattanooga and Watts Bar. The winds compared were those recorded at Chattanooga during the period 1948-74 (the period when the necessary triple-register records were available for analysis) and the Watts Bar record.

A concurrent record is not available; however, the K-S test showed that (except for the noncritical east direction) the record of daily maximum hourly velocities at Chattanooga were equal to or greater than that at Watts Bar. From this analysis it was concluded that use of the Chattanooga wind records to define seasonal maximum winds at the Watts Bar site is conservative.

The available data at Chattanooga included 30-minute and hourly winds by seasons and direction for the 27-year period 1948 through 1974.

The 30-minute wind data were analyzed for both the southwest and northeast directions. The winds from the northeast are considerably less than those from the southwest; hence, the southwest direction is controlling. Figure 2.4-29 shows the plot of the Chattanooga March maximum 30-minute winds from the critical southwest direction. The 2-year, 30-minute wind speed is 21 miles per hour determined from a mathematical fit to the Gumbel distribution. This compares with 15 miles per hour determined for the March season from the noncontrolling northeast direction.

Computation of wind waves used the procedures of the Corps Of Engineers[14]. Wind speed was adjusted based on the effective fetch length for over water conditions. For the diesel generator building, the adjusted wind speed is 23.8 miles per hour. The intake pumping structure maximum adjusted wind speed is 24.2 miles per hour for the critical west face. For the auxiliary, control, and shield buildings the adjusted wind speed is 23.4 miles per hour.

For waves approaching the diesel generator building, the maximum wave height (average height of the maximum 1 percent of waves) would be 1.7 feet high, crest to trough, and the significant wave height (average height of the maximum 33-1/3 percent of waves) would be 1.0 foot high, crest to trough. The corresponding wave period is 2.0 seconds. For the intake pumping structure, the maximum wave height would be 2.2 feet and the significant wave height would be 1.3 feet, with a corresponding wave period of 2.3 seconds. For the critical west face, the maximum wave height would be 1.9 feet high, and the significant wave height would be 1.1 feet high. The corresponding wave period is 2.1 seconds. The maximum wave height approaching the auxiliary, control, and shield buildings would be 1.5 feet high, and the significant wave height would be 0.9 foot high. The corresponding wave period is 1.9 seconds.

Computation of wind setup used the procedures of the Corps Of Engineers[14]. The maximum wind setup is 0.1 foot for all structures. Computation of runup used the procedures of the Corps Of Engineers[14]. At the diesel generator building the corresponding runup on the earth embankment with a 4:1 slope is 2.3 feet and reaches elevation 741.2, including wind setup. The runup on the critical west face wall of the intake pumping station is 2.1 feet and reaches elevation 741.0, including wind setup. The configuration of the north face of the intake pumping station, opposite of the intake channel, allows higher runup of 3.4 feet. The remaining south and east faces allow runup of 2.4 feet. However, there are no credible entry points to the structure on the north, south, or east faces. Therefore, the runup on these faces is discounted. The runup on the walls of the auxiliary, control, and shield buildings is 1.7 feet and reaches elevation 740.6, including wind setup.

Runup does not exceed the design basis flood level for any of the structures. Additionally, runup at the diesel generator building is maintained on the slopes approaching the structure and is below all access points to the building. Runup has no consequence at the shield building because all accesses and penetrations below runup are designed and constructed as watertight elements.

The static effect of wind waves was accounted for by taking the static water pressure from the maximum height of the runup. The dynamic effects of wind waves were accounted for as follows:

The dynamic effect of nonbreaking waves on the walls of safety-related structures was investigated using the Sainflou method[15]. Concrete and reinforcing stresses were found to be within allowable limits.

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[16]. The concrete and reinforcing stresses were found to be less than the allowable stresses.

The dynamic effect of broken waves on the walls of safety-related structures was investigated using the method proposed by the U.S. Army Coastal Engineering Research Center.[15] Concrete and reinforcing stresses were found to be within allowable limits.

2.4.4 Potential Dam Failures, Seismically 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. All 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 Watts Bar Nuclear Plant against failure by floods caused by the assumed failure of dams due to seismic forces. To assure that safe shutdown of the Watts Bar Nuclear Plant is not impaired by flood waters, TVA has in these studies added conservative assumptions to be able to show that the plant can be safety 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 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.

2.4.4.1 Dam Failure Permutations

There are 12 major dams above Watts Bar Nuclear Plant whose failure could influence plant site flood levels. 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; and Norris, Melton Hill, Fontana, and Tellico Dams between Fort Loudoun and Watts Bar. Dam locations with respect to the plant site are shown in Figure 2.1-1.

Analyses to determine dam integrity in seismic events were made for two basic conditions.

(1) Determination of the water level at the plant during one-half the PMF with full reservoirs if its crest were augmented by flood waves from the postulated failure of upstream dams during an operating basis earthquake (OBE).

(2) Determination of the water level at the plant during a 25-year flood with full reservoirs if its crest were augmented by flood waves from the postulated failure of upstream dams during a safe shutdown earthquake (SSE).

The OBE and SSE are defined in Sections 2.5.2.4 and 2.5.2.7 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.

Prior to the 1998 reanalysis, the flood levels from postulated seismic failure of tributary dams given in this report were higher than those in the PSAR. These higher levels resulted from:

(1) Use of unsteady flow models for the Clinch and Little Tennessee Rivers for the routing of Norris and Fontana seismic dam failure surges which replaced approximate routing procedures used in the PSAR analysis.

(2) Use of improved Watts Bar Reservoir unsteady flow models which extended up the Clinch River embayment to Melton Hill Dam.

(3) Use of a discharge rating for Norris failed dam section developed by TVA Engineering Laboratory model studies.

In the 1998 reanalysis all potentially critical seismic events involving dam failures above the plant were reevaluated. These events included the postulated OBE failure of Fontana, the postulated OBE failure of Norris, the postulated OBE failure of Cherokee and Douglas, the postulated SSE failure of Norris, Cherokee and Douglas, and the postulated SSE failure of Norris, Douglas, Fort Loudoun and Tellico.

The highest flood level at Watts Bar from different seismic dam failure and flood combinations would be elevation 727.5 from failure of Norris, Cherokee and Douglas Dams during the SSE earthquake coincident with the twenty-five year flood. Wind wave could raise the level to elevation 728.2. Runup could reach elevation 729.0 on a 3:1 slope.

Plant safety would be assured by shutdown prior to this flood crossing plant grade, elevation 728, using the warning system described in Section 2.4.14.

This is the only combination of seismic dam failures with coincident flood which could result in a flood at Watts Bar exceeding plant grade. All other combinations would produce flood levels well below plant grade.

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 the Watts Bar plant.

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.

The analyses for earthquake are based on the pseudo-static analysis method as given by Hinds[17] with increased hydrodynamic pressures determined by the method developed by Bustamante and Flores[18]. 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 the analysis.

Waves created at the free surface of the reservoir by an earthquake are considered of no importance. Based upon studies by Chopra[19] and Zienkiewicz[20] it is 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 accumulation rate is slow, as measured by TVA for many years[21].

Embankment

Embankment analysis was made using the standard slip circle method. 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 shear strength of the materials in the dam and the foundation, are the same as those used in the original stability analysis.

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

Flood Routing

The runoff model of Section 2.4.3.3 was used to reevaluate potentially critical events involving dam failures above the plant. The remaining events (the postulated OBE failure of Watts Bar, the postulated OBE failure of Fort Loudoun, the postulated SSE failure of Fontana and Douglas, the postulated SSE failure of Norris, and the postulated SSE failure of Fontana, Fort Loudoun, and Tellico) produced plant site flood levels sufficiently lower than the controlling events and were not re-evaluated.

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

OBE Concurrent With One-Half the Probable Maximum Flood

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% of all the spillway base, and about 42% of the powerhouse base, in compression. Results are given in Figure 2.4-68. Dynamic analysis of the concrete structures resulted in the determination that the base acceleration is amplified at levels above the base. This differs from the previous analysis where amplification was not considered.

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 2.4-69.

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 which 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.

A potentially severe condition is the OBE at the onset of the main portion of one-half the probable maximum flood flow into Watts Bar Reservoir when most spillway gates would be closed during bridge failure, as described above. The gate hoisting machinery would be inoperable from being struck by the bridge with the result that 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.

For the condition described above with the most probable embankment breaching from overflow, the outflow of Watts Bar Dam would increase rapidly from about 200,000 cfs prior to the breach to about 660,000 cfs when breaching is complete. Breach time would be about 5 hours.

The 660,000 cfs breach flow is the crest. The flood level at Watts Bar Dam reached elevation 717.5. Elevation at the plant site will be somewhat less, which is safely below plant grade elevation 728. This flood level was not reevaluated using the model described in Section 2.4.3.3, as amended, nor as part of the 1998 reanalysis, because it is clearly not controlling.

For flow conditions between the 25-year flood and one-half the probable maximum flood, when the bottom of the gates are in the water, failure of the bridge during an OBE with consequent striking of the gates by the downstream bridge girders will result in failure of the gate lifting chains. The gates will rotate to the closed position. This condition is less severe than that described above for gates remaining closed during one-half the probable maximum flood; consequently, the resulting flood levels were not determined.

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 2.4-71.

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 2.4-72.

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, it was found that the Watts Bar inflows are much less than the condition resulting from simultaneous failure of Cherokee and Douglas, as described later.

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 2.4-73. The result of the stability analysis of the earth embankment is shown in Figure 2.4-74 and indicates a factor of safety of 1.28.

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 2.4-75. 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.

Figure 2.4-76 shows the likely condition of the dam after failure. Based on stability analyses the non-overflow blocks remaining in place are judged to withstand the OBE. 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 the top of the earth embankment of the main dam; however, the West Saddle Dike would be overtopped and breached. A complete washout of the dike was assumed. The resulting water level at the nuclear plant site is 721.5, 6.5 feet below 728 plant grade.

Cherokee Dam

Results of the Cherokee Dam stability analysis for a typical spillway block are shown in Figure 2.4-77. Based on this analysis the spillway is judged stable at the foundation base elevation 900. Analyses made for other elevations above elevation 900, but not shown in Figure 2.4-77, indicate the resultant of forces falls outside the base at elevation 1010. The spillway is assumed to fail at this elevation.

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

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

Results of the analysis for the highest portion of the south embankment are shown on Figure 2.4-78. 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 OBE. Because the north embankment and saddle dams 1, 2, and 3 are generally about one-half, or less, as high as the south embankment, they are judged to be stable for the OBE.

Figure 2.4-79 shows the assumed condition of the dam after failure. All debris from 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 and Douglas, discussed under multiple failures, is more critical.

Douglas Dam

Results of the Douglas Dam stability analysis for a typical spillway block are shown in Figure 2.4-80. The upper part of the Douglas spillway is approximately 12 feet 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, 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. 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 OBE without failure.

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

Results of the analysis of the saddle dam shown on Figure 2.4-81 indicate a factor of safety of 1. Therefore, the saddle dam is considered to be stable for the OBE.

Figure 2.4-82 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.

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

Fontana Dam

Fontana Dam was assumed to fail in the OBE although no stability analysis was made. Fontana is a high dam constructed with three longitudinal contraction joints in the higher 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 OBE without failure.

Figure 2.4-83 shows the part of Fontana Dam judged to remain in its original position after failure and the assumed location on 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 to 27 containing either two or three longitudinal joints are assumed to fail. Right abutment blocks 1 to 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 1 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 would fail along with Fontana in the OBE. Instant vanishment was assumed. Tellico and Watts Bar Dam spillway gates would be operable during and after the OBE. Failure of the bridge at Fort Loudoun Dam would render the spillway gates inoperable in the wide-open position.

The Fontana failure wave would overtop and fail 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 761.3, 5.7 feet below the top of the embankment. No embankment failure would occur. However, the West Saddle Dike would be overtopped and breached. The elevation at the plant site would be 725.2, 2.8 feet below 728 plant grade.

Multiple Failures

Attenuation studies of the OBE show that above Watts Bar Dam only the simultaneous failures of Cherokee and Douglas Dams need be considered with respect to Watts Bar Nuclear Plant safety. These two dams are only 15 miles apart, and an OBE located midway between them is assumed to cause their simultaneous failure. The degree of failure and likely position of debris are judged to be comparable to that shown for single failure of these dams in Figure 2.4-79 and 2.4-82.

The postulated simultaneous failures of Cherokee and Douglas Dams would reach a maximum headwater elevation of 833.8 feet at Fort Loudoun Dam, 0.55 foot above the top of the embankment. Fort Loudoun would be overtopped for only about six hours to a maximum depth of 0.55 foot. Breaching analysis indicates that this short overtopping time and shallow overflow depth would not fail the dam. Although transfer of water into Tellico would occur, the maximum headwater would only reach Elevation 826, which is four feet below top of dam. At Watts Bar Dam the headwater would reach Elevation 758.2, 8.8 feet below the top of the earth embankment of the main dam. However, the West Saddle Dike would be overtopped and breached. A complete washout of the dike was assumed. The elevation at the plant site would be 723.1, 4.9 feet below plant grade Elevation 728.

SSE Concurrent With 25-Year Flood

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 Watts Bar plant 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 nuclear plant site would be elevation 723, 5 feet below plant grade. This flood level was not reevaluated using the runoff model described in Section 2.4.3.3 as amended because it is clearly not controlling.

Fort Loudoun Dam

Results of the stability analysis for Fort Loudoun Dam are shown on Figure 2.4-86. 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 2.4-87. 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 2.4-88 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 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 2.4-89 shows after failure conditions with all debris assumed located in the channel below the failure elevation.

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

Norris Dam

Under SSE conditions blocks 31 to 45 (833 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 OBE but with the same top level, elevation 970. Figure 2.4-90 shows the part of the dam judged to fail and the location and height of the resulting debris.

This postulated single failure would result in peak headwater at Watts Bar of 747.9, 9.1 feet 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 OBE with the one-half PMF.

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.

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 OBE. None of these single failures need to be carried downstream, however, because elevations would be lower than the same failures in one-half the probable maximum flood.

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 TVA has determined this as 0.14 g). In addition, the SSE must be located in a very precise region to have the potential for multiple dam failures. In order to fail Norris, Cherokee, and Douglas 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.

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 two combinations of dams are postulated to fail as well as the combination of (1) Fontana, Fort Loudoun, and Tellico; and (2) Fontana and Douglas.

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. Using the failure modes shown on Figures 2.4-83, 2.4-88, and 2.4-89 for Fontana, Fort Loudoun, and Tellico respectively, unsteady routing showed the failure wave overtopping Watts Bar Dam with resulting embankment failure. Initial Watts Bar embankment failure begins at headwater level 763.0. Headwater levels will continue to rise to elevation 764.7 because of no spillway discharge. This event would result in a flood level at the nuclear plant site of 720.7, 7.3 feet below 728 plant grade. This flood level was not reevaluated using the update runoff model described in Section 2.4.3.3 as amended, nor as part of the 1998 reanalysis, because it is clearly not controlling.

Norris, Cherokee, and Douglas Dams were also postulated to fail simultaneously. Figure 2.4-91 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 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. At least this most conservative assumption was used. 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 2.4-76, 2.4-79, and 2.4-82 for single dam failure.

The flood for the postulated failure combination would overtop and breach Fort Loudoun Dam. Although transfer of water into Tellico would occur, the maximum headwater would only reach Elevation 820, which is 10 feet below top of dam. At Watts Bar Dam the headwater would reach Elevation 764.9, 2.1 feet below the top of the earth embankment of the main dam. However, the West Saddle Dike would be overtopped and breached. The elevation at the plant site would be 727.5, 0.5 feet below plant grade Elevation 728. This is the highest flood resulting from any combination of seismic and flood events.

The flood elevation hydrograph at the plant site is shown on Figure 2.4-111.

Norris, Douglas, Fort Loudoun, and Tellico Dams were postulated to fail simultaneously. Figure 2.4-93 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 2.4-76, 2.4-82, 2.4-88 and 2.4-89 for single dam failure. For this re-evaluation, Fort Loudoun and Tellico were assumed to fail completely as the portions judged to remain are relatively small. This is conservative.

This postulated failure combination results in Watts Bar headwater Elevation 758.9, 8.1 feet below the top of the earth embankment of the main dam. However, the West Saddle Dike would be overtopped and breached. A complete washout of the dike was assumed. The elevation at the plant site would be 722.8, 5.2 feet below plant grade Elevation 728.

Douglas and Fontana were postulated to fail simultaneously. Figure 2.4-94 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, the portions judged to remain and the debris arrangements are as given in Figures 2.4-82 and 2.4-83 for single dam failure. 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. The Fontana failure flood wave would overtop and breach Tellico Dam and its saddle dikes. The flood from the Douglas failure would reach Fort Loudoun after Tellico has been overtopped and breached. Although Fort Loudoun gates are inoperable in the closed position, the Fort Loudoun-Tellico canal would provide enough relief to keep Fort Loudoun Dam from being overtopped. The combined Douglas-Fontana failure surge would reach elevation 751.7 at Watts Bar Dam, 5.3 feet below dam top. Resulting water surface at the Watts Bar plant would reach elevation 721.2, 6.8 feet below plant grade. This flood level was not reevaluated using the model described in Section 2.4.3.3 as amended, nor as part of the 1998 reanalysis because it is not controlling.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

Unsteady flow routing techniques[23] were used to evaluate plant site flood levels from postulated seismically induced dam failures wherever their inherent accuracy was needed. In addition to the flow models described in Section 2.4.3.3 the unsteady flow models described below were used as adjuncts to route floods from postulated dam failures.

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.

2.4.4.3 Water Level at Plantsite

The unsteady flow analyses of different postulated combinations of seismic dam failures coincident with floods described in Section 2.4.4.1 yields a maximum elevation of 727.5, excluding wind wave effects. As shown in Table 2.4-14, it would result from the SSE failure of Norris, Cherokee, and Douglas Dams coincident with the twenty-five year flood postulated to occur in June when reservoir levels are high. A June wind with 50% exceedance probability over the 1.3-mile effective fetch is 12 miles per hour overland. Flood waves, crest to trough, are about 1.0 foot high resulting in maximum water elevation of 728.2. Runup could reach elevation 729.0 on a 3:1 earth slope. The static and dynamic effects of wind waves on structures are described in Section 2.4.3.6.

2.4.5 Probable Maximum Surge and Seiche Flooding

Chickamauga Lake level during non-flood conditions would not exceed elevation 682.5, normal maximum pool level, for any significant time. No conceivable meteorological conditions could produce a seiche nor reservoir operations a surge which would reach plant grade elevation 728, some 45 feet above normal maximum pool levels.

2.4.6 Probable Maximum Tsunami Flooding

Because of its inland location the Watts Bar plant is not endangered by tsunami flooding.

2.4.7 Ice Effects

Because of its location in a temperate climate significant amounts of ice do not form on lakes and rivers in the plant vicinity and ice jams are not a source of major flooding.

The present potential for generator of significant surface ice at the site is less today than prior to closure of Chickamauga and Watts Bar Lakes in 1940 and 1942, respectively. This condition exists because of (1) daily water level fluctuations from operating Chickamauga Reservoir downstream and Watts Bar Reservoir upstream would break up surface icing before significant thickness could be formed, (2) flows are warmed by releases from near the bottom of Watts Bar Reservoir, and (3) increased water depths due to Chickamauga Reservoir result in a greater mass needing to be cooled by radiation compared to pre-reservoir conditions.

After closure of Watts Bar in January 1942, there have been no extended periods of cold weather and no serious icing conditions in the Watts Bar Nuclear Plant site region. On several occasions, ice has formed near the shore and across protected inlets but has not constituted a problem on the main reservoirs.

The lowest water temperature observed in Watts Bar Lake at the dam during the periods 1942-1953, and June 1967 to November 1973 for which records were kept, was 39 degrees on January 30, 1970, the coldest January since 1940 in the eastern part of the Basin. This lake temperature is indicative of the lowest water temperature released from Watts Bar Lake during winter months.

The most severe period of cold weather recorded in the Valley was January and early February 1940 prior to present lake conditions at the plantsite. A maximum ice depth of five inches was recorded on the Tennessee River at Chattanooga. There were no ice jams except one small one on the lower French Broad River.

Records of icing are limited and none are available at the site prior to 1942. From newspaper records, the earliest known freeze in the vicinity was at Knoxville in 1796. More recently, newspaper accounts and U.S. Weather Bureau records for Knoxville provide a fairly complete ice history from 1840 to 1940. At Knoxville the Tennessee River was frozen over 16 times, and floating ice was observed six other times.

The most severe event in this period prior to 1940 was in December-January 1917-18 when ice jammed the Tennessee River at Knoxville for 1 to 2 weeks, reaching 10 feet high at some places. In late January rain and temperature rise produced flooding on the Clinch River referred to by local people as the "ice tide." There is no record of ice jamming, however.

There are no safety-related facilities at the Watts Bar site which could be affected by an ice jam flood, wind-drive ice ridges, or ice-produced forces other than a flooding of the plant itself. An ice jam sufficient to cause plant flooding is inconceivable. There are no valley restrictions in the 1.9-mile reach below Watts Bar Dam to initiate a jam, and an ice dam would need to reach at least 68 feet above streambed to endanger the plant.

Intake pump suctions which will be used for the intake of river water will be located a minimum of 7.6 feet below minimum reservoir water level; hence, no thin surface ice which may form will effect the pipe intake. In the assumed event of complete failure of Chickamauga Dam downstream, the minimum release from Watts Bar Dam will ensure a 5.9 foot depth of water in the intake channel.

2.4.8 Cooling Water Canals and Reservoirs

The intake channel, as shown in Figure 2.1-5, extends approximately 800 feet from the edge of the reservoir through the flood plain to the intake pumping station. The channel, as shown in Figure 2.4-99, has an average depth of 36 feet and is 50 feet wide at the bottom. The side slopes are 4 on 1 and are designed for sudden drawdown, due to assumed loss of downstream dam, coincident with a safe shutdown earthquake.

In response to multipurpose operations, the level of Chickamauga Reservoir fluctuates between a normal minimum of 675.0 feet and a normal maximum of 682.5 feet. The minimum average elevation of the reservoir bottom at the intake channel is 656 feet and the elevation of the intake channel bottom is 660 feet. The 15 foot normal minimum depth of water provided in the intake channel is more than ample to guarantee flow requirements. The intake provides cooling water makeup to the closed-cycle cooling system and the essential raw cooling water systems. The maximum flow requirement for the plant for all purposes is 178 cfs based on four ERCW pumps and six RCW pumps inservice.

The protection of the intake channel slopes from wind-wave activity is afforded by the placement of riprap, shown in Figures 2.4-99 in accordance with TVA design standards, from elevation 660 to elevation 690. The riprap is designed for waves resulting from a wind velocity of 50 mph.

2.4.9 Channel Diversions

Channel diversion is not a potential problem for the plant. Currently, no channel diversions upstream of the Watts Bar plant would cause diverting or rerouting of the source of plant cooling water, and none are anticipated in the future. The floodplain is such that large floods do not produce major channel meanders or cutoffs. The topography is such that only an unimaginable catastrophic event could result in flow diversion above the plant.

2.4.10 Flooding Protection Requirements

Assurance that safety-related facilities are capable of surviving all possible flood conditions is provided by the discussions given in Sections 2.4.14, 3.4, 3.8.1, 3.8.2 and 3.8.4

The plant is designed to shut down and remain in a safe shutdown condition for any rainfall flood exceeding plant grade, up to the "design basis flood" discussed in Section 2.4.3 and for lower, seismic-caused floods discussed in Section 2.4.4. Any rainfall flood exceeding plant grade will be predicted at least 28 hours in advance by TVA's Water Resources organization.

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

See Section 2.4.14 for a detailed presentation of the flood protection plan.

2.4.11 Low Water Considerations

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

2.4.11.1 Low Flow in Rivers and Streams

The probable minimum water level at the Watts Bar plant is elevation 673 and would occur in the winter flood season as a result of special Chickamauga Reservoir preflood drawdown, at which time flows would be substantial. The most severe drought in the history of the Tennessee Valley region occurred in 1925. Frequency studies for the 1874-1935 period prior to regulation show that there is less than one percent change that the 1925 observed minimum 1-day flow of 3300 cfs downstream at Chattanooga might occur in a given year. At the plantsite the corresponding minimum 1-day flow is estimated to be 2700 cfs compared to 2600 cfs in the PSAR.

Although dependable flow under extreme drought conditions is sufficient to meet all plant requirements, there is the added assurance of large quantities of water in TVA's multiple purpose tributary reservoirs upstream. Stored water at prescribed minimum pool levels in these reservoirs (Tellico Reservoir excluded) could provide more than 1,000 cfs at the Watts Bar site for 2 years with no rainfall. These minimum levels will not be violated without specific TVA Board of Directors' action in which the safety of Watts Bar would be a controlling consideration. This guarantees that adequate water would be available if needed at the Watts Bar site.

In the assumed event of complete failure of Chickamauga Dam and with the headwater before failure assumed to be the normal summer level, elevation 682.5, the water surface at the site will begin to drop 4 hours after failure of the dam and will fall at a fairly uniform rate to elevation 666 in approximately 22 hours from failure. This time period is more than ample for initiating the release of water from Watts Bar Dam.

The estimated minimum flow requirement for the ERCW system is 50 cfs; however, in order to guarantee both ample depth and supply of water, a minimum flow of 2,000 cfs will be released from Watts Bar Dam. With flow of 2,000 cfs water surface elevation would be 665.9 producing 5.9-foot depth in the intake channel.

2.4.11.2 Low Water Resulting From Surges, Seiches, or Tsunami

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

2.4.11.3 Historical Low Water

From the beginning of stream gage records at Chattanooga in 1874 until the closure of Chickamauga Dam in January 1940, the estimated minimum daily flow at Watts Bar Nuclear Plant site was 2700 cfs on September 7 and 13, 1925. The next lowest estimated flow of 3900 cfs occurred in 1881 and also in 1883.

Since January 1942 low flows at the site have been regulated by TVA reservoirs, particularly by Watts Bar and Chickamauga Dams. Under normal operating conditions, there may be periods of several hours daily when there are no releases from either or both dams, but average daily flows at the site have been less than 5,000 cfs only 0.9% of the time and have been less than 10,000 cfs only 4.8% of the time.

On March 30 and 31, 1968, during special operations for the control of water milfoil, there were no releases from either Watts Bar or Chickamauga Dams during the 2-day period. Daily average releases of zero have been recorded on four other occasions during the past 25 years.

Since January 1940, water levels at the plant have been controlled by Chickamauga Reservoir. Since then the minimum level at the dam was 673.3 on January 21, 1942.

2.4.11.4 Future Control

Future added controls which could alter low flow conditions at the plant are not anticipated because no sites that would have a significant influence remain to be developed.

2.4.11.5 Plant Requirements

The engineering safety feature water supply system requiring river water is the essential raw cooling water (ERCW). Also, the high pressure fire pumps perform an essential safety function during flood conditions by providing a feedwater supply to steam generators, makeup to the spent fuel pool, and auxiliary boration makeup tank. For interface of the fire protection system with the auxiliary feedwater system, see Section 10.4.9. The ERCW pumps are located on the intake pumping station deck at elevation 741.0 and the ERCW pump intake is at elevation 653.33 feet. The ERCW intake will require 5 feet of submergence. Based on a minimum river surface elevation of 665.9 feet, a minimum of 12.07 feet of pump suction submergence will be provided.

In the assumed event of complete failure of Chickamauga Dam and with the headwater before failure assumed to be the normal summer level, elevation 682.5, the water surface at the site will begin to drop 4 hours after failure of the dam and will fall at a fairly uniform rate to elevation 666 in approximately 22 hours from failure. This time period is more than ample for initiating the release of water from Watts Bar Dam.

The estimated minimum flow requirement for the ERCW system is 50 cfs. However, in order to guarantee both ample depth and supply of water, a minimum flow of 2,000 cfs will be released from Watts Bar Dam.

This flow will give a river surface elevation of 665.9, which ensures a 5.9-foot depth of water in the intake channel and approximately 10 feet in the river. The river surface elevation is controlled by the weir effect of Hunter Shoals, elevation 661.2, located approximately 7.5 miles downstream from the site. The stage discharge rating curve at the entrance to the intake channel is shown by Figure 2.4-95. Cross sections of Hunter Shoals are shown in Figure 2.4-96. Figure 2.4-97 shows the channel profile of the Tennessee River for the reach from mile 520.0 to 521.37.

A flow of at least 2,000 cfs can be released at the upstream dam, Watts Bar, through the spillway gates, the turbines or the lock. The spillway gates offer the largest flow of water. There are twenty 40-foot-wide radial gates operated by two traveling gate hoists on the deck and one of the hoists is always located over a gate. At minimum headwater elevation 735.0, one gate opened 2 feet will provide a flow of 2,000 cfs; fully open, 15,000 cfs will be provided.

There are five turbines, each with a maximum flow of 9,400 cfs and an estimated speed/non-load flow of 900 to 1100 cfs. The lock culvert emptying and filling valves are electrically operated segmental type with a bypass switch located in each of the four valve control stations. These can be used at any time to open or close both filling and emptying valves.

In the improbable event of loss of station service power at the dam, a 300-kVA gasoline-engine-driven generator located in the powerhouse will supply emergency power. The generator feeds into the main board when used and the emergency power is adequate to operate each of the three sources of water supply discussed.

For concurrent loss of the upstream and downstream dams, assurance that sufficient flow will be available is provided by records of the minimum natural flow at the plantsite before construction of dams on the Tennessee River. This flow is estimated to be 2,700 cfs. Since this flow exceeds the 2,000 cfs specified above to be released through Watts Bar Dam, it is not necessary to reserve a storage volume in Watts Bar Reservoir.

2.4.12 Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents

2.4.12.1 Radioactive Liquid Wastes

A discussion of the routine handling and release of liquid radioactive wastes is found in Section 11.2, "Liquid Waste Systems." The routine and nonroutine nonradiologial liquid discharges are addressed in the Watts Bar Nuclear Plant's NPDES permit (Permit No. TN0020168) and the Spill, Prevention, Control, and Countermeasure Plan (SPCC plan), respectively. The nonradiological liquid discharges are under the regulatory jurisdiction of the State of Tennessee.

2.4.12.2 Accidental Slug Releases to Surface Water

An accidental release of radioactive or nonradioactive liquid from the plant site would be subject to naturally induced mixing in the Tennessee River. The worst case for a given volume, V_o (cubic feet), of liquid is a release which takes place over a short period of time. Calculations have been made to determine the reduction in concentration of such a release as it progresses downstream; particular emphasis has been placed on the concentrations at the surface water intakes downstream of the plant. The model used here is based on the convective diffusion equation as applied to the dispersion in natural streams[24,25]. The major assumptions used in this analysis are:

(1) The release is assumed to occur at the right bank with no diffuser induced mixing whether the release occurs at the bank or through the diffuser.

(2) The effluent becomes well mixed vertically (but not horizontally) relatively rapidly (well before reaching first downstream water intake). This assumption is usually justified in riverine situations[26,27].

(3) The river flow is uniform and one-dimensional over a rectangular cross-section.

Other less restrictive assumptions are described in Reference [27].

Under assumption 2, the two-dimensional form of the convective diffusion equation is sufficient and may be written as

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = E_x\frac{\partial^2 c}{ux^2} + E_y\frac{\partial^2 c}{\partial y^2} \qquad (1)$$

in which C is the concentration of radioactive effluent in the river; u is cross-sectionally averaged river velocity; x and y are coordinates in the downstream and lateral directions, respectively; and E_x and E_y are the dispersion coefficients in the x and y directions. Following Reference [25], it is assumed that the formal dependence of E_x and E_y on river parameters is

$$E_x = a_x U^* H \qquad (2a)$$

and

$$E_y = a_y U^* H \qquad (2b)$$

in which a_x and a_y are empirical coefficients, U^* is the river shear velocity, and H is the river depth. Relationships between U^* and bulk river parameters may be found in any open channel hydraulics text.[28]

Equation (1) was solved for the slug release by applying the method of images[27,29] to the instantaneous infinite flow field solution of equation (1) which is given in Reference [29]

$$\frac{C}{C_0} = \frac{V_0}{4\pi Ht\ E_x E_y} \exp^{-\left[\frac{(x-ut)^2}{4E_x t} + \frac{(y-y_0)^2}{4E_y t}\right]} \qquad (3)$$

in which C_0 is the initial concentration of radioactive material in the liquid effluent, t is the time elapsed since the release of the slug and y is the distance of the release from the right bank. Equation (3) was used in the method of images solutions.

2.4.12.2.1 Calculations

The above model was applied to predict the maximum concentrations which would be observed on the right bank of the Tennessee River at two downstream locations; the right bank concentrations will always be higher than those on the left bank. The release is assumed to occur on the right bank at Tennessee River Mile (TRM) 528; the river width is assumed constant at 1,100 feet and the river depth is assumed constant at 30 feet. The Watts Bar Dam discharge equaled or exceeded 50% of the time is 28,200 cfs.

The coefficients a_x and a_y in Equation (2) were chosen to be 100 and 0.6, respectively; these values are based on the results in Reference [25]. The shear velocity, U^* was computed assuming a Manning's n of 0.030 to describe the bed roughness of the river. Because the actual release volume, V_0, is not known *a priori*, results are presented in terms of a relative concentration, defined as $C/(C_0,V_0)$. Thus, to obtain the concentration reduction factor C/C_0, this relative concentration must be multiplied by the release volume V_0 (in cubic feet).

Calculations show that the concentrations along the right bank at the downstream water intakes will be as follows:

Water Intake	Tennessee River Mile	Relative Concentration (l/cu. ft.)
Dayton	503.8	2.8×10^{-9}
East Side Utility (formerly Volunteer Army Ammunition Plant)	473.0	1.3×10^{-9}

2.4.12.3 Effects on Ground Water

The plant site is underlain by terrace deposits of gravel, sand, and clay, having an average thickness of 40 feet. The deposit is variable in grain-size composition from place to place. Locally, very permeable gravel is present. Essentially all of the ground water under the site is in this deposit.

Bedrock of the Conasauga Shale underlies the terrace deposit. Foundation exploration drilling and foundation excavation revealed that very little water occurs in the bedrock.

The average saturated thickness of the terrace deposit is about 25 feet. Discharge from this material is mostly small springs and seeps to drainways along the margin of the site. Directions of ground water flow are discussed in Section 2.4.13.

The nearest point of probable ground water discharge is along a small tributary to Yellow Creek, which at its nearest point is 2,600 feet from the center of the plant. In this direction, the hydraulic gradient (dh/dl) is 26 feet (maximum) in 2,600 feet, or 0.01. The hydraulic conductivity (K) of the terrace materials is estimated to be 48 feet/day. (The basis for this estimate is described in Section 2.4.13.3.) Porosity (0) is estimated to be 0.15.

Average ground water velocity = (K dh/dl)/O = 3.2 ft/day or 812 days average travel time through the terrace deposit to the nearest point of ground water discharge.

Estimating the density of the water-bearing material to be 2.0 and the distribution coefficient for strontium to be 20, the computed average travel time for strontium indicates a period of over 200 times longer than that for water, or 1.8×10^5 days (almost 500 years) travel time from the plant site to the nearest point of ground water discharge. This time of travel would be further increased by accounting for the delay resulting from movement through and absorption by unsaturated materials above the water table.

Water available for dilution, based on-the estimated porosity of 0.15 and a saturated thickness of 25 feet, is estimated to be 3.75 cubic feet per square foot of surface area. In a 1000-foot wide strip extending from the plant site to the nearest point of ground water discharge, the volume of stored water would be 9.8×10^6 cubic feet.

There are no data on which to base a computation of dispersion in the ground water system. For a conservative analysis, it would be necessary to assume that no dispersion occurs.

2.4.13 Groundwater

2.4.13.1 Description and On-Site Use

Only the Knox Dolomite is regionally significant as an aquifer. This formation is the principal source of base flow to streams of the region. Large springs, such as Ward Spring 2.7 miles west of the site, are fairly common, especially at or near the contact between the Knox Dolomite and the overlying Chickamauga Limestone. Water occurs in the Knox Dolomite in solution openings formed along bedding planes and joints and in the moderately thick to thick cherty clay overburden. The formation underlies a 1- to 2-mile wide belt 2.5 miles west of the site at its nearest point; a narrow slice, the tip of which is about one mile north of the site; and a 1- to 2-mile wide belt, one mile east of the site and across Chickamauga Lake.

Within a two-mile radius of the site, there is no use of the Knox Dolomite as a source of water to wells for other than small supplies.

Other formations within the site region, described in detail in Section 2.5.1.1, include the Rome Formation, a poor water-bearing formation; the Conasauga Shale, a poor water-bearing formation; and the Chickamauga Limestone, a poor to moderate water-bearing formation that normally yields no more than 25 gallons per minute (gpm) to wells.

The plant site is underlain by the Conasauga Shale, which is made up of about 84% shale and 16% limestone and occurs as thin discontinuous beds (Section 2.5.1.2). Surficial materials are older terrace deposits and recent alluvial deposits, fine-grained, poorly sorted, and poorly waterbearing.

The pattern of groundwater movement shown on Figure 2.4-105 indicates that recharge of the shallow water-bearing formations occurs from infiltration of local precipitation and from lateral underflow from the area north of the plant site. All ground-water discharge from the site is to Chickamauga Lake, either directly or via Yellow Creek.

Potable water for plant use is obtained from the Watts Bar Utility District. Their water is obtained from 3 wells located 2.5 miles northwest of the plant.

2.4.13.2 Sources

Ground water sources within a two-mile radius of the site are listed in Table 2.4-15 and their locations are shown on Figure 2.4-102. Of the 89 wells listed, only 58 are equipped with pumps. Two of the thirteen spring sources listed are equipped with pumps. Seventy-nine residences are supplied by ground water, with one well supplying five houses. Assuming three persons per residence and a per capita use rate of 75 gpd, total ground-water use is less than 10,000 gpd.

Drawdown data are available only for the Watts Bar Reservation wells, as listed in the previous section.

Water-level fluctuations have been observed monthly in six observation wells since January 1973. Data collection for wells 7, 8, & 9 began in December 1981. The locations of these wells are shown on Figure 2.4-104. Data for the period January 1973 through December 1975 is shown on Figure 2.4-103.

As elsewhere in the region, water levels normally reach maximum elevations in February or March and are at minimum elevations in late summer and early fall. Depth to the water table is generally less than 20 feet throughout the plant site.

Figure 2.4-105 is a water-table contour map of the area within a two-mile radius of the plant site, based on 48 water-level measurements made in January 1972. The water table conforms fairly closely to surface topography, so that directions of ground-water movement are generally the same as those of surface-water movement. The water-table gradient between plant site and Chickamauga Lake at maximum water-table elevation and minimum river stage is about 44 feet in 3200 feet, or 0.014.

Water occurs in the Consauga Shale in very small openings along fractures and bedding planes. Examination of records of 5500 feet of foundation exploration drilling showed only one cavity, 0.6-foot thick, penetrated.

Water occurs in the terrace deposit material in pore spaces between particles. The deposit is composed mostly of poorly-sorted clay- to gravel-sized particles and is poorly water bearing, although an approximately six-foot-thick permeable gravel zone is locally present at the base of the terrace deposit. The foundation excavation required only intermittent dewatering after initial drainage. The excavation was taken below the base of the terrace deposit into fresh shale. No weathered shale was found to be present; the contact between the terrace deposit and fresh shale is sharp.

The average depth to the water table in the plant area, based on data collected during August through December 1970, is 17 feet; the average overburden thickness is 40 feet; the saturated overburden thickness is therefore, 24 feet. No weathered zones or cavities were penetrated in the Conasauga Shale below a depth of 85 feet, so that the average saturated thickness of bedrock is assumed to be less than 50 feet.

The plant site is hydraulically isolated by Yellow Creek and Chickamauga lake to the west, south, and east; it is hydraulically isolated to the north by the relatively impermeable Rome Formation underlying the site. Therefore, it is believed that any off-site groundwater withdrawals could not result in altered groundwater movement at the site.

No attempt was made to measure hydraulic properties of overburden or of bedrock at this site because of the very limited occurrence of ground water and the heterogeneity and anisotropy of the materials underlying the site.

2.4.13.3 Accident Effects

Assuming a maximum annual range in saturated thickness of overburden of between 23 feet and 33 feet, and a porosity of 0.15, total water stored in this material, and the maximum volume available for dilution, ranges seasonally between 4.6 and 6.6 cubic feet per square foot of surface area. Water available for dilution in bedrock is very small and may be less than 0.01 cubic foot per square foot of surface area.

Since dispersion and exchange characteristics are not known, it must be assumed that these are not factors in a release of liquid radioactive material which would then travel to discharge points at the same rate as water movement. There are no direct pathways to ground-water users since all groundwater discharge from the site is to adjacent surface-water bodies.

Groundwater travel time has been estimated for water in the terrace deposit, in which essentially all ground water at the site occurs.

The nearest point of possible groundwater discharge is 2600 feet west of the plant site, along a tributary to Yellow Creek. In this direction the maximum hydraulic gradient is 26 feet in 2600 feet, or 0.01. The maximum hydraulic conductivity of the terrace materials is estimated to be 48 ft/day, based on particle-size analyses of terrace-deposit materials as related to permeability[30].

$$v = \frac{K\ dh/dl}{O}$$

$$\text{where } v = \text{mean velocity, ft/day;}$$

$$K = \text{hydraulic conductivity} = 48 \text{ ft/day;}$$

$$dh/dl = \text{hydraulicgradient} = .01$$

$$O = \text{porosity} = 0.15 \text{ (extimated average effective)}$$

$$V = 48\frac{(.01)}{(.15)} = 3.2 \text{ ft/day}$$

or 812 days travel time from plant to nearest point of groundwater discharge.

Packer tests on the Conasauga Shale in foundation holes, using water at 50 psi, showed no acceptance, although one 0.6 foot cavity was penetrated in one hole in a total of more than 5,000 feet of drilling. Therefore, no estimate of time of water travel was made for water in bedrock.

2.4.13.4 Monitoring and Safeguard Requirements

The potential for the plant to affect groundwater users is very low because of its physical location, however, any provisions for radiological groundwater monitoring will be as described in the Watts Bar Monitoring Plan. A network of observation wells will be maintained as needed and ground water will be analyzed for radioactivity as required by the Technical Specifications.

In the event of accidental release of radioactivity to the groundwater system, nearby groundwater users will be advised not to use their wells for drinking water until an investigation can be made of the extent, rate, and direction of movement of the contaminant.

Monitoring and notification for both the routine and any accidental nonradioactive liquid discharges to either surface or groundwaters would be implemented as required by the facilities NPDES permit (Permit No. TN0020168) and the Spill, Prevention, Control, and Countermeasure Plan (SPCC plan), respectively. These requirements for the nonradiological liquid discharges are under the regulatory jurisdiction of the State of Tennessee.

2.4.13.5 Design Basis for Subsurface Hydrostatic Loading

The ground water levels used for structural design are discussed in Section 2.5.4.6.

Dewatering of the construction excavation is discussed in Section 2.5.4.6.

2.4.14 Flooding Protection Requirements

Assurance that safety-related facilities are capable of surviving all possible flood conditions is provided by the discussions given in Section 2.4.2.2, Section 3.4, Sections 3.8.1 and 3.8.4 and this section, 2.4.14.

2.4.14.1 Introduction

This subsection describes the methods by which the Watts Bar Nuclear Plant is capable of tolerating floods above plant grade without jeopardizing public safety. Since flooding of this magnitude, as illustrated in Section 2.4.2 is most unlikely, extreme steps are considered acceptable, including actions that create or allow extensive economic consequence to the plant. The actions described herein will be implemented for floods ranging from slightly below plant grade, to allow for wave runup to the design basis flood. The plant Flood Protection Plan (Technical Requirement 3.7.2) specifies the flood warning conditions and subsequent actions.

2.4.14.1.1 Design Basis Flood

The design basis flood (DBF) is the calculated upper-limit flood that includes the probable maximum flood (PMF) plus the wave runup as discussed in Section 2.4.3.6. The table below gives representative levels of the DBF at different plant locations.

Design Basis Flood (DBF) Levels

Probable Maximum Flood (still reservoir)	738.8
DBF Runup on 4:1 sloped surfaces	741.2
DBF Runup on the critical vertical wall of the intake pumping structure	741.0
DBF Runup on vertical walls of the auxiliary, control, and shield buildings	740.6

In addition to flood level considerations, plant flood preparations cope with the "fastest rising" flood which is the calculated flood, including seismically induced floods, that can exceed plant grade with the shortest warning time. Reservoir levels for large rainfall floods in the Tennessee Valley can be predicted well in advance. By dividing the pre-flood preparation steps into two stages, a minimum of a 27 hour, pre-flood transition interval is available between the time a flood warning is received and the time the flood waters exceed plant grade. The first stage, a minimum of 10 hours long, commences upon receipt of a flood warning. The second stage, a minimum of 17 hours long, is based on a confirmed estimate that conditions will produce a flood. This two-stage scheme is designed to prevent excessive economic loss in case a potential flood does not fully develop. Refer to Section 2.4.14.4.

2.4.14.1.2 Combinations of Events

Because floods above plant grade, earthquakes, tornadoes, or design basis accidents, including a LOCA, are individually very unlikely, a combination of a flood plus any of these events, or the occurrence of one of these during the flood recovery time, or of the flood during the recovery time after one of these events, is considered incredible. However, as an exception, certain reduced levels of floods are considered together with a seismic event. Refer to Sections 2.4.14.10 and 2.4.4.

2.4.14.1.3 Post Flood Period

Because of the improbability of a flood above plant grade, no detailed procedures are established for return of the plant to normal operation unless and until a flood actually occurs. If flood mode operation (Section 2.4.14.2) should ever become necessary, it is possible to maintain this mode of operation for a sufficient period of time (100 days) so that appropriate recovery steps can be formulated and taken. The actual flood waters are expected to recede below plant grade within 1 to 4 days.

2.4.14.1.4 Localized Floods

Localized plant site flooding due to the probable maximum storm (Section 2.4.2.3) will not enter vital structures or endanger the plant. Any offsite power loss resulting from water ponding on the switchyard or water entry into the Turbine Building will be similar to a loss of offsite power situation as described in Chapter 15. The other steps described in this subsection are not applicable to this case. Refer to Section 2.4.2.3.

2.4.14.2 Plant Operation During Floods Above Grade

"Flood mode" operation is defined as the set of conditions described below by means of which the plant is safely maintained during the time when flood waters exceed plant grade (elevation 728.0) and during the subsequent period until recovery (Section 2.4.14.7) is accomplished.

2.4.14.2.1 Flooding of Structures

The Reactor Building will be maintained dry during the flood mode. Walls and penetrations are designed to withstand all static and dynamic forces imposed by the DBF; minor seepage through the concrete walls and any seepage through the leading penetrations into the annulus will be allowed to flow to the Reactor Building floor and equipment drain sump by removing the blind flange on penetration X-118. The Reactor Building floor and equipment drain sumps are more than capable of pumping this flow.

The Diesel Generator Buildings also will remain dry during the flood mode since its lowest floor is at elevation 742.0. Other structures, including the Service, Turbine, Auxiliary, and Control Buildings, would be allowed to flood as the water exceeds their grade level entrances. Equipment that is located in these structures and required for operation in the flood mode is either above the DBF or suitable for submerged operation.

2.4.14.2.2 Fuel Cooling

Spent Fuel Pool

Fuel in the spent fuel pool is cooled by the Spent Fuel Pool Cooling and Cleanup System (SFPCCS), the active components of which are located above flood waters. During the flood mode of operation, heat is removed from the heat exchangers by essential raw cooling water instead of component cooling water. The SFPCCS cooling circuit is assured of two operable SFPCCS pumps (a third pump is available as a backup) as well as two SFPCCS heat exchangers. High spent fuel pool temperature causes an annunciation in the Main Control Room indicating equipment malfunction. Additionally, that portion of the cooling system above flood water is inspected approximately every 8 hours to confirm continued proper operation. As a backup to spent fuel cooling, water from the High Pressure Fire Protection (HPFP) system can be added to the spent fuel pool.

Reactors

Residual core heat is be removed from the fuel in the reactors by natural circulation in the reactor coolant system. Heat removal from the steam generators is accomplished by adding river water from the HPFP system (Section 9.5.1) and relieving steam to the atmosphere through the power operated relief valves. This transition from auxiliary feedwater to river water is accomplished during Stage II of the flood preparation procedures. Refer to Section 2.4.14.4.1. Reactor coolant system pressure is maintained at less than 350 psig by operation of the pressurizer relief valves and heaters. Secondary side pressure is maintained below 125 psig by operation of the power operated relief valves. At times beyond approximately 10 hours following shutdown of the plant two relief valves have sufficient capacity to remove the steam generated by decay heat. Since 10 hours is less than the minimum flood warning time available, the plant can be safely shut down and decay heat removed by operation of two power operated relief valves per unit.

The earliest that the HPFP pumps would be utilized to supply auxiliary feedwater would be about 20 hours after reactor shutdown. At this time, in order to remove the decay heat from both reactor units, the water requirement to the steam generators would be approximately 300 gpm. Later times following reactor shutdown would have gradually decreasing HPFP system makeup water flow rate requirements. With the steam generator secondary side pressure less than 125 psig, a single HPFP pump can supply makeup water well in excess of the requirement of 300 gpm. Additional surplus flow is available since there are four HPFP pumps, two powered from each emergency power train. The HPFP pump head-capacity curve is illustrated in Chapter 9.

The main steam power operated relief valves are adjusted by controls in the auxiliary control room as required to maintain the steam pressure within the desired pressure range. The controls in the main control room also can be utilized to operate the valves in an open-closed manner. Also, a manual loading station and the relief valve handwheel provide additional backup control for each relief valve.

The power operated relief valves would be used to depressurize the steam generators as discussed above to maintain steam generator pressure sufficiently below the developed head of the fire pumps. Note that even in the event of a total loss of makeup water flow at the time of maximum decay heat load, approximately 6 hours are available to restore makeup water flow before the steam generators would boil dry.

If one or both reactors are open to the containment atmosphere during the refueling operations, then the decay heat of the fuel in the open unit(s) and spent fuel pool heat is removed in the following manner. The refueling cavity is filled with borated water (approximately 2,000 ppm boron concentration) from the refueling water storage tank. The SFPCCS pump takes suction from the spent fuel pool and discharges to the SFPCCS heat exchangers. The SFPCCS heat exchanger output flow is directed by a temporary piping connection to the Residual Heat Removal (RHR) system upstream to the RHR heat exchangers. This piping (spool piece) connection is prefabricated and is installed only during preparation for flood mode operation. (The tie-in locations in the SFPCCS and RHRS are shown in Figures 2.4-106 and 2.4-107 respectively.) After passing through the RHR heat exchangers, the water enters the reactor vessel through the normal cold leg RHR injection paths, flows downward through the annulus, upward through the core (thus cooling the fuel), then exits the vessel directly into the refueling cavity. This results in a water level differential between the spent fuel pool and the refueling cavity with sufficient water head to assure the required return flow through the twenty-inch diameter fuel transfer tube thereby completing the path to the spent fuel pool.

Any leakage from the reactor coolant system will be collected to the extent possible in the reactor coolant drain tank; nonrecoverable leakage is made up from supplies of clean water stored in the four cold leg accumulators, the pressurizer relief tank, and the demineralized water tank. Even if these sources are unavailable, the fire protection system can be connected to the auxiliary charging system (Section 9.3.6) as a backup. Whatever the source, makeup water is filtered, demineralized, tested, and borated, as necessary, to the normal refueling concentration, and pumped by the auxiliary charging system into the reactor (see Figures 2.4-108 and 2.4-109).

2.4.14.2.3 Cooling of Plant Loads

Plant cooling requirements with the exception of the fire protection system which must supply makeup water to the steam generators, are met by the Essential Raw Cooling Water (ERCW) System. The intake pumping station is designed to retain full functional capability of the ERCW system and HPFP system water intakes for all floods up to and including the DBF. The ERCW system and HPFP system water intakes also remain fully functional in the remote possibility of a flood induced failure of Chickamauga Dam. (Refer to Sections 9.2.1 and 9.5.1.)

2.4.14.3 Warning Scheme

See Section 2.4.14.8 (Warning Plan).

2.4.14.4 Preparation for Flood Mode

An abnormal operating instruction is available to support operation of Unit 1.

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

2.4.14.4.1 Reactor Initially Operating at Power

If both reactors are operating at power, Stage I and then, if necessary, Stage II procedures are initiated. Stage I procedures consist of a controlled reactor shutdown and other easily revokable steps, such as moving flood mode supplies above the maximum possible flood elevation and making load adjustments on the onsite power supply. After scram, the reactor coolant system is cooled by the auxiliary feedwater (Section 10.4.9) and the pressure is reduced to less than 350 psig. Stage II procedures are the least easily revokable and more damaging steps necessary to have the plant in the flood mode when the flood exceeds plant grade. HPFP system water (Section 9.5.1) will replace auxiliary feedwater for steam generator makeup water. Other essential plant cooling loads are transferred from the component cooling water system to the ERCW system and the ERCW replaces raw cooling water to the ice condensers (Section 9.2.1). The radioactive waste (Chapter 11) system will be secured by filling tanks below DBF level with enough water to prevent flotation. One exception is the waste gas decay tanks, which are sealed and anchored against flotation. Power and communication cables below the DBF level that are not required for submerged operation are disconnected, and batteries beneath the DBF level are disconnected.

2.4.14.4.2 Reactor Initially Refueling

If time permits, fuel is removed from the unit(s) undergoing refueling and placed in the spent fuel pool; otherwise fuel cooling is accomplished as described in Section 2.4.14.2.2. If the refueling canal is not already flooded, the mode of cooling described in Section 2.4.14.2.2 requires that the canal be flooded with borated water from the refueling water storage tank. If the flood warning occurs after the reactor vessel head has been removed or at a time when it could be removed before the flood exceeds plant grade, the flood mode reactor cooling water flows directly from the vessel into the refueling cavity.

Flood mode operation requires that the prefabricated piping be installed to connect the RHR and SFPC systems, that the proper flow to the spent fuel pit diffuser and the RHRS be established and that essential raw cooling water be directed to the secondary side of the RHRS and SFPCCS heat exchangers. The connection of the RHR and SFPC systems is made using prefabricated in-position piping which is normally disconnected. During flood mode preparations, the piping is connected using prefabricated spool pieces.

2.4.14.4.3 Plant Preparation Time

The steps needed to prepare the plant for flood mode operation can be accomplished within 24 hours of notification that a flood above plant grade is expected. An additional 3 hours are available for contingency margin. Site grading and building design prevent any flooding before the end of the 27 hour preflood period.

2.4.14.5 Equipment

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

2.4.14.5.1 Equipment Qualification

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

2.4.14.5.2 Temporary Modification and Setup

Normal plant systems used in flood mode operation and in preparation for flood mode operation may require modification from their normal plant operating configuration. Such modification, since it is for a limiting design condition and since extensive economic consequences are acceptable, is permitted to allow operation of systems outside of their normal plant configuration. However, most alterations will be only temporary and inconsequential in nature. For example, the switchover of plant cooling loads from the component cooling water to essential raw cooling water system is done through valves and prefabricated spool pieces, causing little system disturbance or damage.

2.4.14.5.3 Electric Power

Because there is a possibility that high winds could destroy power lines and disconnect the plant from offsite power at any time during the preflood transition period, the preparation procedure and flood mode operation are accomplished assuming only onsite power circuits available. While most equipment requiring ac electric power is a part of the permanent emergency onsite power distribution system other components, if required, could be temporarily connected, when the time comes, by prefabricated jumper cables.

The loads that are normally supplied by onsite power but are not required for the flood are disconnected early in the preflood period. Those loads used only during the preflood period are disconnected from the onsite power system during flood mode operations. DC electric power is similarly disconnected from unused loads and potentially flooded cables.

Charging is maintained for each battery by the onsite ac power system as long as it is required. Batteries that are beneath the DBF level are disconnected during the preflood period when they are no longer needed.

2.4.14.5.4 Instrument, Control, Communication and Ventilation Systems

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

Instrumentation is provided to monitor vital plant parameters such as the reactor coolant temperature and pressure and steam generator pressure and level. Important plant functions are either monitored and controlled from the main control area, or, in some cases where time margins permit, from other points in the plant that are in close communication with the main control area.

Communications are provided between the central control area (the Main and Auxiliary Control Rooms) and other vital areas that might require operator attention, such as the Diesel Generator Building.

Ventilation, when necessary, and limited heating or air conditioning is maintained for locations throughout the plant where operators might be required to go or where required by equipment heat loads.

2.4.14.6 Supplies

The equipment and most supplies required for the flood are on hand in the plant at all times. Some supplies may require replenishment before the end of the period in which the plant is in the flood mode. In such cases supplies on hand are sufficient to last through the short time (Section 2.4.14.1.3) that flood waters will be above plant grade and until replenishment can be supplied.

2.4.14.7 Plant Recovery

The plant is designed to continue safely in the flood mode for 100 days even though the water is not expected to remain above plant grade for more than 1 to 4 days. After recession of the flood, damage will be assessed and detailed recovery plans developed. Arrangements will then be made for reestablishment of off-site power and removal of spent fuel. A decision based on economics would be made on whether or not to regain the plant for power production. In either case, detailed plans would be formulated after the flood, when damage can be accurately assessed. The 100-day period provides a more than adequate time for the development of procedures for any maintenance, inspection, or installation of replacements for the recovery of the plant or for a continuation of flood mode operations in excess of 100 days.

2.4.14.8 Warning Plan

Plant grade elevation 728.0 can be exceeded by rainfall floods and closely approached by seismic-caused dam failure floods. A warning plan is needed to assure plant safety from these floods.

The warning plan is divided into two stages: Stage I, a minimum of 10 hours long and Stage II, a minimum of 17 hours so that unnecessary economic consequences can be avoided, while adequate time is allowed for preparing for operation in the flood mode.

Stage I allows preparation steps causing minimal economic consequences to be sustained but will postpone major economic damage until the Stage II warning predicts a likely forthcoming flood above grade.

2.4.14.8.1 Rainfall Floods

Protection of the Watts Bar Plant from rainfall floods that might exceed plant grade utilizes a flood warning issued by TVA's Water Management. TVA's climatic monitoring and flood predicting systems and flood control facilities permit early identification of potentially critical flood producing conditions and reliable prediction of floods which may exceed plant grade well in advance of the event.

The Watts Bar Nuclear Plant flood warning plan provides a minimum of 27 hours to prepare for operation in the flood mode, 3 hours more than the 24 hours needed. Four additional preceding hours would be available to gather and analyze rainfall data and produce the warning. The first stage, Stage I, of shutdown begins when there is sufficient rainfall on the ground in the upstream watershed to yield a projected plantsite water level of elevation 714.5 in the winter months (October 1 through April 15) and elevation 726.5 in the summer (April 16 through September 30). This assures that additional rain will not produce water levels to elevation 727.0 in less than 27 hours from the time shutdown is initiated. The water level of elevation 727.0 (one foot below plant grade) allows margin so that waves due to winds cannot disrupt the flood mode preparation.

The plant preparation status is held at Stage I until either Stage II begins or TVA's Water Management determines that floodwaters will not exceed elevation 727.0 at the plant. The Stage II warning is issued only when enough additional rain has fallen to predict that elevation 727.0 (winter or summer) is likely to be reached.

2.4.14.8.2 Seismically-Induced Dam Failure Floods

Only one postulated combination of seismically induced dam failures and coincident storm conditions was shown to result in a flood which could exceed Elevation 727 at the plant. Watts Bar plant protection from this flood utilizes TVA’s Water Management forecast system to identify when a critical combination exists. Stage I shutdown is initiated upon notification that a critical dam failure combination has occurred or loss of communication prevents determining a critical case has not occurred. Stage I shutdown continues until it has been determined positively that critical combinations do not exist. If communications do not document this certainty, shutdown procedures continue into Stage II activity. Stage II shutdown continues to completion or until lack of critical combinations is verified.

2.4.14.9 Basis For Flood Protection Plan In Rainfall Floods

2.4.14.9.1 Overview

Large Tennessee River floods can exceed plant grade elevation 728.0 at Watts Bar Nuclear Plant. Plant safety in such an event requires shutdown procedures which may take 24 hours to implement. TVA flood forecast procedures are used to provide at least 27 hours of warning before river levels reach elevation 727.0. Use of elevation

727.0, 1 foot below plant grade, provides enough margin to prevent wind generated waves from endangering plant safety during the final hours of shutdown activity. Forecast will be based upon rainfall already reported to be on the ground.

To be certain of 27 hours for preflood preparation, flood warnings with the prospect of reaching elevation 727.0 must be issued early when lower target elevations are forecast. Consequently, some of the warnings may later prove to have been unnecessary. For this reason preflood preparations are divided into two stages. Stage I steps requiring 10 hours are easily revokable and cause minimum economic consequences. The estimated probability is less than 0.0032 that a Stage I warning will be issued during the 40-year life of the plant.

Added rain and stream-flow information obtained during Stage I activity will determine if the more serious steps of Stage II need to be taken with the assurance that at least 17 hours will be available before elevation 727.0 is reached. The probability of a DBF occurring during the 40-year life of the plant is very small.

Flood forecasting, to assure adequate warning time for safe plant shutdown during floods, will be conducted by TVA's Water Management.

2.4.14.9.2 TVA Forecast System

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

Elements of the present (1998) forecast system above Watts Bar Nuclear Plant include the following:

(1) Ninety-eight (98) rain gages, and measure rainfall, with an average density of 165 square miles per rain gage. Of these, 54 are GOES Data Collection Platform (DCP) Satellite telemetered gages, and 27 are Data Logger telephone telemetered gages which depend upon commercial telephone lines, and 17 are observer gages located at TVA hydro and fossil plants and non-TVA hydro plants. In the case of commercial telephone line failure, field personnel can be notified by radio to interrogate and provide data from the 27 Data Logger gages.

The telephone gages are interrogated on a two hour interval on the even hour (Central prevailing time) to obtain hourly rainfall readings. During flood periods, the gages can be interrogated more frequently if desired. The satellite gages transmit hourly rainfall data every 3 hours during normal operations. In addition, the satellite gages event report when 0.1 inch or greater rainfall accumulates. The normal and event transmissions are conveyed on separate satellite channels. Personnel at the TVA installations record six-hour rainfall data. Information from these sites is available for the normal forecast run at 6 a.m. Central time.

(2) Streamflow data are available from 23 stream gages in the system. Of these, 12 are GOES Data Collection Platform satellite telemetered gages, and 11 are Data Logger telephone telemetered gages which depend upon commercial telephone lines.

The telephone gages are interrogated on a two hour interval on the even hour (Central prevailing time) to obtain 15-minute stage readings. During flood periods, the gages can be interrogated more frequently if desired. The satellite gages transmit 15-minute stage data every 3 hours during normal operations. Random stream gage transmission is currently being tested. Information from these sites is available for the normal forecast run at 6 a.m. Central time.

(3) Hourly headwater elevation, tailwater elevation, and discharge data are received from 14 TVA and 4 non-TVA hydro plants. More frequent data can be obtained during flood operations.

(4) Weather forecasts including quantitative precipitation forecasts received at least twice daily and at other times when changes are expected.

(5) Computer programs which translate rainfall into streamflow based on current runoff conditions and which permit a forecast of flows and elevations based upon both observed and predicted rainfall. A network of UNIX workstation computers are utilized and are designed to provide backup for each other. One computer is used primarily for data collection, with the others used for executing forecasting programs for reservoir operations. The time interval between receiving input data and producing a forecast is less than 4 hours. Forecasts normally cover at least a 3-day period.

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

TVA's normal operation produces daily forecasts by 12 noon made from data collected at 6 a.m. Central time. When serious flood situations demand, personnel of Reservoir Operations work around the clock with forecasts as frequent as at 6-hour intervals.

2.4.14.9.3 Basic Analysis

The forecast procedure to assure safe shutdown of Watts Bar Nuclear Plant for flooding is based upon an analysis of 17 hypothetical PMP storms, including their antecedent storms. They enveloped potentially critical areal and seasonal variations and time distributions of rainfall. To be certain that fastest rising flood conditions were included, the effects of varied time distribution of rainfall were tested by alternatively placing the maximum daily PMP on the first, the middle, and the last day of the 3-day main storm. In each day the maximum 6-hour depth was placed during the second interval except when the maximum daily rain was placed on the last day. Then the maximum 6-hour amount was placed in the last 6 hours. The warning system is based on those PMP storm situations which resulted in the shortest time interval between watershed rainfall and elevation 727, thus assuring that this elevation could be predicted at least 27 hours in advance.

The procedures used to compute flood flows and elevations for those flood conditions which establish controlling elements of the forecast system are described in Section 2.4.3 as amended. The analyses of the remaining floods which identified the critical flood conditions were made using earlier versions of the procedures described in Section 2.4.3.

2.4.14.9.4 Hydrologic Basis for Warning System

A minimum of 27 hours has been allowed for preparation of the plant for operation in the flood mode. An additional 4 hours for communication and forecasting computations is provided to translate rain on the ground to river elevations at the plant. Hence, the warning plan provides 31 hours from arrival of rain on the ground until elevation 727 could be reached. The 27 hours allowed for shutdown at the plant consists of a minimum of 10 hours of Stage I preparation and an additional 17 hours for Stage II preparation that is not concurrent with the Stage I activity.

Although river elevation 727, 1 foot below plant grade to allow for wind waves, is the controlling elevation for determining the need for plant shutdown, lower forecast target levels are used in some situations to assure that the 27 hours pre-flood transition interval will always be available. The target river levels differ with season.

During the October 1 through April 15 "winter" season, Stage I shutdown procedures will be started as soon as target river elevation 714.5 has been forecast. Stage II shutdown will be initiated and carried to completion if and when target river elevation 727 has been forecast. Corresponding target river elevations for the April 16 through September 30 "summer" season are elevation 726.5 and elevation 727.

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

Resumption of plant operation following just Stage I shutdown activities will be allowable only after flood levels and weather conditions have returned to a condition in which 27 hours of warning will again be available.

2.4.14.9.5 Hydrologic Basis for Target States

Figure 2.4-110, in four parts, shows how target forecast flood elevations at the Watts Bar plant have been determined to assure adequate warning times. The floods shown are the fastest rising probable maximum floods at the site. Only the principal storm in which the PMP occurs has been shown. These have been preceded 3 days earlier by a 3-day storm having 40% of the main storm rainfall.

Figure 2.4-110 (A,B,C) shows the winter PMP which would produce the fastest rising flood which would cross plant grade and variations caused by changed time distribution. The fastest rising flood occurs during a PMP when the 6-hour increments increase throughout the storm with the maximum 6 hours occurring in the last period. Figure 2.4-110 (A) shows the essential elements of this storm which provides the basis for the warning plan. In this flood 8.7 inches of rain would have fallen 31 hours (27 + 4) prior to the flood crossing elevation 727 and would produce elevation 714.5 at the plant. Hence, any time rain on the ground results in a predicted plant elevation of 714.5 a Stage I shutdown warning will be issued. Examination of Figure 2.4-110 (B and C) show that following this procedure in these noncritical floods would result in a lapsed time of 47 and 49 hours between when 8.7 inches had fallen and the flood would exceed elevation 727.

An additional 3.8 inches of rain must fall promptly for a total of 12.5 inches of rain to cause the flood to exceed elevation 727. In the fastest rising flood, Figure 2.4-110 (A), this rain would have fallen in the next 8 hours. A Stage II warning would be issued within the next 4 hours. Thus, the Stage II warning would be issued 8 hours after issuance of a Stage I warning and 19 hours before the flood would exceed elevation 727.0. In the slower rising floods, Figure 2.4-110 (B and C), the time between issuance of a Stage I warning and when the 12.5 inches of rain required to put the flood to elevation 727.0 would have occurred, is 18 and 12 hours respectively. This would result in issuance of a Stage II warning not less than 4 hours later or 25 or 33 hours before the flood would reach elevation 727.0.

The summer flood, shown by Figure 2.4-110 (D), with the maximum 1-day rain on the last day provides controlling conditions when reservoirs are at summer levels. At a time 31 hours (27 + 4) before the flood reaches elevation 727.0, 11.6 inches of rain would have fallen. This 11.6 inches of rain under these runoff conditions would produce elevation 726.5, so this level becomes the Stage I target. An additional 0.2 inch of rain must fall promptly for a total of 11.8 inches of rain to cause the flood to exceed elevation 727.0. In this fastest rising summer flood, Figure 2.4-110 (D), this rain would have fallen in the next hour. A Stage II warning would be issued within the next 4 hours. Thus, the Stage II warning would be issued one hour after issuance of a Stage I warning and 26 hours before the flood would exceed elevation 727.0.

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

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

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

	Forecasts Elevations at Watts Bar	
Season	for Stage I shutdown	for Stage II shutdown
Winter (October 1 - April 15)	714.5	727
Summer (April 16 - September 30)	726.5	727

2.4.14.9.6 Communications Reliability

Communication between projects in the TVA power system is via (a) TVA-owned microwave network, (b) Fiber-Optics Systems, and (c) by commercial telephone. In emergencies, additional communication links are provided by Transmission Power Supply radio networks. The four networks provide a high level of dependability against emergencies.

The hydrologic network for the watershed above Watts Bar that would be available in flood emergencies if commercial telephone communications are lost includes 61 rainfall gages (17 at power installations and 54 satellite gages). The Reservoir Operations is linked to the TVA power system by all five communication networks. The data from the satellite gages are received via a data collection platform-satellite computer system located in the Reservoir Operations office. These are distributed over the watershed such that reasonable flood forecasting can be done from this data while the balance of data is being secured from the remaining hydrologic network stations.

The preferred, complete coverage of the watershed employs numerous rainfall and stream-flow locations above the Watts Bar plant (See Section 2.4.14.9.2). Involved in the communications link to these locations are routine radio, radio satellite and the commercial telephone system networks. In an emergency, available radio communications would be called upon to assist.

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

2.4.14.10 Basis for Flood Protection Plan in Seismic-Caused Dam Failures

Floods resulting from combined seismic and flood events can closely approach plant grade, thus requiring emergency measures. The 1998 reanalysis showed that only one seismic dam failure combination coincident with a flood, i.e., the SSE failure of Norris, Cherokee and Douglas concurrent with the 25-year flood, would result in a flood approaching plant grade. As shown in Table 2.4-14 all other candidate combinations of events would create flood levels well below plant grade Elevation 728. Dam failure during non-flood periods would not present a problem at the plant as resulting flood levels for all candidate combinations would be well below plant grade. The reanalysis showed that failure of the controlling combination in a non-flood period and at summer flood guide levels would produce Elevation 725.2 at the plant, 2.8 feet below plant grade. All other combinations in non-flood periods would produce elevations much lower.The time from seismic occurrence to arrival of failure surge at the plant in the critical event is about 50 hours as shown in Figure 2.4-111 and is adequate to permit safe plant shutdown in readiness for flooding.

The warning scheme for safe plant shutdown is based on the fact that a combination of critically centered large earthquake and rain produced flood conditions must coincide before the floodwave from seismically caused dam failures will approach plant grade. In flood situations, an extreme earthquake must be precisely located to fail Norris, Cherokee and Douglas dams before a flood threat to the site would exist.

The warning system utilizes TVA's flood forecast system to identify when flood conditions will be such that seismic failure of critical dams could cause a floodwave to approach elevation 728 at the plant site. These conditions combined with any concern by TVA Water Management that failure of a single upstream dam has occurred or is imminent will lead to an early warning. A Stage I warning is declared once failure of Norris, Cherokee and Douglas Dams has been confirmed.

If loss of or damage to an upsteam dam is suspected, efforts will be made by Hydro Operations to determine whether dam failure has occurred. If the critical case has occurred or it cannot be determined that it has not occurred, Stage I shutdown will be initiated in time to assure the 27 hour flood preparation period. Once initiated, the flood preparation procedures will be carried to completion unless it is determined that the critical case has not occurred.

Communications between the plant, dams, power system control center, and TVA Water Management at Knoxville are accomplished by microwave networks, fiber options networks, radio networks, and commercial telephone service. These systems are described in FSAR Section 9.5.2.3.

2.4.14.11 Special Condition Allowance

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

REFERENCES

(1) National Weather Service, "Probable Maximum and TVA Precipitation for Tennessee River Basins up to 3,000 Square Miles in Area and Durations to 72 Hours," Hydrometeorological Report No. 45, 1969, with Addendum of June 1973.

(2) U.S. Army Corps of Engineers, "Standard Project Flood Determination,: Civil Works Engineer Bulletin 52-8, March 1952.

(3) SCS National Engineering Handbook, Section 4, Hydrology, July 1969.

(4) U.S. Weather Bureau, "Probable Maximum and TVA Precipitation Over The Tennessee River Basin Above Chattanooga," Hydrometeorological Report No. 41, 1965.

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

(6) Garrison, J. M., Granju, J. P., and Price, J. T., "Unsteady Flow Simulation in Rivers and Reservoirs," Journal of the Hydraulics Division, ASCE, Volume 95, No. HY5, Proceedings Paper 6771, September 1969, pages 1559-1576.

(7) Eagleson, Peter S., "A Distributed Linear Model for Peak Catchment Discharge," Proceedings, The International Hydrology Symposium, September 1967, Fort Collins, Colorado, Volume 1.

(8) Kulandaiswamy, V. C., "A Nonlinear Approach to Runoff Studies," Proceedings, The International Hydrology Symposium, September 1967, Fort Collins, Colorado, Volume 1.

(9) Ardis, C. V., Jr., "A Nonlinear Channel Routing Model, "Proceedings, Theme 1, May 1971 Flow Symposium, Instrument Society of America, Pittsburgh, 1974.

(10) Reference deleted by Amendment 63.

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

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

(13) National Climatic Center, Asheville, North Carolina, "Extreme Wind Study for Selected Stations in the Tennessee Valley," prepared under Contract No. TV-36522A, August 1975.

(14) U.S. Army Corps of Engineers, "Computation of Freeboard Allowances for Waves in Reservoirs," Engineering Technical Letter No. 1110-2-8, August 1966.

(15) U.S. Army coastal Engineering Research Center, "Shore Protection Planning and Design," Third Edition, 1966.

(16) Anderson, Paul, "Substructure Analysis and Design," 1948.

(17) Hinds, Julian, Cregar, William P., and Justin, Joel D., "Engineering For Dams," Volume 11, Concrete Dams, John Wiley and Sons, Incorporated, 1945.

(18) Bustamante, Jurge I., Flores, Arando, "Water Pressure in Dams Subject to Earthquakes," Journal of the Engineering Mechanics Division, ASCE Proceedings, October 1966.

(19) Chopra, Anil K., "Hydrodynamic Pressures on Dams During Earthquakes," Journal of the Engineering Mechanics Division ASCE Proceedings, December 1967, pages 205-223.

(20) Zienkiewicz, O. C., "Hydrodynamic Pressures Due to Earthquakes," Water Pressures Due to Earthquakes," Water Power, Volume 16, September 1964, pages 382-388.

(21) Tennessee Valley Authority, "Sedimentation in TVA Reservoirs," TVA Report No. 0-6693, Division of Water Control Planning, February 1968.

(22) Reference deleted by Amendment 63.

(23) Price, J. T. and Garrison, J. M., Flood Waves From Hydrologic and Seismic Dam Failures," paper presented at the 1973 ASCE National Water Resources Engineering Meeting, Washington, D. C.

(24) Fisher, H. B., "Longitudinal Dispersion in Laboratory and Natural Systems" Keck Laboratory Report KH-R-12, California Institute of Technology, Pasadena, California, June 1966.

(25) Fisher, H. B., "The Mechanics of Dispersion in Natural Streams," Journal of the Hydraulics Division, ASCE Vol. 93, No HY6, November 1967.

(26) Yotsukura, N., "A Two-Dimensional Temperature Model for the Thermally Loaded River with Steady Discharge" Proceedings of the Eleventh Annual Environmental and Water Resources Engineering Conference, Vanderbilt University, Nashville, Tennessee, 1972.

(27) Almquist, C. W., "A Simple Model for the Calculation of Transverse Mixing in Rivers with Application to the Watts Bar Nuclear Plant," TVA, Division of Water Management, Water Systems Development Branch, Technical Report No. 9-2012, March 1977.

(28) Henderson, E. M., Open Channel Flow, MacMillen, 1966.

(29) Carlslaw, B. S. and J. C. Jaeger, Conduction of Heat in Solids, Oxford University Press, London England, 1959.

(30) Johnson, A. E., 1963, Application of Laboratory Permeability.

(31) Stoker, J. J., "Water Waves," Interscience Publishers, Inc., New York, 1966, Up. 333-341.

(32) Bretschneider, C. L., "Wave Refraction, Diffraction and Reflection Chapter F of Estuary and Coastline Hydrodynamics. MIT Hydrodynamic Lab, Cambridge, Massachusetts.

(33) Keulegan, G. H., "Wave Damping Effects of Fibrous Screens," Research Report H-72-2. Corps of Engineers, Vicksburg, Mississippi, 1972.

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

(35) National Weather Service, "Probable Maximum and TVA Precipitation Estimates with Areal Distribution for Tennessee River Drainages Less Than 3,000 Square Miles in Area," Hydrometeorological Report No. 56, October 1986.

(36) U.S. Geological Survey, National Water Information System: Web Interface, USGS Surface-Water Data for the Nation, Website, http://waterdata.usgs.gov/usa/nwis/ws, accessed April 2006.

Table 2.4-1 Location of Surface Water Supplies in the 58.9 Mile Reach of the Mainstream of the Tennessee River Between Watts Bar Dam (Trm 529.9) and Chichamagua Dam (Trm 471.0)

Plant Name	Use (MGD)	Location	(Bank)	Approximate Distance From Site (River Miles)	Type Supply
Watts Bar Dam	#	TRM 529.9		1.9 (Upstream)	Industrial
Watts Bar Steam Plant	##	TRM 529.9	R	1.9 (Upstream)	Industrial
Watts Bar Nuclear Plant	###	TRM 528	R	0	Industrial
City of Dayton	1.78	TRM 503.8	R	24.2 (Downstream)	Municipal
Sequoyah Nuclear Plant	1615.68	TRM 483.6	R	44.4 (Downstream)	Industrial
East Side Utility	5.00	TRM 473	L	55.0 (Downstream)	Municipal
Chickamagua Dam	#	TRM 471		57.0 (Downstream)	Industrial

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

Not active at this time. If plant is reactivated, new numbers will be needed.

Not in operation at this time. When operational maximum intake will be - 115 million gallons per day.

Table 2.4-2 Facts About TVA Dams and Reservoirs
(Page 1 of 2)

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

Table 2.4-2 Facts About TVA Dams and Reservoirs
(Page 2 of 2)

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

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

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

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

e) At June 1 flood guide elevation.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Watts Bar - 3 feet: embankment at elevation 767 raised to elevation 770.
Fort Loudoun - 3.75 feet: embankment at elevation 830 was raised 7 feet to elevation 837 (3.75 feet above top of concrete wall at elevation 833.25).
Tellico - 4 feet: embankment at elevation 830 raised to elevation 834.
Cherokee - 3 feet: embankment at elevation 1089 raised to elevation 1092.

Table 2.4-3 TVA Dams - River Mile Distances to WBNP
(Page 1 of 2)

River	Structure/River Mouth	River Mile[a]	Distance from WBNP (mi.)
Tennessee River			
	Chickamauga Dam	471	57
	Hiwassee River	499.5	28.5
	WBNP	528	-
	Watts Bar Dam	530	2
	Clinch River	568	40
	Little Tennessee River	601	73
	Fort Loudoun Dam	602	74
	Holston River	652	124
	French Broad River	652	124
Hiwassee River		0	28.5
	Ocoee River	34.5	63
	Apalachia Dam	66	94.5
	Hiwassee Dam	76	104.5
	Nottely River	92	120.5
	Chatuge Dam	121	149.5
Ocoee River		0	63
	Ocoee #1 Dam	12	75
	Ocoee #2 Dam	24	87
	Ocoee #3 Dam	29	92
	Toccoa River	38(b)	101
Toccoa River		0	101
	Blue Ridge Dam	15(b)	116
Nottely River		0	120.5
	Nottely Dam	21	141.5
Clinch River		0	40
	Melton Hill Dam	23	63
	Norris Dam	80	120
Little Tennessee River		0	73
	Tellico Dam	0.5	73.5

Table 2.4-3 TVA Dams - River Mile Distances to WBNP
(Page 2 of 2)

River	Structure/River Mouth	River Mile[(a)]	Distance from WBNP (mi.)
	Chilhowee Dam	33.5	106.5
	Calderwood Dam	43.5	116.5
	Cheoah Dam	51.5	124.5
	Fontana Dam	61	134
Holston River		0	124
	Cherokee Dam	52	176
French Broad River		0	124
	Douglas Dam	32	156

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

Table 2.4-4 Facts about TVA Dams Above Chickamauga

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

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

Table 2.4-5 Facts About Non-TVA Dams and Reservoirs

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

(1) Volume at top of gates.

I

Table 2.4-6 Flood Detention Capacity - TVA Projects Above Watts Bar Nuclear Plant

Project	FloodStorage January 1 (ac.-ft.)	Flood Storage March 15 (ac.-ft.)	Flood Storage Summer (ac.-ft.)
Tributary			
Boone	75,800	48,200	12,900
Cherokee	749,400	749,400	118,100
Douglas	1,082,000	1,020,000	237,500
Fontana	514,000	514,000	73,000
Norris	1,113,000	1,113,000	512,000
South Holston	252,800	220,000	106,000
Tellico	120,000	120,000	32,000
Watauga	152,800	152,800	108,500
Main River			
Fort Loudoun	111,000	111,000	30,000
Watts Bar	379,000	379,000	165,000
Total	4,549,800	4,427,400	1,395,000

Table 2.4-7 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 1 of 5)

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

Table 2.4-7 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 2 of 5)

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

Table 2.4-7 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 3 of 5)

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

Table 2.4-7 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 4 of 5)

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

Table 2.4-7 Peak Streamflow of the Tennessee River at Chattanooga, TN (USGS Station 03568000) 1867 – 2007
(Page 5 of 5)

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

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

(b) Not reported.

[36]

Table 2.4-8 Weir Length Description and Coefficients of Discharge For Areas 3 and 4
(Sheet 1 of 1)

	Weir Parameters			
Watershed Area	**Description**	**Length Feet**	**Description of Control**	**Coefficient of Discharge "C" in Q=CLH3/2**
3	Area bounded by Reactor and Turbine Buildings to the west, embankment to the north, and railroad tracks to the east and south.	450(1)	Main plant track, elevation = 728.0	3.0
		555	Transformer yard track, elevation = 728.25	3.0
4	Area consisting of the switchyard area west of the condenser tube access track and south of the Turbine Building.	590	East condenser tube access track elevation = 728.22	3.0
		1220	Perimeter road, elevation = 728.0	3.0
	Area consisting of the switchyard area east of the condenser tube access track and south of the transformer yard track	540(2)	East and south end of switchyard area, elevation = 728.0	3.0

(1) Actual crest length is 600 ft. Length reduced by 150 ft. to conservatively account for decreased flow through fence.

(2) Actual crest length is 1080 ft. Length reduced 50% to conservatively account for decreased flow through fence.

Table 2.4-9 Drainage Area Peak Discharge
(Sheet 1 of 1)

Watershed Area[1]	Description	Drainage Area (Acres)	Maximum Elevation	
1	Channel formed by the west end of the switchyard and the adjacent embankment.	15.92	728.85	Backwater was computed in the channel from the road leading to the chemical holdup ponds to the gatehouse and Office Building. The estimated peak discharge at the road (15.92 acres) was 570 cfs. Discharges at upstream locations were decreased proportional to drainage area.
2	Natural drain with flow to the west from the perimeter road to the access road.	67.0	728.81	Backwater was computed from the crossing the perimeter road, to the Reactor, Diesel Generator, and Waste Wvaporation System Buildings. The estimated peak discharge at the access road was 6053 cfs. This includes flow from area 2 (67 acres), flow over the construction access road north of the site (100 acres), and flow from 60 acres to the northwest of the site. Discharges at upstream locations were decreased proportional to drainage area.
3	Pool bounded by Reactor and Turbine Buildings to the west, embankment to the north, and railroad tracks to the east and south.	22.9	728.87	The inflow hydrograph was routed with a starting elevation of 728.0 and outflow over the main and transformer yard railroad tracks.
	Flow reach extending from the main railroad track to a section between the east Reactor Ruilding and embankment to the north.	10.0	728.79	Backwater was computed from the main railroad track to the Reactor Building. The estimated peak discharge at the railroad was 420 cfs. Discharges at upstream locations were decreased proportional to drainage area.
4	Pool consisting of the switchyard area west of the condenser tube access track and south of the Turbine Building.	11.4	728.50	The inflow hydrograph was routed with a starting elevation of and outflow to the south and west over the perimeter road, and to the east over the condenser tube access track.
	Pool consisting of the switchyard area east of the condenser tube access track and south of the transformer yard track.	6.2	728.75	The inflow hydrograph was routed with a starting elevation of 728.0 and outflow to the south and east toward the perimeter road.
North of Site	Pool bounded by the embankment to the south of the access railroad to the east.	50	725.67	The inflow hydrograph was routed with a starting elevation of 716 (invert of double 96-inch pipe) and outflow through the double 96-inch pipe.
	Pool bounded by the dike to the east, the access highway to the west, and the construction access road to the south.	100	735.28[2]	The inflow hydrograph routed with a starting elevation of 727.1 (invert of 81- by 59-inch pipe arch) and outflow through the pipe arch and over the construction access road.

1. Watershed areas 1 - 4 are shown on Figure 2.4-40a; 150-acre area north of site shown on Figure 2.4-40b.

2. Maximum elevation reached at the construction access road.

Table 2.4-10 Seasonal Variations of Rainfall (PMP)

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

Source: HMR Report 41

Table 2.4-11 Probable Maximum Storm Precipitation and Precipitation Excess
(Page 1 of 2)

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

Table 2.4-11 Probable Maximum Storm Precipitation and Precipitation Excess (Continued)
(Page 2 of 2)

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

a. Unit area corresponds to Figure 2.4-9 numbered areas.

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

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

Table 2.4-12 Historical Flood Events

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

Table 2.4-13 Unit Hydrograph Data
(Page 1 of 2)

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

Table 2.4-13 Unit Hydrograph Data (Continued)
(Page 2 of 2)

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

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

Table 2.4-14 Floods From Postulated Seismic Failure of Upstream Dams (Plant Grade is Elevation 728)

		Watts Bar Nuclear Plant Elevation
OBE Failures With One-half Probable Maximum Flood		
1.	Norris	721.5
2.	Cherokee-Douglas	723.1[f]
3.	Fontana[a]	725.2[f]
4.	Watts Bar Gate opening prevented by bridge failure	715.5[b]
5.	Fort Loudoun Gate opening prevented by bridge failure	No elevations calculated; would be significantly less than for Cherokee-Douglas failure, line 2.
SSE Failures with 25-Year Flood		No elevations calculated; would be considerably lower than for Norris failure in OBE with 1/2 PMF.
6.	Norris	
7.	Norris, Cherokee, Douglas[d]	727.5
8.	Norris, Douglas, Fort Loudoun, Tellico	722.8
9.	Fontana-Douglas[d]	721.2[c]
10.	Fontana, Fort Loudoun, Tellico[e]	720.7[c]

a. Includes failure of five ALCOA dams--Nantahala, upstream; Santeetlah, on a downstream tributary; and Cheoah, Calderwood, and Chilhowee, downstream. Fort Loudoun gates are inoperable in open position.

b. Watts Bar tailwater elevation. Elevation at nuclear plant will be less. Not re-evaluated in 1998 reanalysis.

c. Not re-evaluated in 1998 reanalysis.

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

e. Gate opening at Watts Bar prevented by bridge failure.

Table 2.4-15 Well and Spring Inventory
Within 2-mile Radius of Watts Bar Nuclear Plant Site
(1972 Survey Only)
(Page 1 of 4)

Map Ident No.	Location Latitude	Longitude	Depth	Estimated Elevation Ground (feet)	Water Surface (feet)	Casing Size	Pump Data
1	35°36'08"	87°47'03"	200+	743	712	0.5	*No pump
2	35°36'24"	84°47'41"	59	726	723	0.5	*No pump
3	35°36'10"	84°47'50"	102	721	704	0.5	*No pump
4	35°36'00"	84°47'48"	43.5	730	718	0.5	*No pump
5	35°35'42"	84°47'49"	45	710	687	0.5	*No pump
6	35°35'55"	84°47'48"	6	705	705	2.5	*No pump
7	35°36'04"	84°48'16"	107	710	684	0.5	*No pump
8	35°36'11"	84°48'16"	30	702	684	4.0	*No pump
9	35°36'23"	84°48'06"	**	-	740	-	No pump
10	35°37'15"	84°49'04"	99	742	696	0.5	1/3 hp
11	35°37'06"	84°49'10"	87	753	Unknown	0.5	1/2 hp
12	35°37'03"	84°49'04"	150	704	700	0.5	1/2 hp
13	35°37'05"	84°49'02"	175	704	698	0.5	1 hp
14	35°37'15"	84°49'01"	140	740	720	0.5	1 hp
15	35°37'03"	84°48'48"	83	729	693	0.5	Hand pump
16	35°36'46'	84°48'18"	205	780	665	0.5	Submerged, Unknown
17	35°36'34"	84°48'13"	28	768	768	0.5	1 hp
18	35°36'30"	84°48'20"	95	794	777	0.5	1 hp
19	35°35'35"	84°48'52"	111	713	715	0.6	No pump, 1 gpm
20	35°36'54"	84°49'10"	68	710	Unknown	0.5	Unknown
21	35°36'18"	84°49'24"	125	725	695	0.5	1/2 hp
22	35°36'20"	84°49'20"	130	729	655	0.5	3/4 hp
23	35°35'20"	84°48'55"	225	730	715	0.5	1 hp
24	35°35'15"	84°48'56"	79	715	705	0.5	1/2 hp
25	35°35'44"	84°49'07"	14	805	804	8.0	No pump
26	35°35'46"	84°49'31"	385	718	Unknown	0.5	1/2 hp
27	35°35'29"	84°49'16"	240	770	600	Unknown	Unknown
28	35°37'14"	84°47'04"	***	-	Watts Bar Lake 735 - 745	-	2, 50 hp=500 gpm
29	35°37'19"	84°45'57"	100	706	660	0.5	1 hp
30	35°36'39"	84°45'59"	65	714	unknown	0.5	1/2 hp
31	35°35'49"	84°46'15"	Spring	-	710	-	No pump

Table 2.4-15 Well and Spring Inventory
Within 2-mile Radius of Watts Bar Nuclear Plant Site (Continued)
(1972 Survey Only)
(Page 2 of 4)

Map Ident No.	Location Latitude	Location Longitude	Depth	Estimated Elevation Ground (feet)	Estimated Elevation Water Surface (feet)	Casing Size	Pump Data
32	35°36'19"	84°45'21"	32.5	747	740	2'-10" Square	Windlass and bucket, no pump
33	35°35'26"	84°46'44"	Spring	-	800	-	No pump
34	35°35'25"	84°47'02"	120	725	705	Unknown	4 hp
35	35°35'12"	84°47'15"	225	730	710	0.5	No pump
36	35°35'19"	84°47'25"	110	734	715	0.5	3/4 hp
37	35°35'15"	84°47'25"	175	730	710	0.7	No pump
38	35°35'14"	84°47'27"	100	730	710	0.7	3/4 hp
39	35°37'26"	84°45'50"	40	710	702	0.5	1/4 hp
40	35°35'16"	84°47'28"	165	725	705	0.5	3/4 hp
41	35°35'19"	84°47'30"	110	734	695	0.5	3/4 hp
42	35°35'14"	84°47'28"	73	724	724	0.5	No pump
43	35°35'14"	84°47'22"	105	724	720	0.5	1/2 hp
44	35°35'12"	84°47'29"	Spring	-	710	-	1/2 hp
45	35°35'15"	84°47'16"	125	730	690	0.5	1/2 hp
46	35°35'09"	84°47'31"	105	730	722	0.5	1-1/2 hp
47	35°35'14"	84°47'41"	164	764	755	0.5	1-1/2 hp
48	35°36'55"	84°45'35"	Spring	-	720	-	3/4 hp
49	35°35'00"	84°47'50"	100	748	708	0.5	1-1/2 hp
50	35°34'48"	84°47'42"	80	710	688	0.5	3/4 hp
51	35°35'02"	84°47'38"	100	750	720	0.5	1/2 hp
52	35°34'58"	84°47'34"	99	722	711	0.5	2 hp
53	35°34'55"	84°47'37"	54	719	691	0.5	3/4 hp
54	35°34'44"	84°47'48"	52	718	703	3.0	Not used
55	35°34'39"	84°47'50"	257	720	692	0.5	5 gpm for five houses, lowered well 20 feet
56	35°34'39"	84°47'29"	56	701	691	0.5	1hp
57	35°34'37"	84°47'32"	252	714	602	0.5	125 gph, 1 hp
58	35°34'59"	84°47'33"	Spring	-	710	-	Not used
59	35°35'03"	84°47'38"	Spring	-	730	-	Cattle pond
60	35°35'04"	84°47'58"	Spring	-	710	-	Not used

Investigation made on January 10-11, 1972.
* Residence purchased for Watts Bar Nuclear Plant construction.
** Spring fed pond of approximately 50 feet in diameter.
*** Watts Bar Dam, Steam Plant, and Pete Smith Resort water supply taken from Watts Bar Lake.

Table 2.4-15 Well and Spring Inventory
Within 2-mile Radius of Watts Bar Nuclear Plant Site (Continued)
(1972 Survey Only)
(Page 3 of 4)

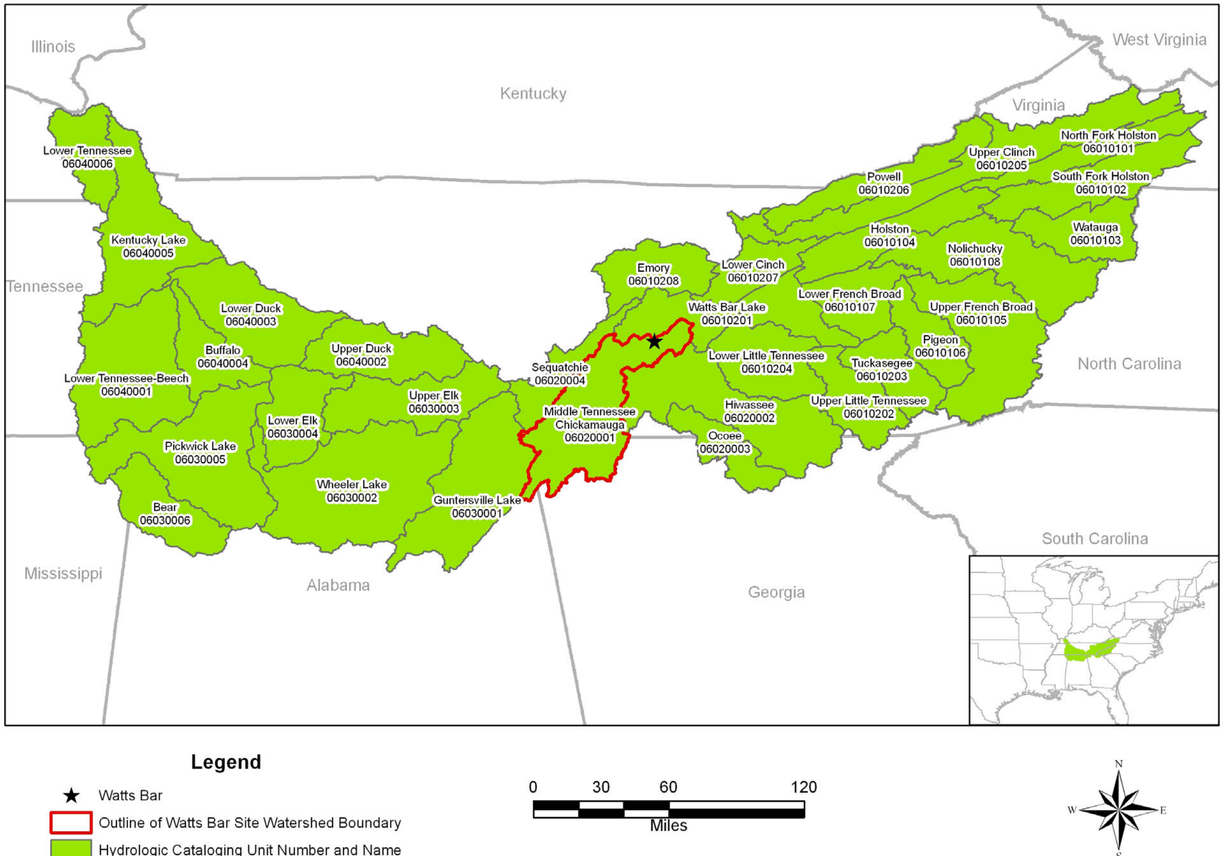
Map Ident No.	Location Latitude	Location Longitude	Depth	Estimated Elevation Ground (feet)	Estimated Elevation Water Surface (feet)	Casing Size	Pump Data
61	35°36'58"	84°45'22"	NA*	750	NA	NA	NA
62	35°36'50"	84°45'24"	NA	710	NA	NA	NA
63	35°35'42"	84°47'32"	150	742	INK**	0.5	Yes
64	35°37'16"	84°49'00"	100	740	50	0.33	Yes
65	35°36'29"	84°48'20"	200	710	19	0.5	Yes
66	35°36'52"	84°49'08"	70-83	700	INK	0.5	Yes
67	35°36'50"	84°49'08"	70-83	700	INK	0.5	Yes
68	35°36'49"	84°49'09"	70-83	700	INK	0.5	Yes
69	35°36'47"	84°49'10"	70-83	700	INK	0.5	Yes
70	35°37'03"	84°49'09"	NA	750	NA	NA	No
71	35°37'05"	84°49'10"	NA	750	NA	Hand dug	No
72	35°35'41"	84°49'16"	NA	720	NA	NA	NA
73	35°35'43"	84°48'48"	NA	800	NA	NA	NA
74	35°36'53"	84°48'49"	INK	720	INK	INK	Yes
75	35 35'07"	84°47'58"	100+	760	Below River	INK	Yes
76	35°35'07"	84°48'00"	INK	740	INK	INK	Yes
77	35 35'06"	84°48'01"	NA	720	NA	NA	NA
78	35°35'08"	84°48'01"	NA	720	NA	NA	NA
79	35°35'09"	84°47'54"	NA	800	NA	NA	NA
80	35°35'11"	84°47'42"	NA	760	NA	NA	NA
81	35 35'14"	84°47'41"	NA	760	NA	NA	NA
82	35°35'13"	84°47'37"	400+	760	INK	0.5	Yes
83	35°35'14"	84°47'37"	300+	760	INK	0.5	Yes
84	35°35'10"	84°47'34"	NA	740	NA	NA	NA
85	35°35'14"	84°47'31"	NA	720	NA	NA	NA
86	35-35'18"	84°47'26"	450	720	20	0.125	Yes
87	35°35'24"	84°47'14"	300	740	INK	INK	Yes
88	35°35'17"	84°47'15"	300	730	INK	0.5	Yes
89	35°35'19"	84°47'12"	265	730	INK	0.5	Yes
90	35°35'18"	84°47'12"	150	730	INK	0.5	Yes
91	35°35'17"	84°47'09"	NA	730	NA	NA	NA
92	35°35'14"	84°47'13"	NA	720	NA	NA	NA
93	35°35'06"	84°47'17"	210	720	20	0.5	Yes

Table 2.4-15 Well and Spring Inventory
Within 2-mile Radius of Watts Bar Nuclear Plant Site (Continued)
(1972 Survey Only)
(Page 4 of 4)

Map Ident No.	Location Latitude	Longitude	Depth	Estimated Elevation Ground ---feet---	Water Surface	Casing Size	Pump Data
94	35°35'08"	84°46'58"	130	760	15	0.5	Yes
95	35°35'08"	84°46'55"	NA	800	NA	NA	NA
96	35°35'19"	84°46'41"	80	990	20	0.5	Yes
97	35°35'22"	84°46'34"	600	960	INK	0.5	Yes
98	35°35'39"	84°46'34"	INK	740	INK	INK	Yes
S-99	35°37'04"	84°48'59"	Spring	710	-	-	No
S-100	35°35'45"	84°49'04"	Spring	840	-	-	No
S-101	35°35'40'	84°49'14"	Spring	730	-	-	No
S-102	35°35'16"	84°46'44"	Spring	980	-	-	No
S-103	35°35'06"	84°46'57"	Spring	800	-	-	No

* none available, many of these residences appeared to be summer houses, 2-3 attempts to locate home owners in the evening hours and on the weekend were unsuccessful.
**Information not known by homeowner.
***No pump sizes were known by current homeowners.

Table 2.4-16 Deleted by Amendment 98



WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

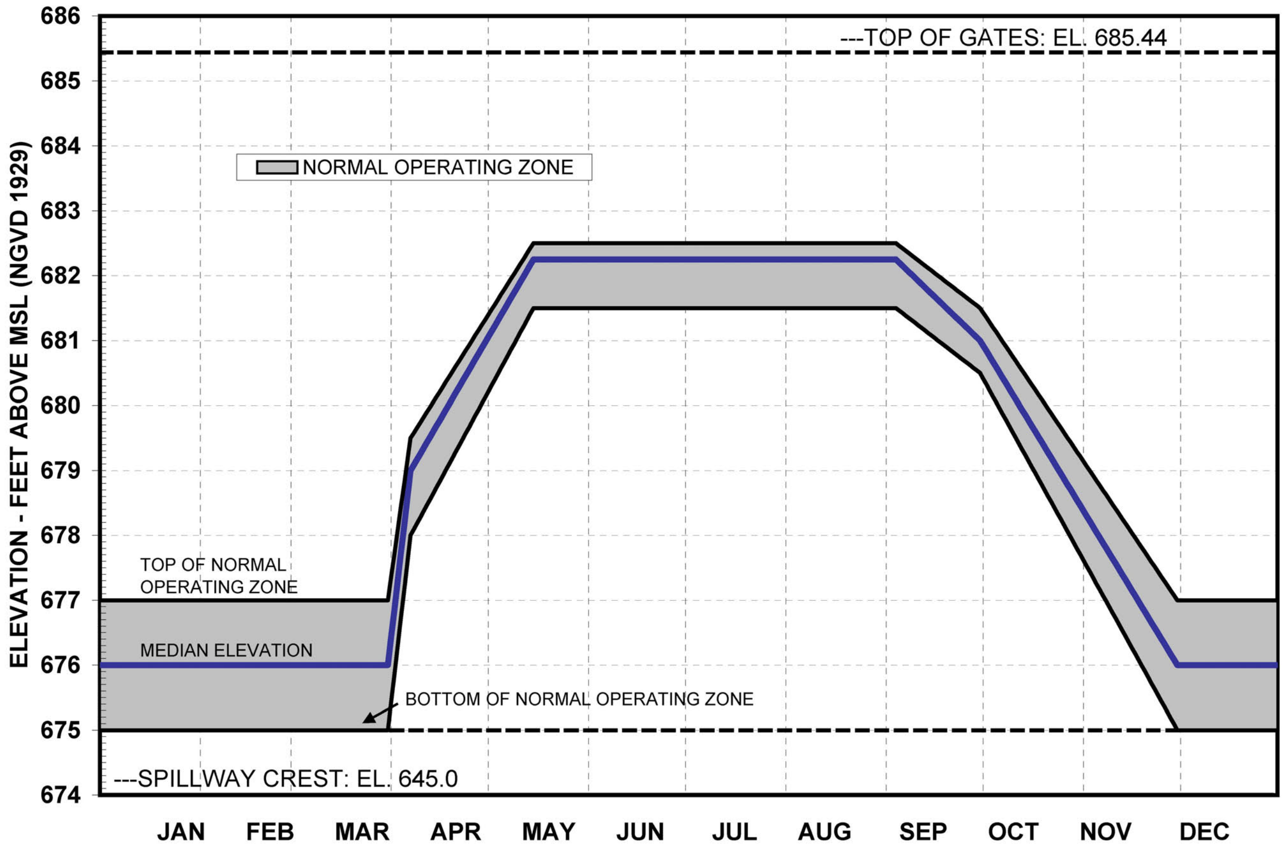
USGS Hydrologic Units within the Tennessee River Watershed

Figure 2.4-1

Figure 2.4-1 USGS Hydrologic Units within the Tennessee River Watershed

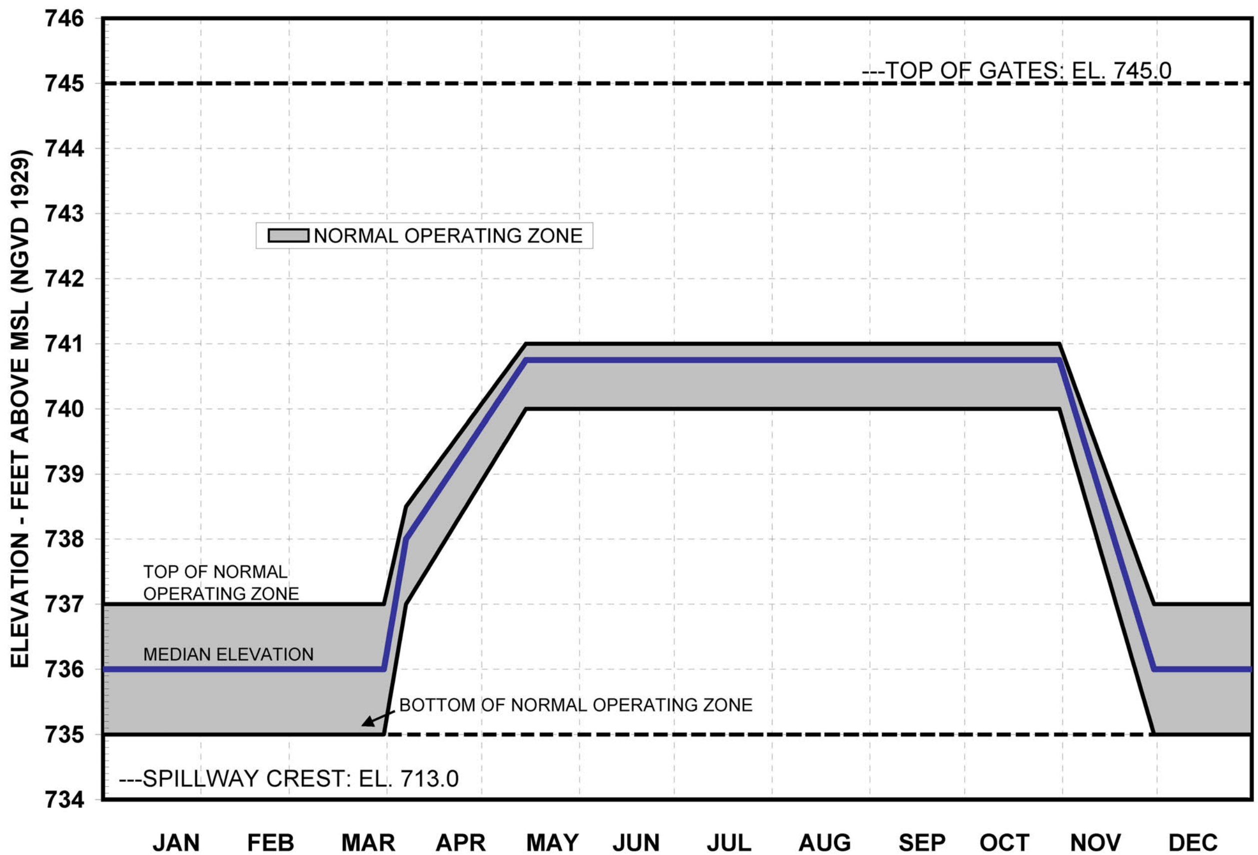
Security-Related Information - Withheld Under 10CFR2.390

Figure 2.4-2 TVA Water Control System



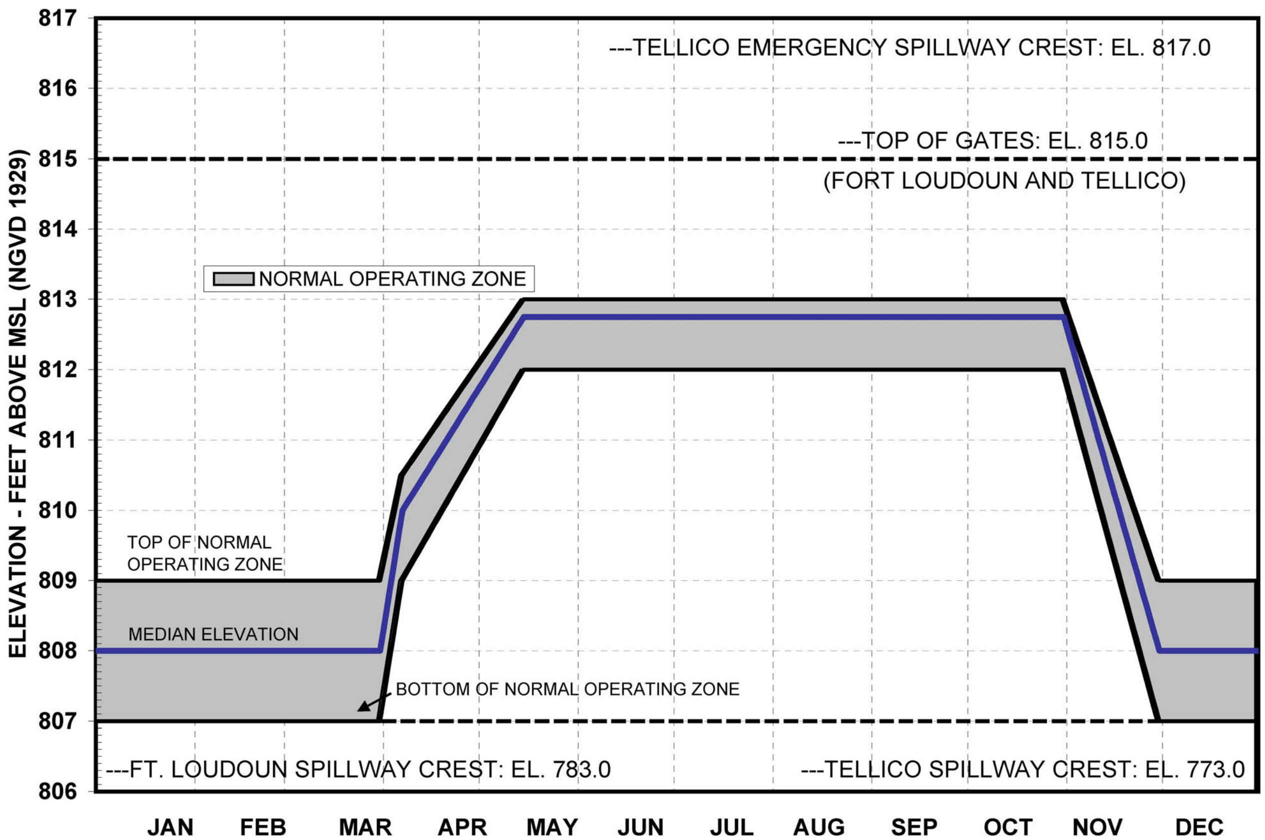
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Chickamauga Figure 2.4-3 (Sheet 1 of 12)

Figure 2.4-3 Seasonal Operating Curve, Chickamauga (Sheet 1 of 12)



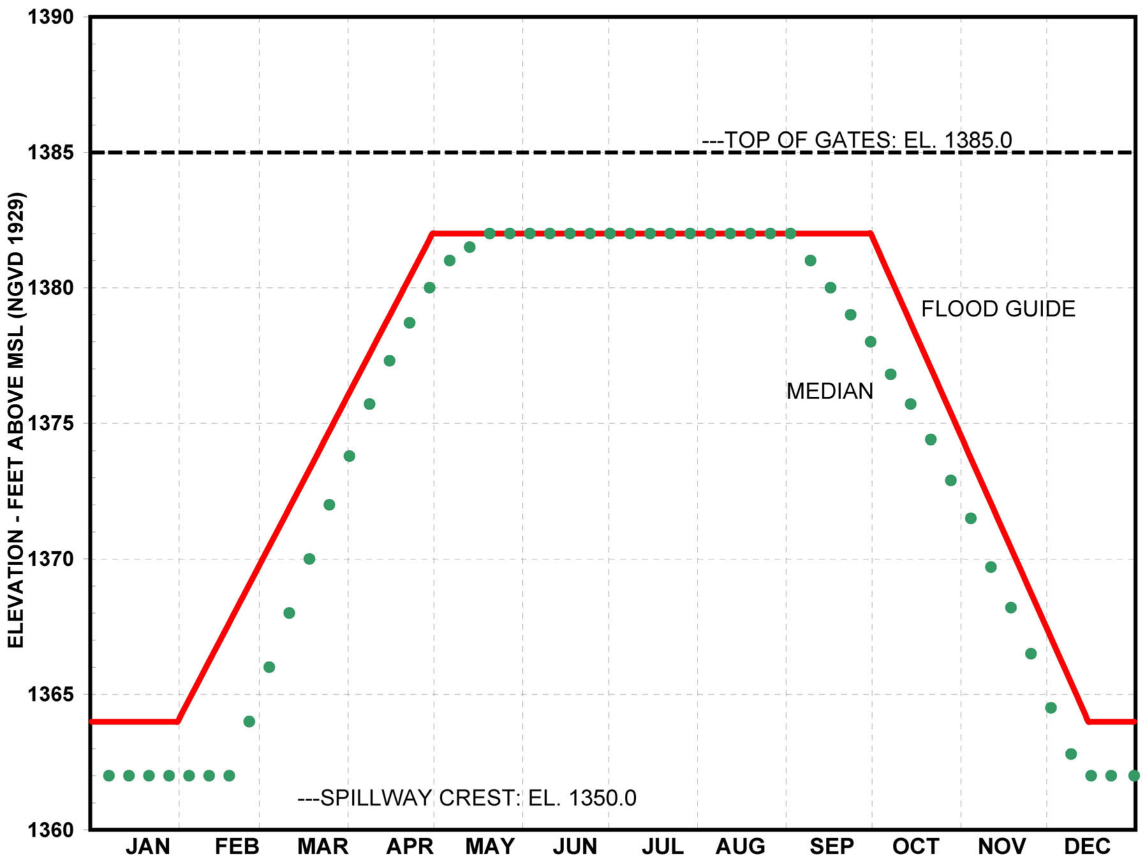
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Watts Bar Figure 2.4-3 (Sheet 2 of 12)

Figure 2.4-3 Seasonal Operating Curve, Watts Bar (Sheet 2 of 12)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Fort Loudoun - Tellico Figure 2.4-3 (Sheet 3 of 12)

Figure 2.4-3 Seasonal Operating Curve, Fort Loudoun - Tellico (Sheet 3 of 12)

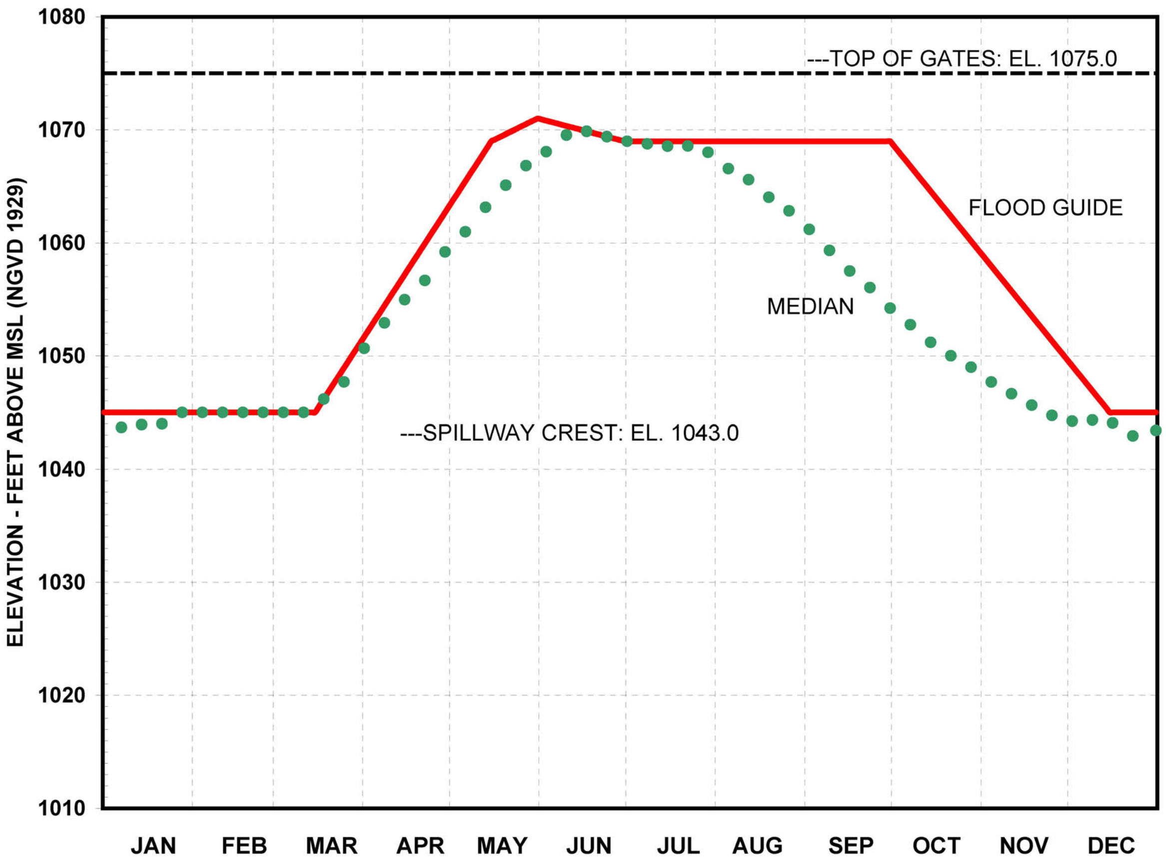


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Seasonal Operating Curve, Boone

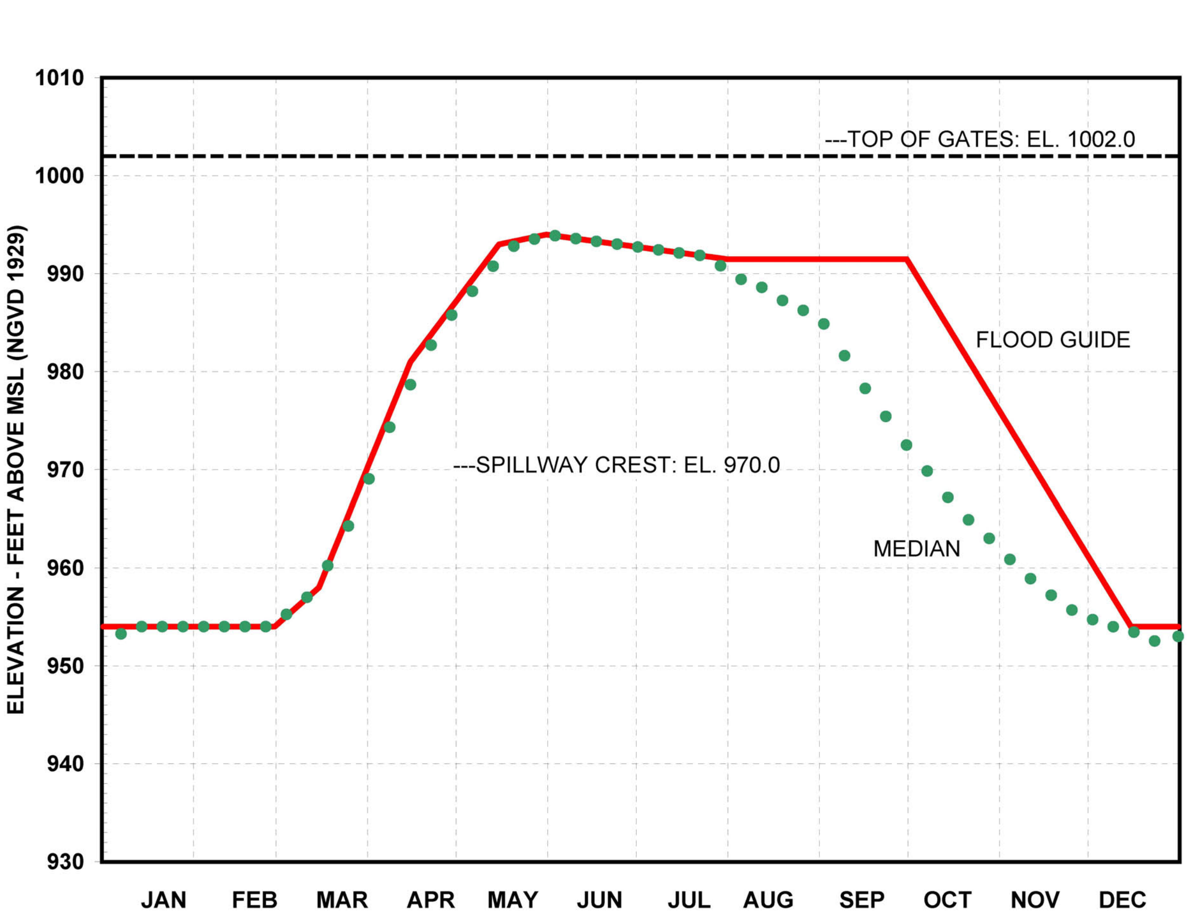
Figure 2.4-3 (Sheet 4 of 12)

Figure 2.4-3 Seasonal Operating Curve, Boone (Sheet 4 of 12)



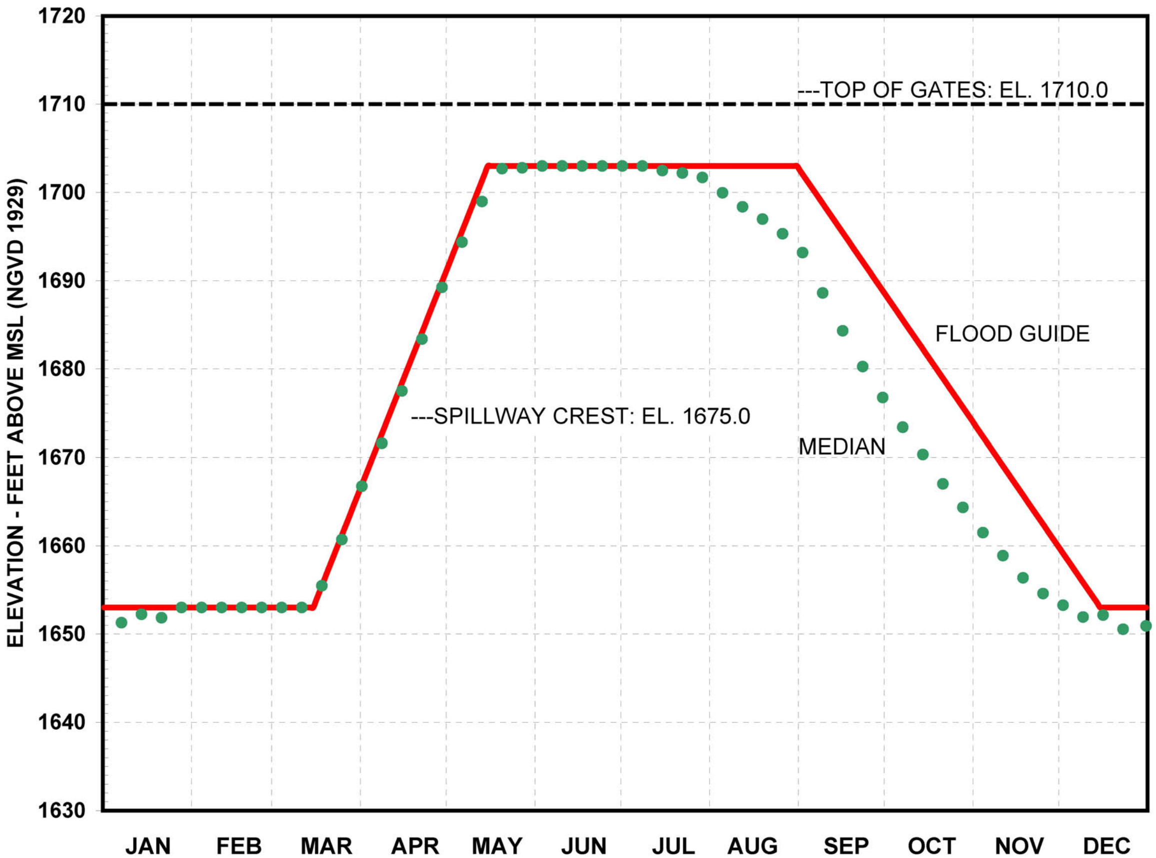
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Cherokee Figure 2.4-3 (Sheet 5 of 12)

Figure 2.4-3 Seasonal Operating Curve, Cherokee (Sheet 5 of 12)



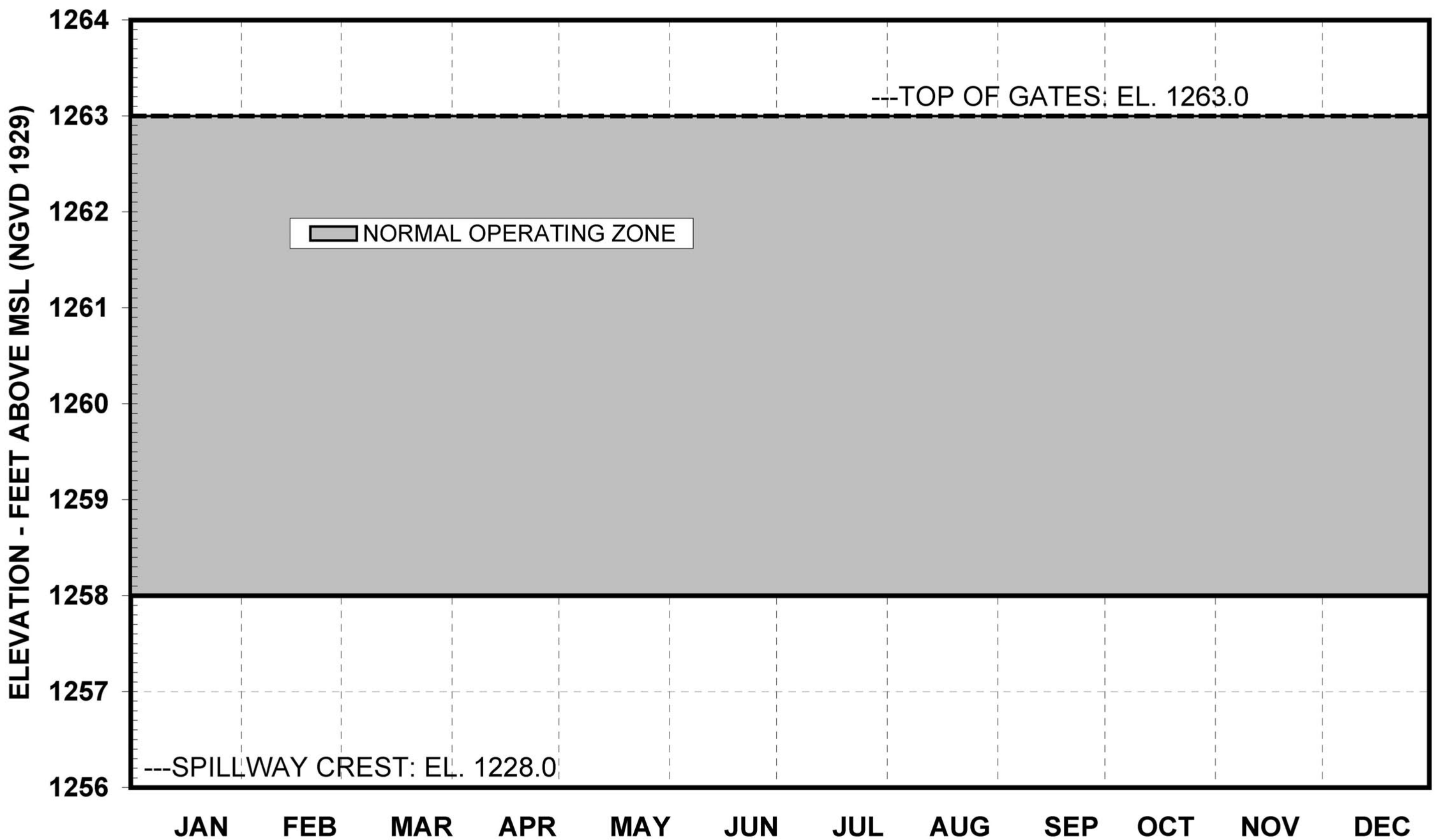
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Douglas Figure 2.4-3 (Sheet 6 of 12)

Figure 2.4-3 Seasonal Operating Curve, Douglas (Sheet 6 of 12)



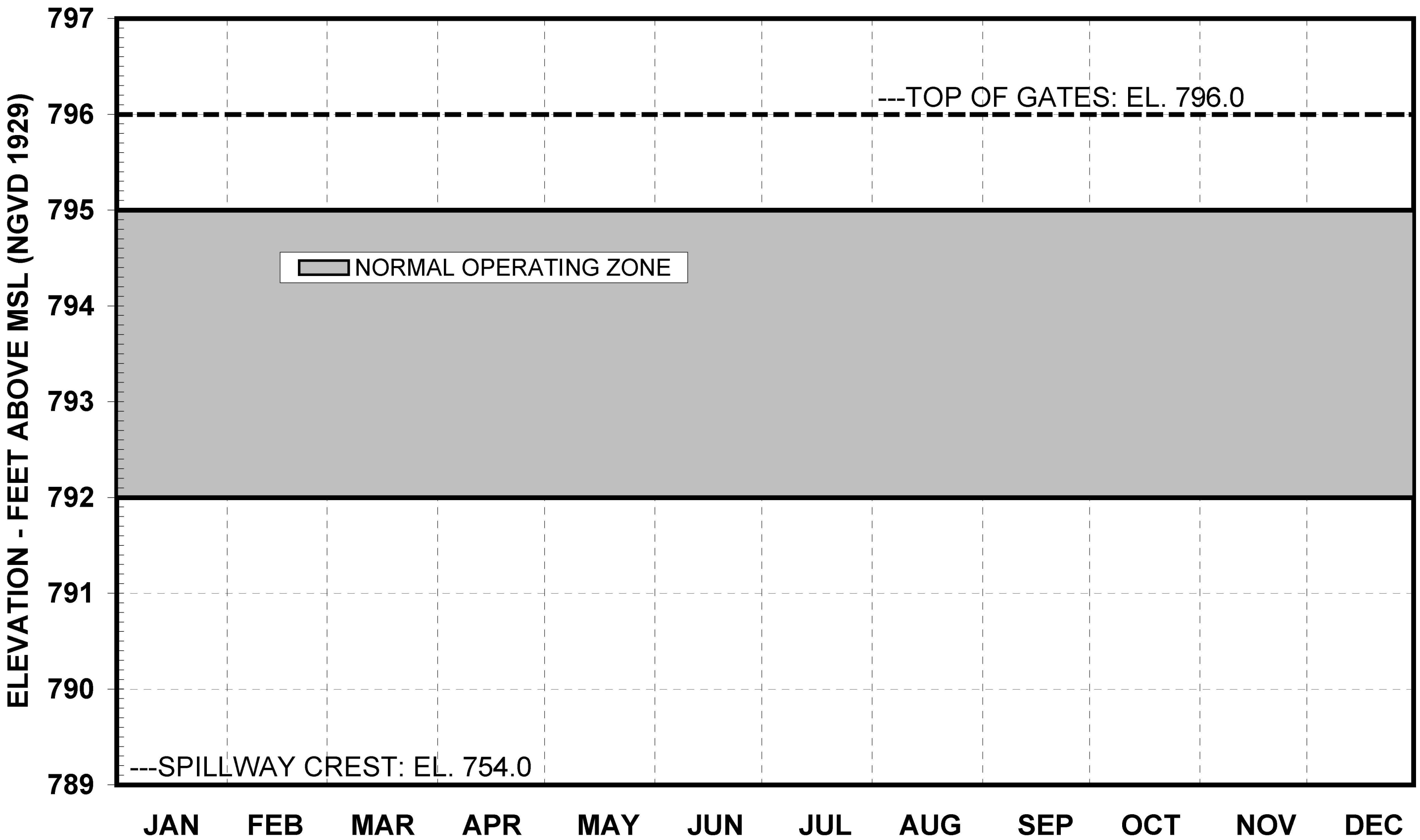
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Fontana Figure 2.4-3 (Sheet 7 of 12)

Figure 2.4-3 Seasonal Operating Curve, Fontana (Sheet 7 of 12)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Fort Patrick Henry Figure 2.4-3 (Sheet 8 of 12)

Figure 2.4-3 Seasonal Operating Curve, Fort Patrick Henry (Sheet 8 of 12)

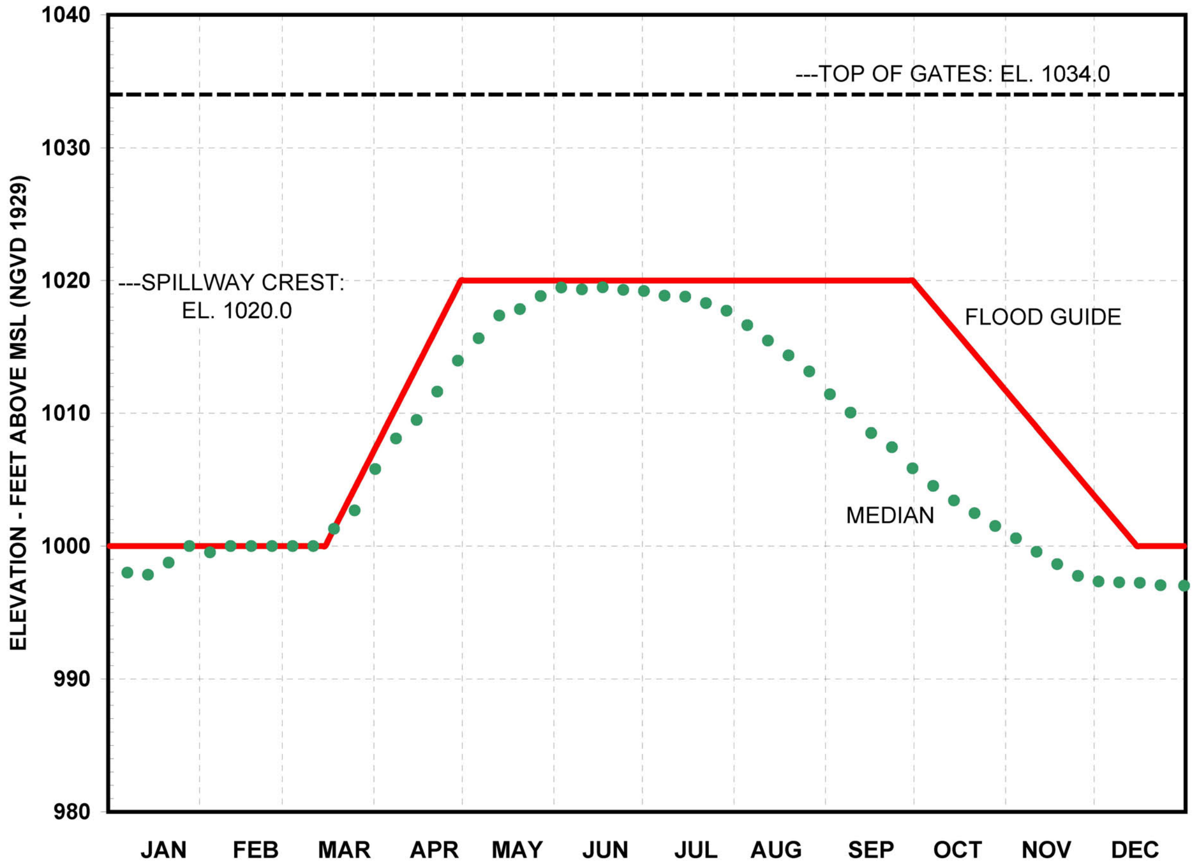


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Seasonal Operating Curve, Melton Hill

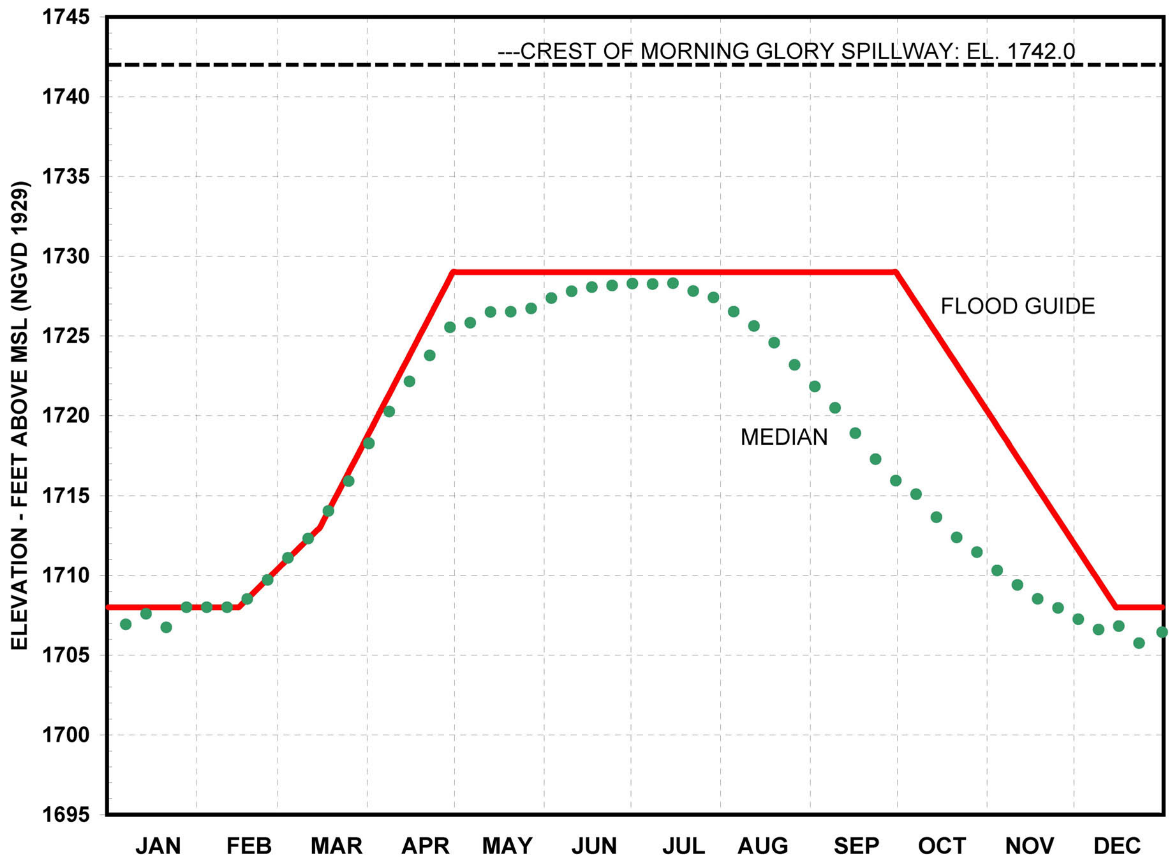
Figure 2.4-3 (Sheet 9 of 12)

Figure 2.4-3 Seasonal Operating Curve, Melton Hill (Sheet 9 of 12)



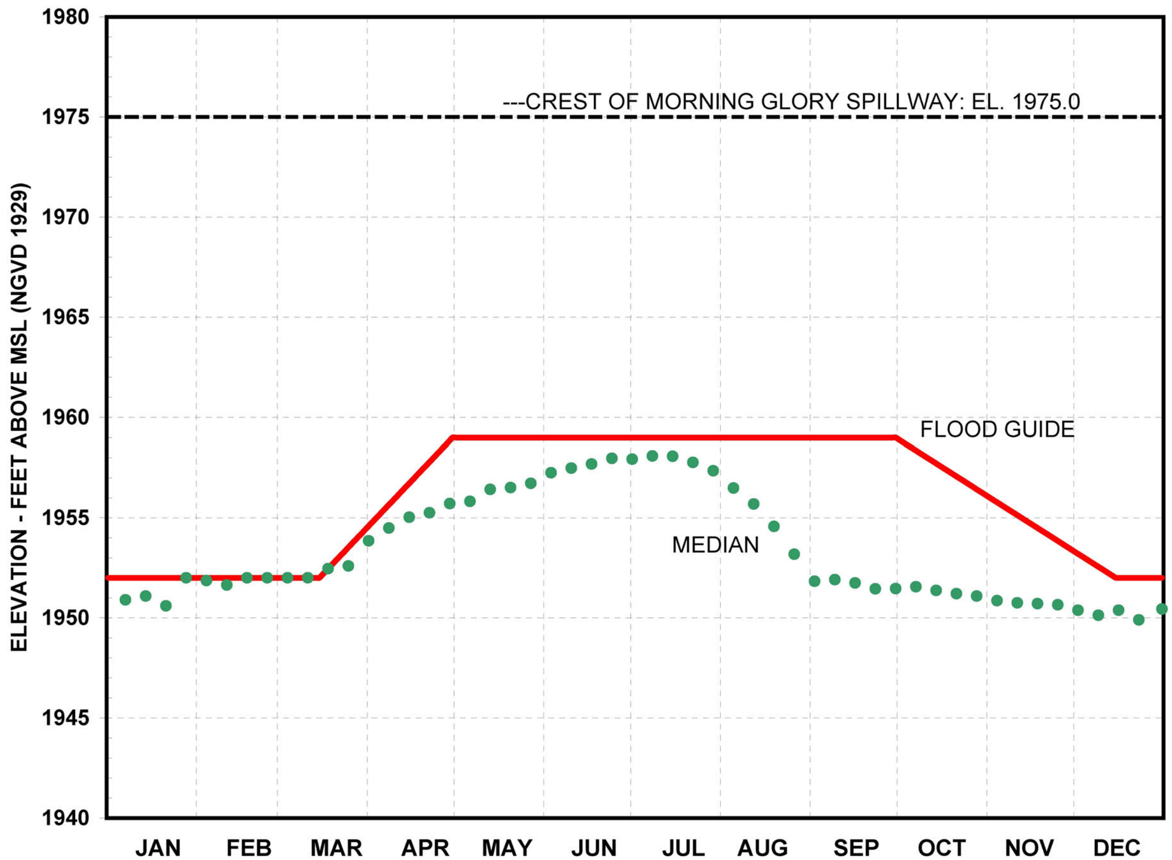
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Norris Figure 2.4-3 (Sheet 10 of 12)

Figure 2.4-3 Seasonal Operating Curve, Norris (Sheet 10 of 12)



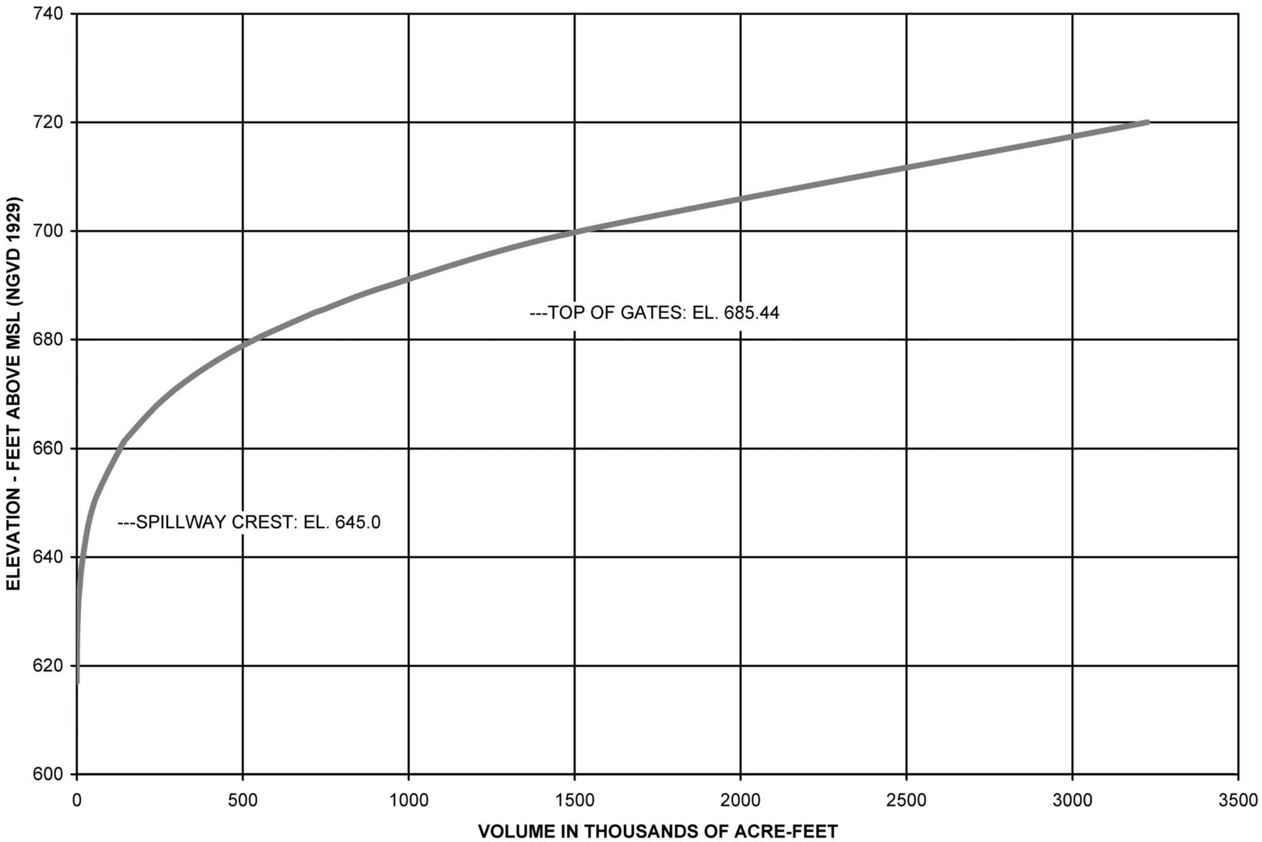
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, South Holston Figure 2.4-3 (Sheet 11 of 12)

Figure 2.4-3 Seasonal Operating Curve, South Holston (Sheet 11 of 12)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Seasonal Operating Curve, Watauga Figure 2.4-3 (Sheet 12 of 12)

Figure 2.4-3 Seasonal Operating Curve, Watauga (Sheet 12 of 12)

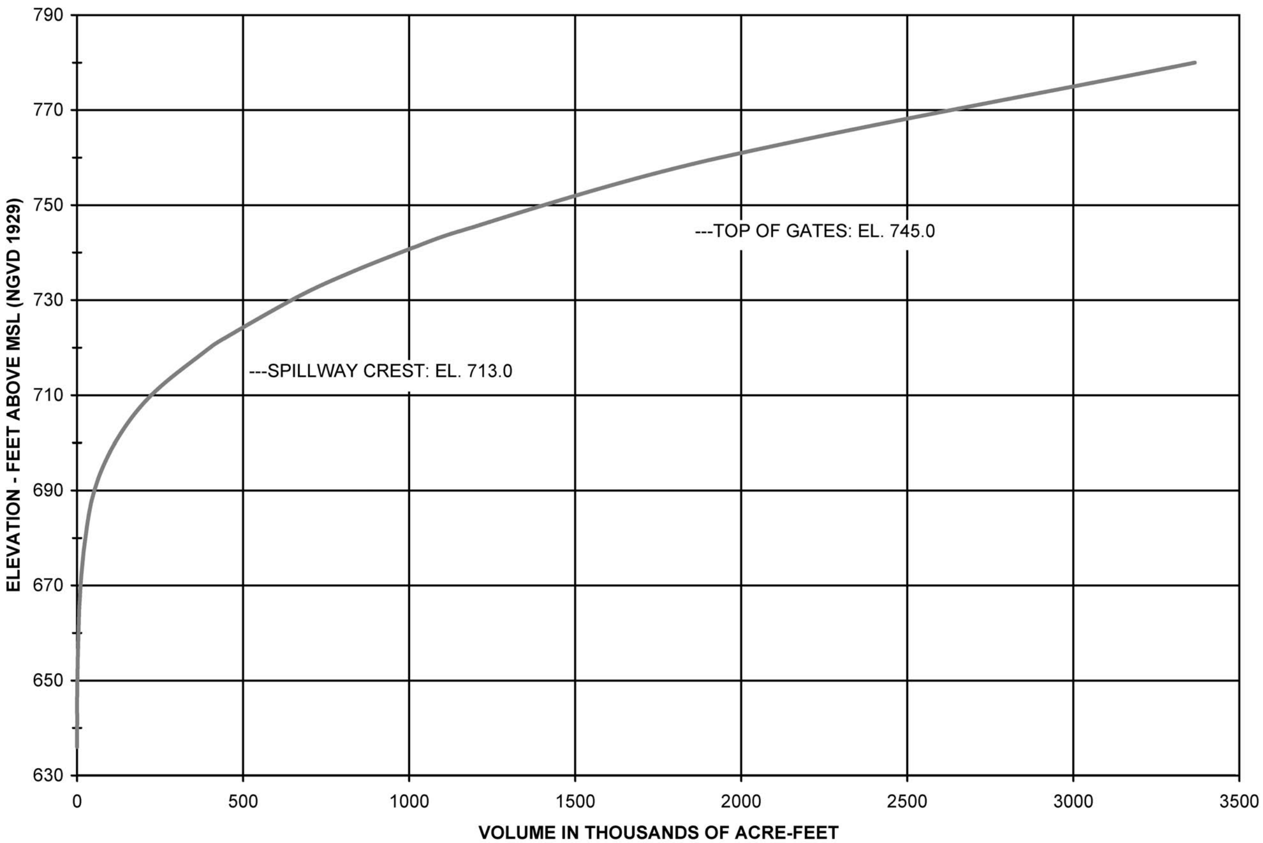


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Reservoir Elevation - Storage
Relationship, Chickamauga

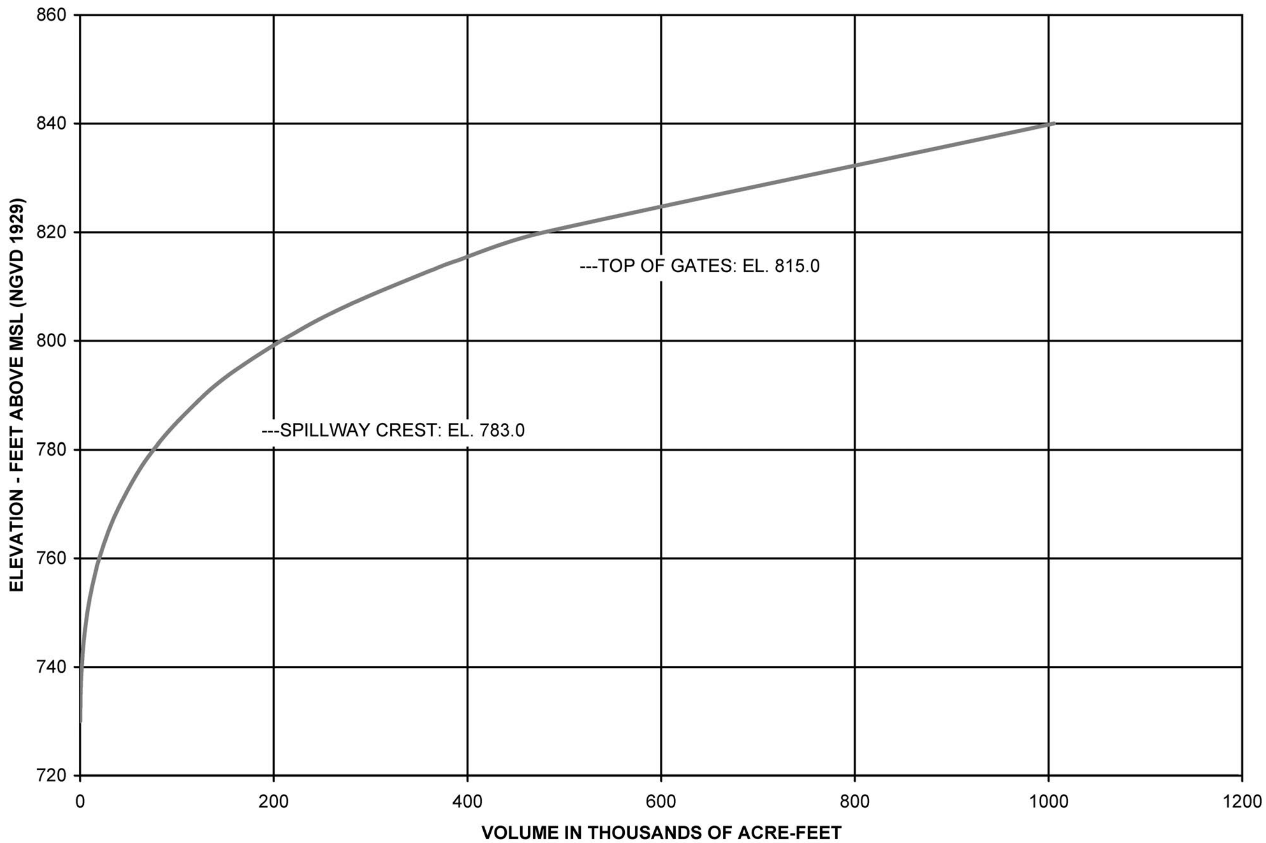
Figure 2.4-4 (Sheet 1 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Chickamauga (Sheet 1 of 13)



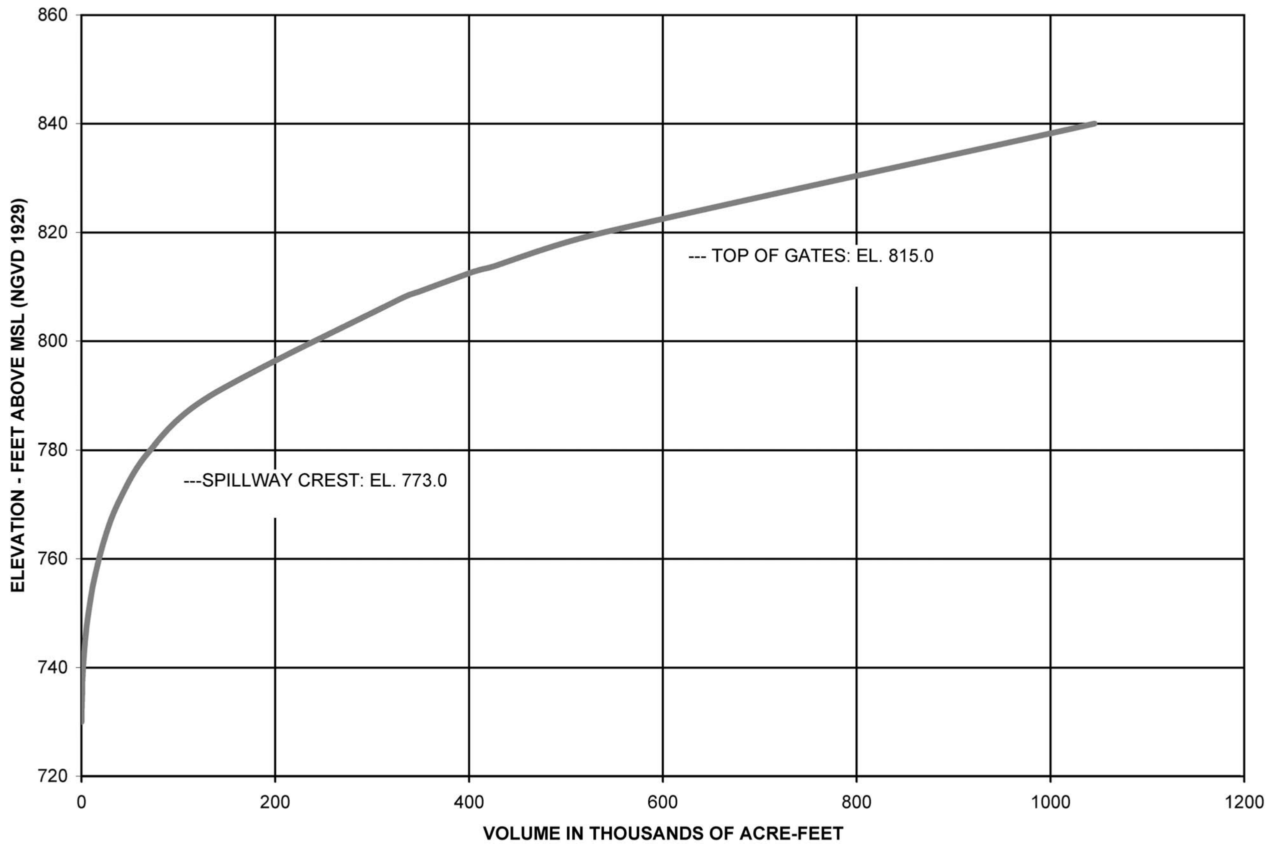
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Watts Bar Figure 2.4-4 (Sheet 2 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Watts Bar (Sheet 2 of 13)



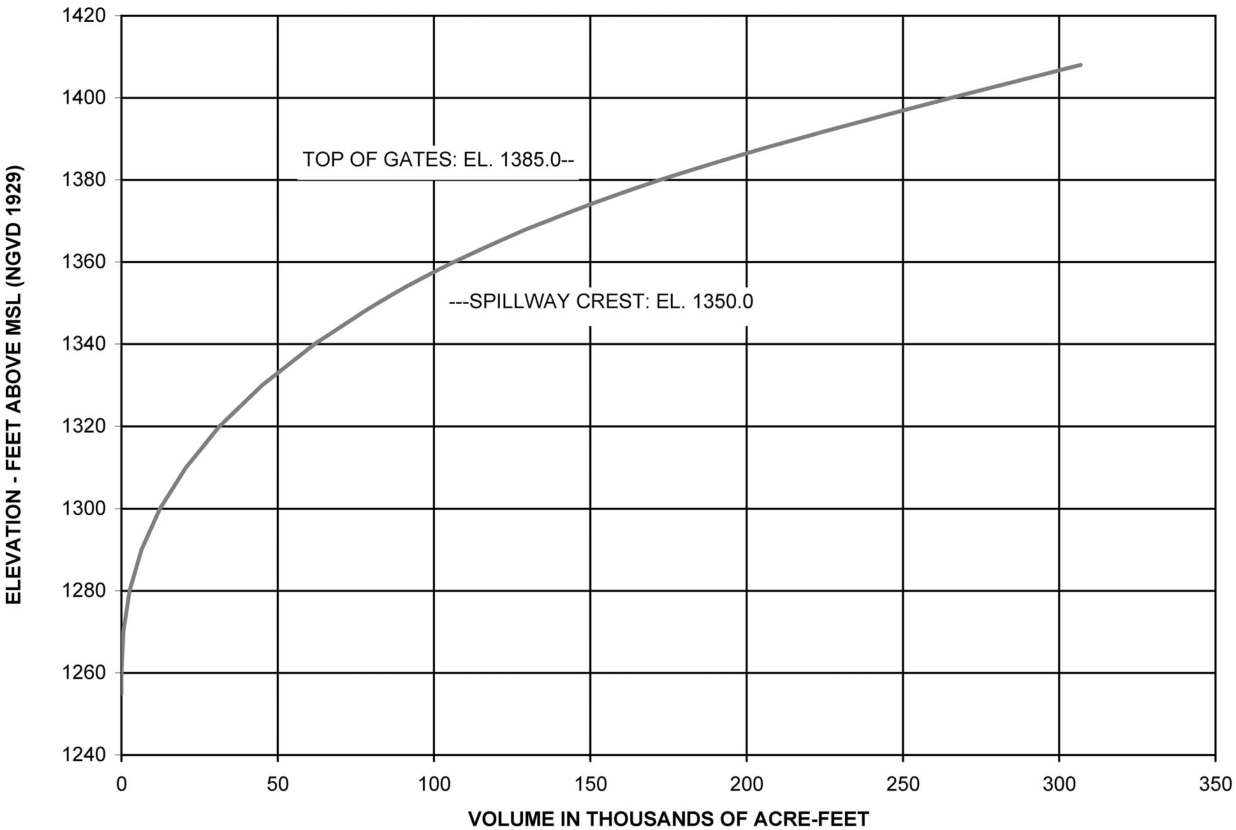
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Fort Loudoun Figure 2.4-4 (Sheet 3 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Fort Loudoun (Sheet 3 of 13)



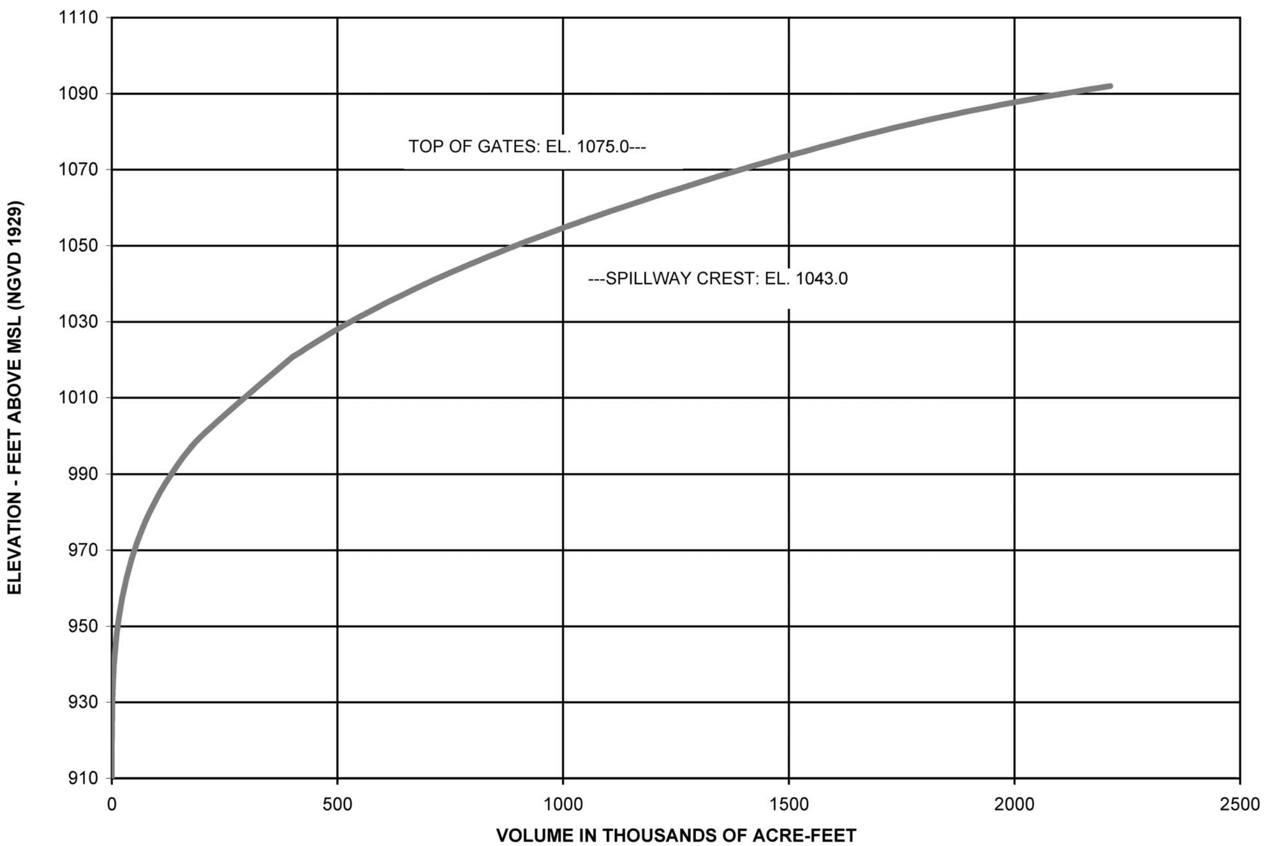
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Tellico Figure 2.4-4 (Sheet 4 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Tellico (Sheet 4 of 13)



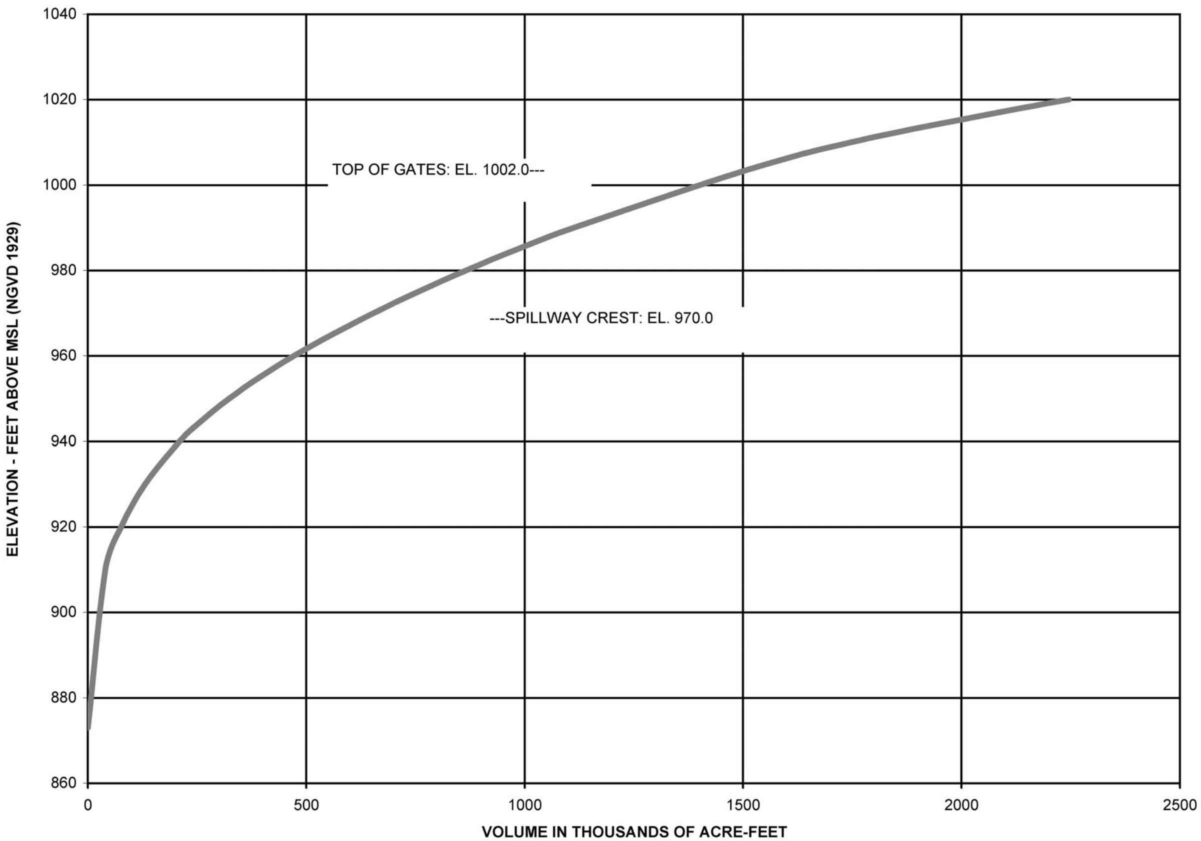
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Boone Figure 2.4-4 (Sheet 5 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Boone (Sheet 5 of 13)



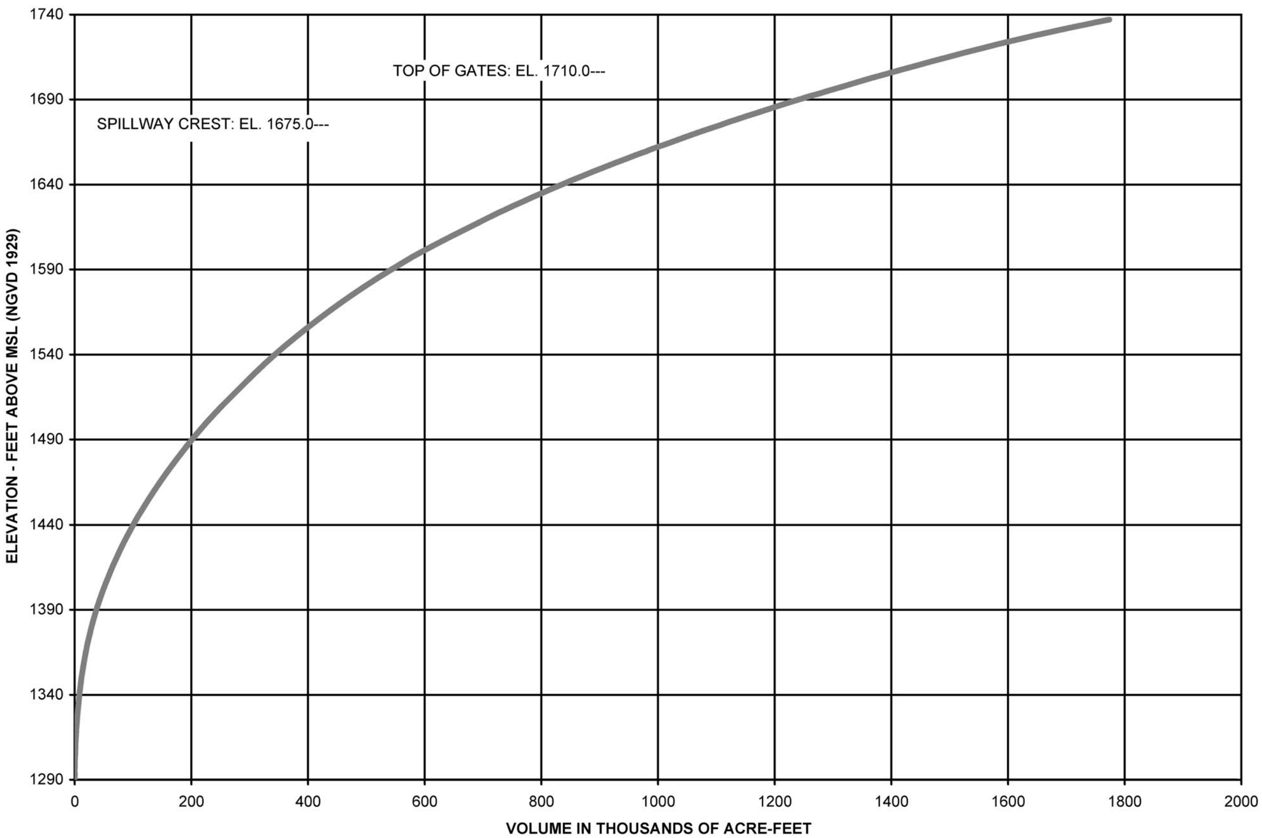
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Cherokee Figure 2.4-4 (Sheet 6 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Cherokee (Sheet 6 of 13)



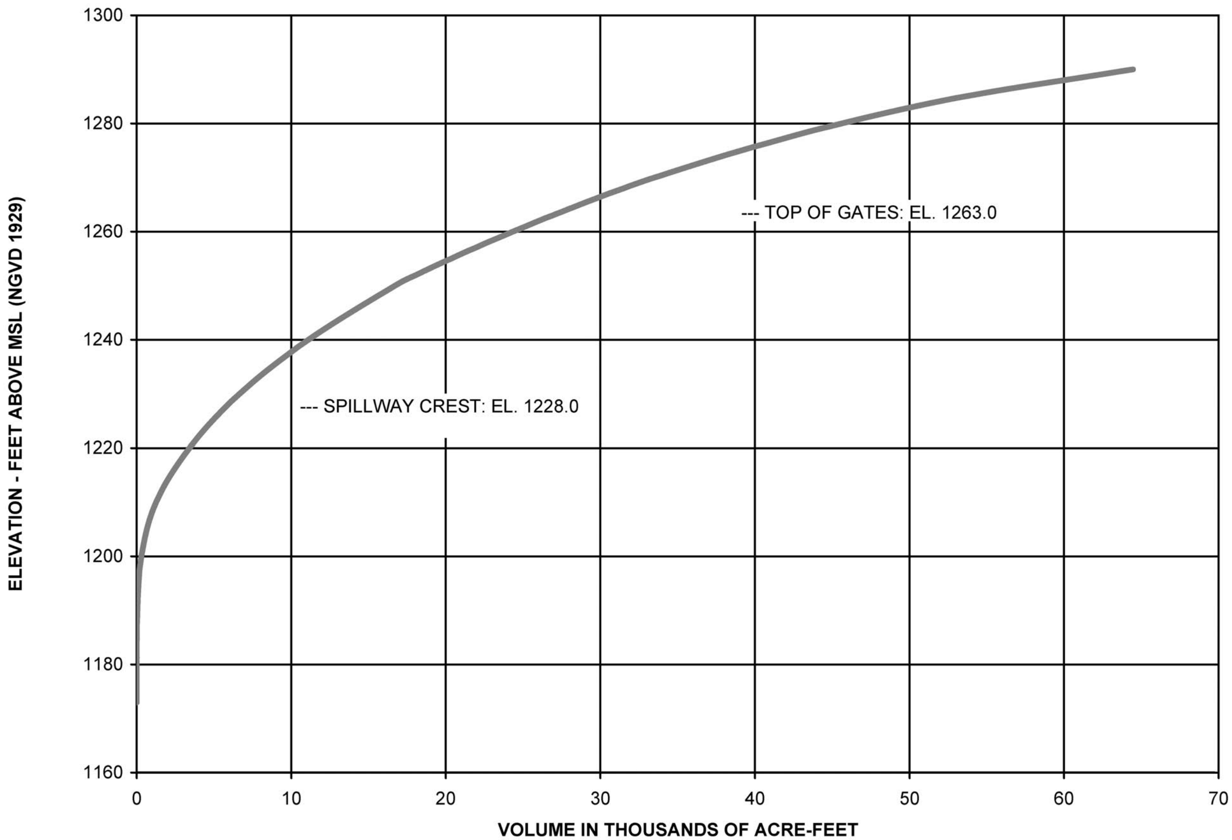
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Douglas Figure 2.4-4 (Sheet 7 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Douglas (Sheet 7 of 13)



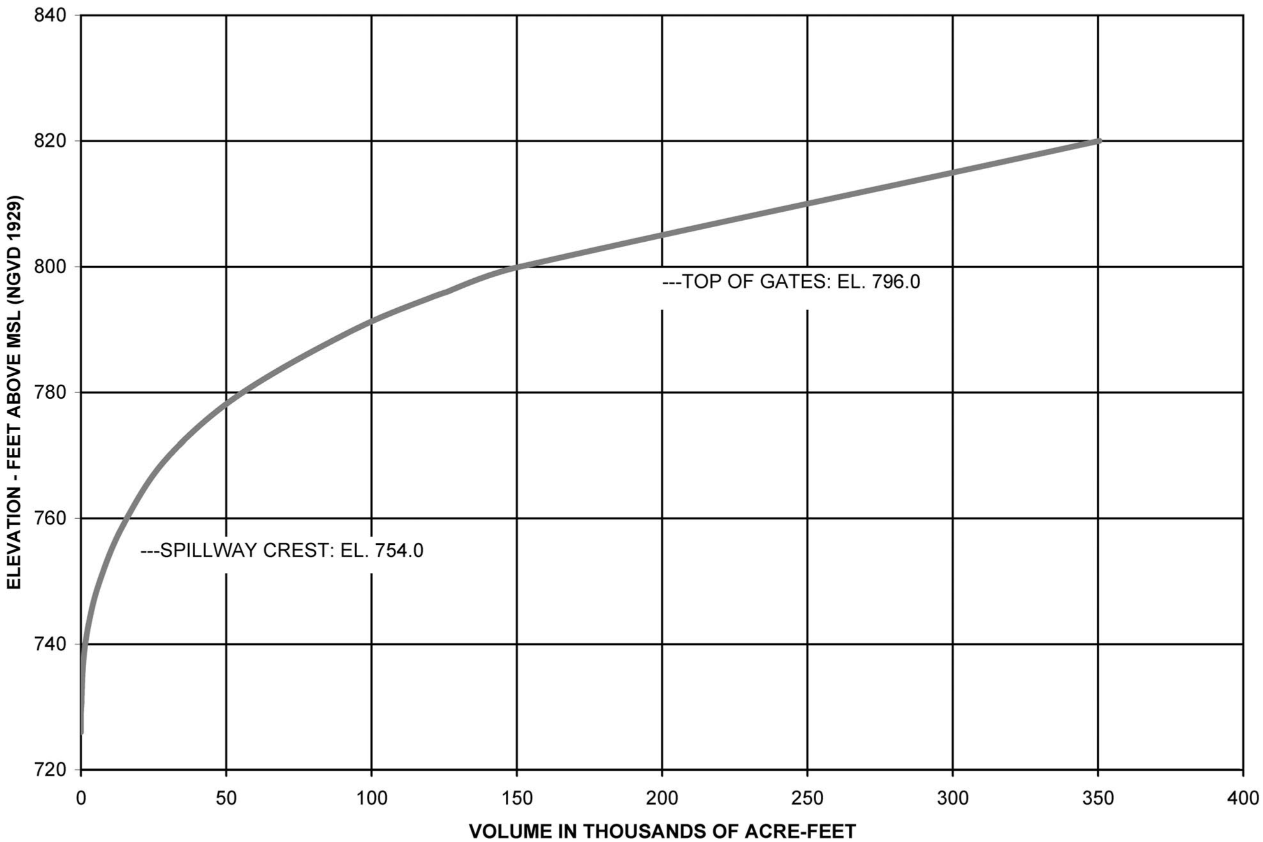
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Fontana Figure 2.4-4 (Sheet 8 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Fontana (Sheet 8 of 13)



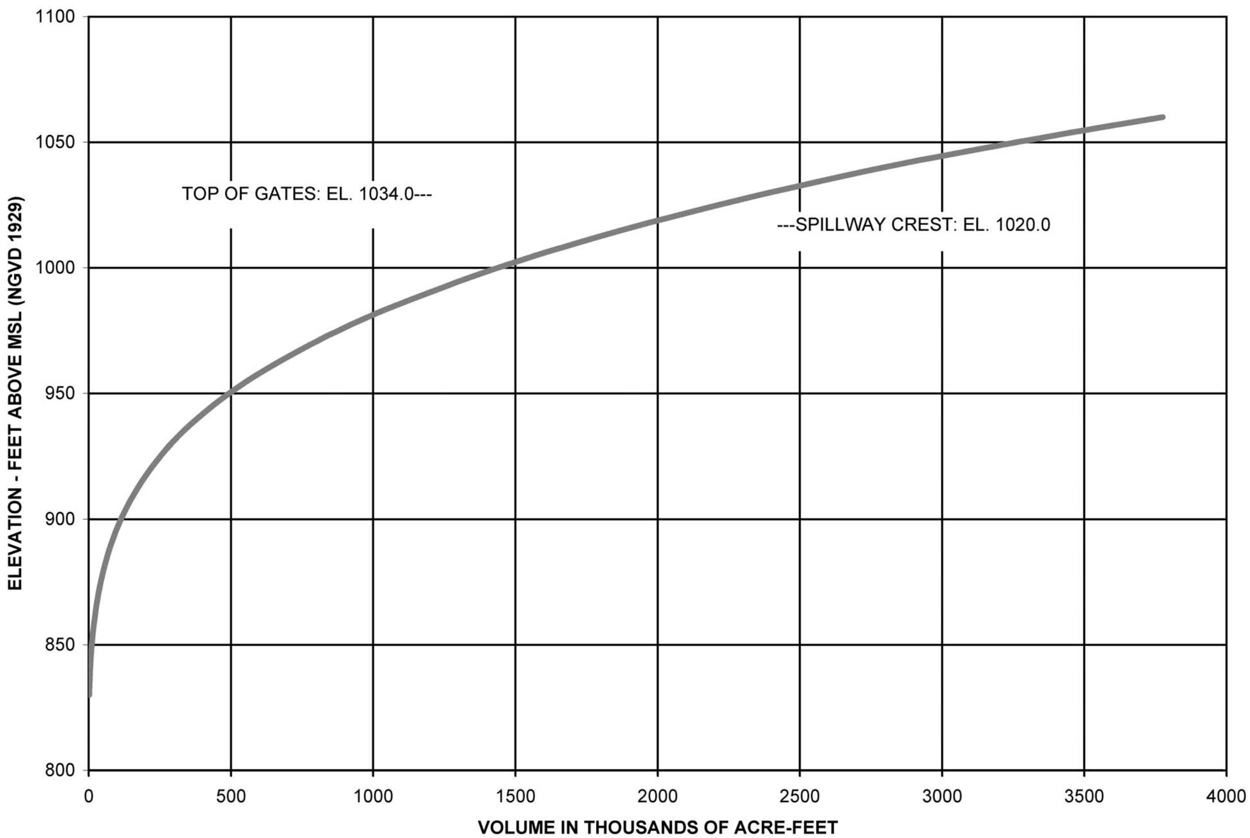
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Fort Patrick Henry Figure 2.4-4 (Sheet 9 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Fort Patrick Henry (Sheet 9 of 13)



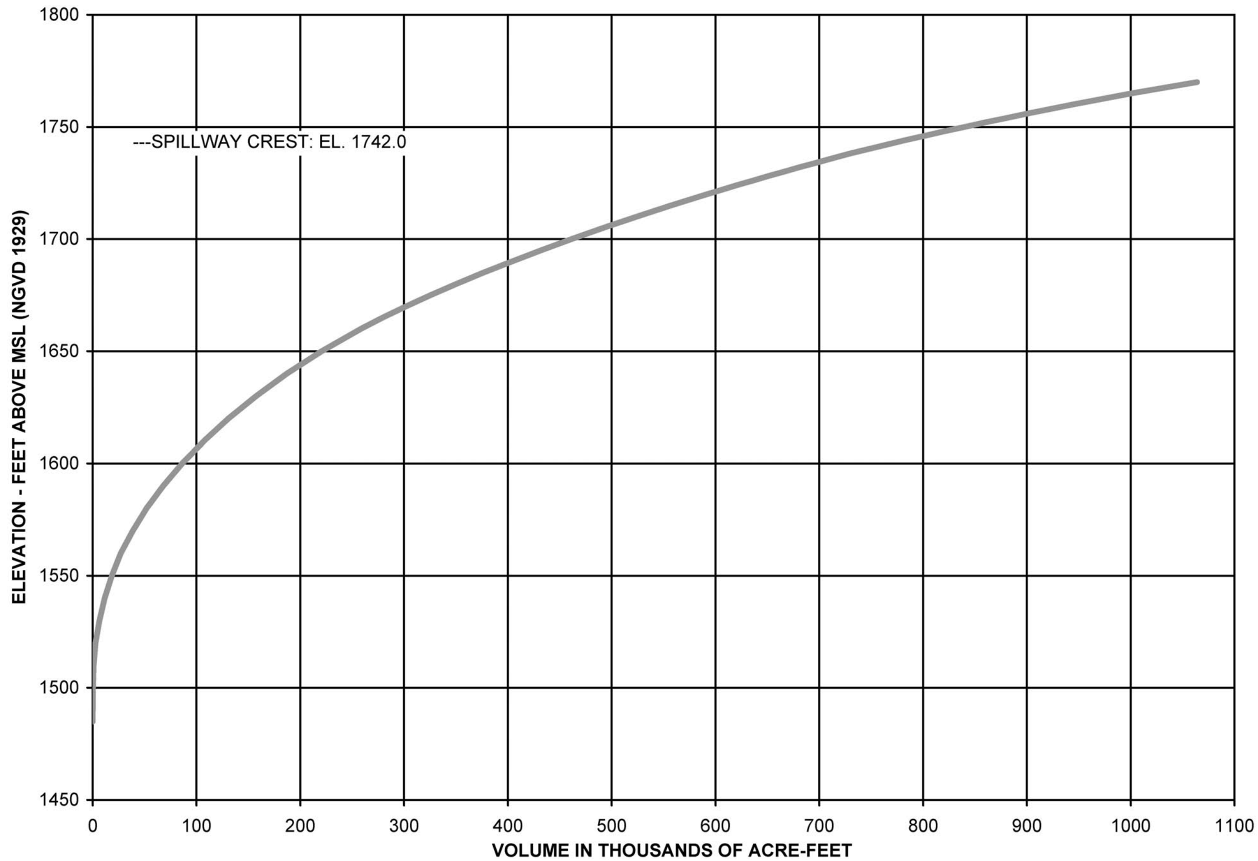
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Melton Hill Figure 2.4-4 (Sheet 10 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Melton Hill (Sheet 10 of 13)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Reservoir Elevation - Storage Relationship, Norris Figure 2.4-4 (Sheet 11 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, Norris (Sheet 11 of 13)

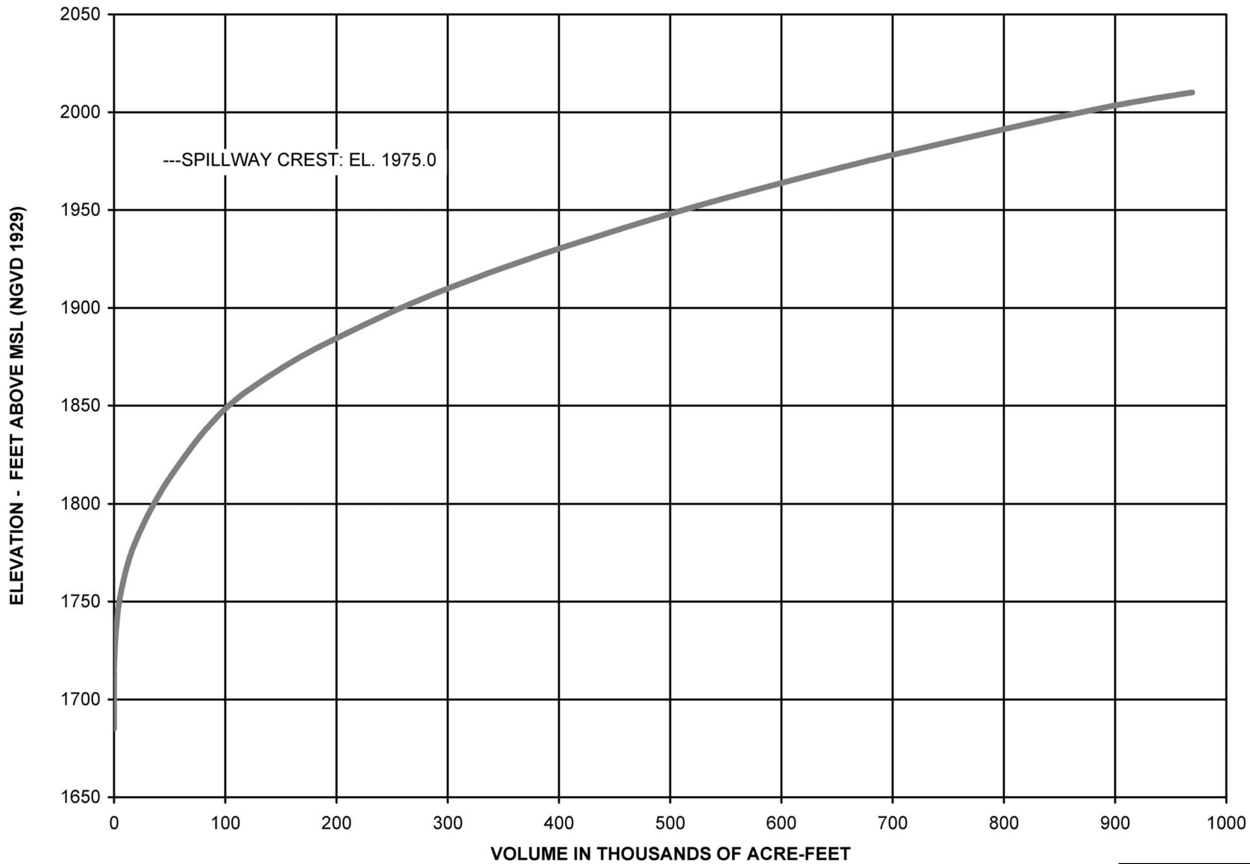


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Reservoir Elevation - Storage
Relationship, South Holston

Figure 2.4-4 (Sheet 12 of 13)

Figure 2.4-4 Reservoir Elevation - Storage Relationship, South Holston (Sheet 12 of 13)

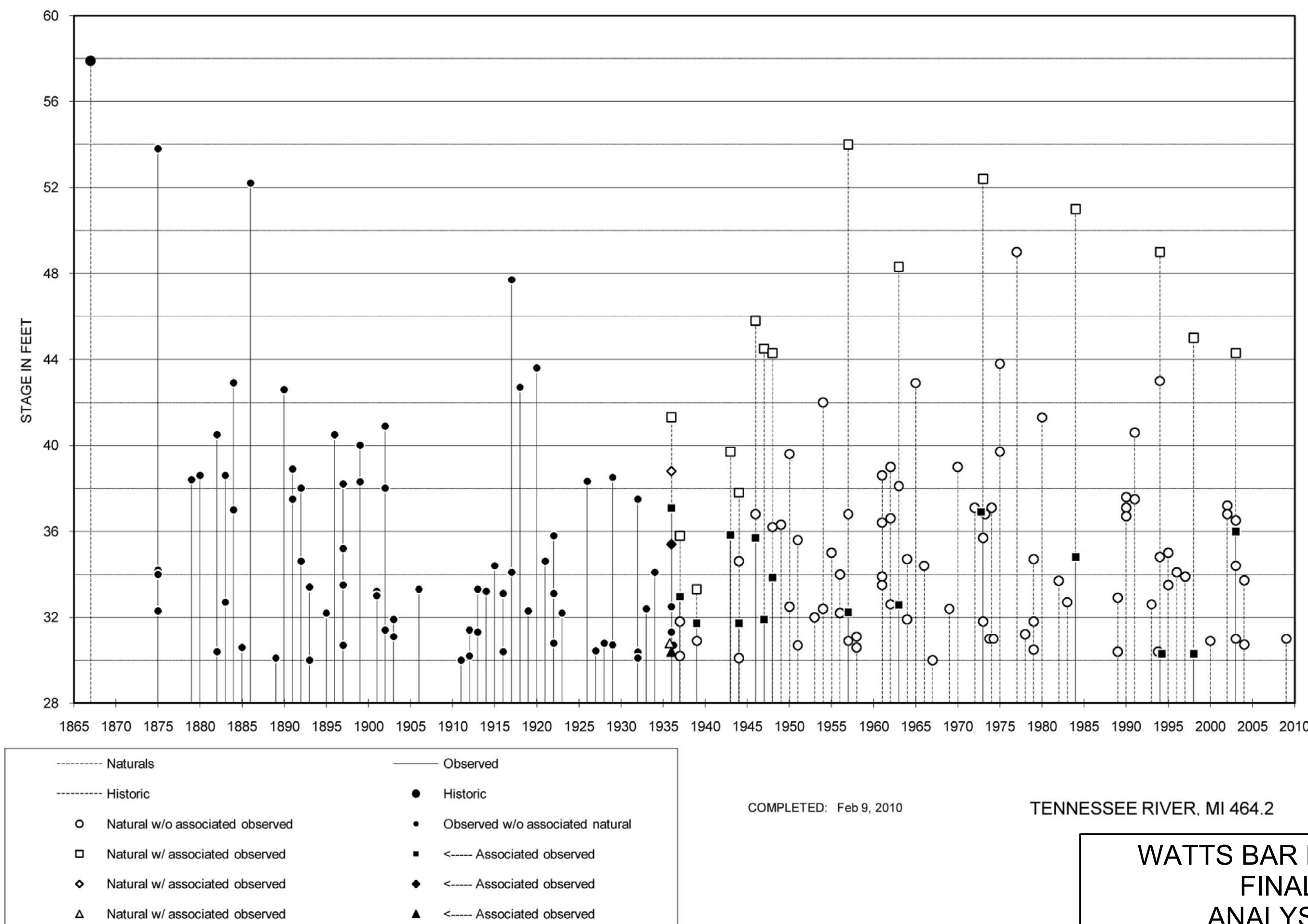


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Reservoir Elevation - Storage
Relationship, Watauga

Figure 2.4-4 (Sheet 13 of 13)

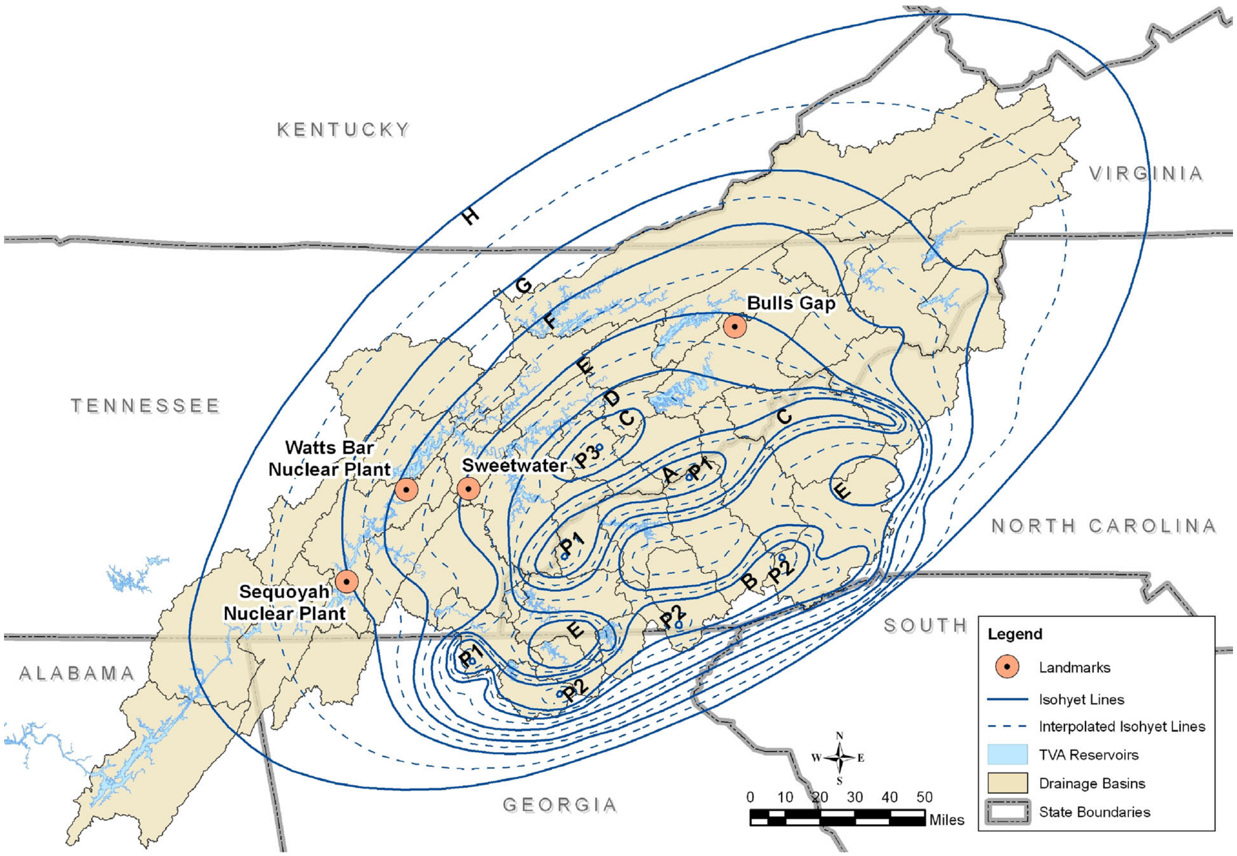
Figure 2.4-4 Reservoir Elevation - Storage Relationship, Watauga (Sheet 13 of 13)



WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Tennessee River Mile 464.2 -
Distribution of Floods at
Chattanooga, Tennessee
Figure 2.4-5

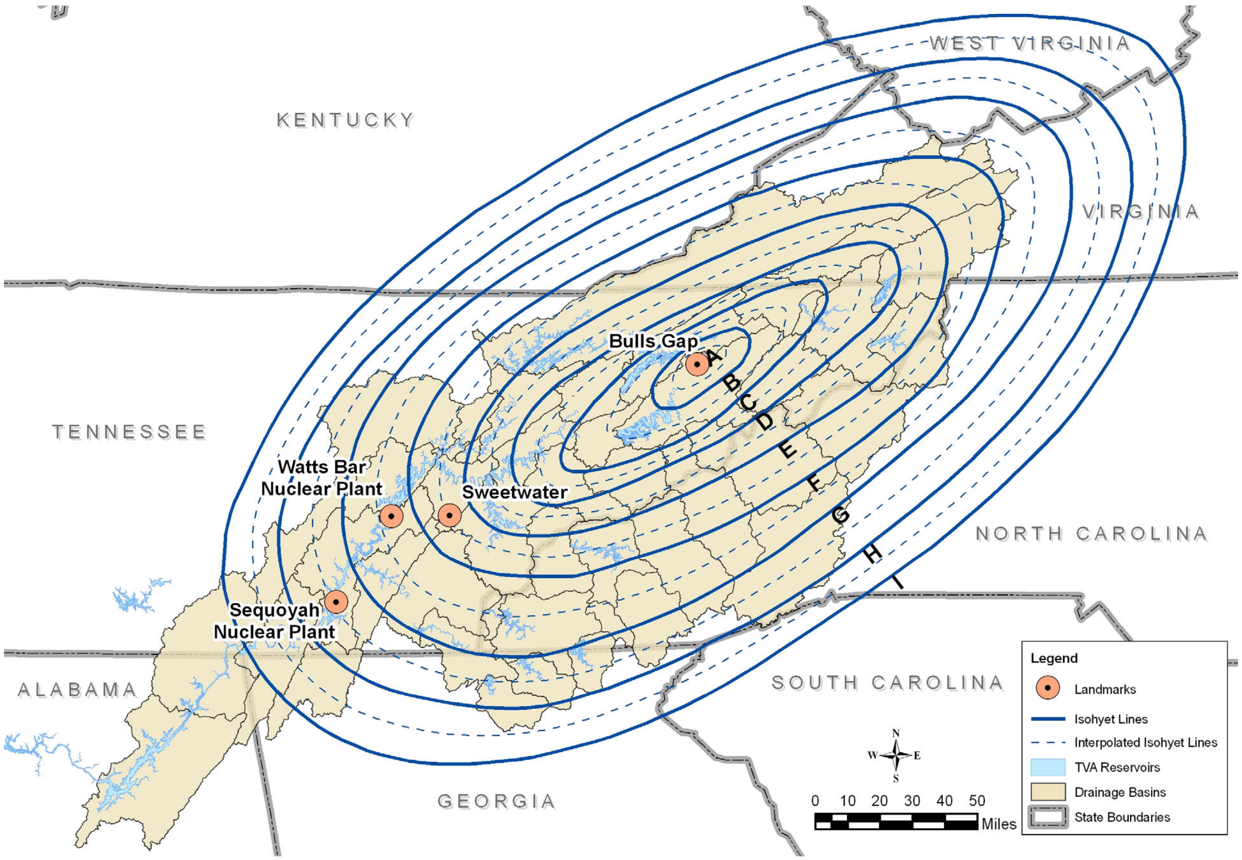
Figure 2.4-5 Tennessee River Mile 464.2 - Distribution of Floods at Chattanooga, Tennessee



WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Probable Maximum Precipitation
Isohyets for 21,400 Sq. Mi. Event,
Downstream Placement
Figure 2.4-6

Figure 2.4-6 Probable Maximum Precipitation Isohyets for 21,400 Sq. Mi. Event, Downstream Placement



WATTS BAR NUCLEAR PLANT
FINAL SAFETY
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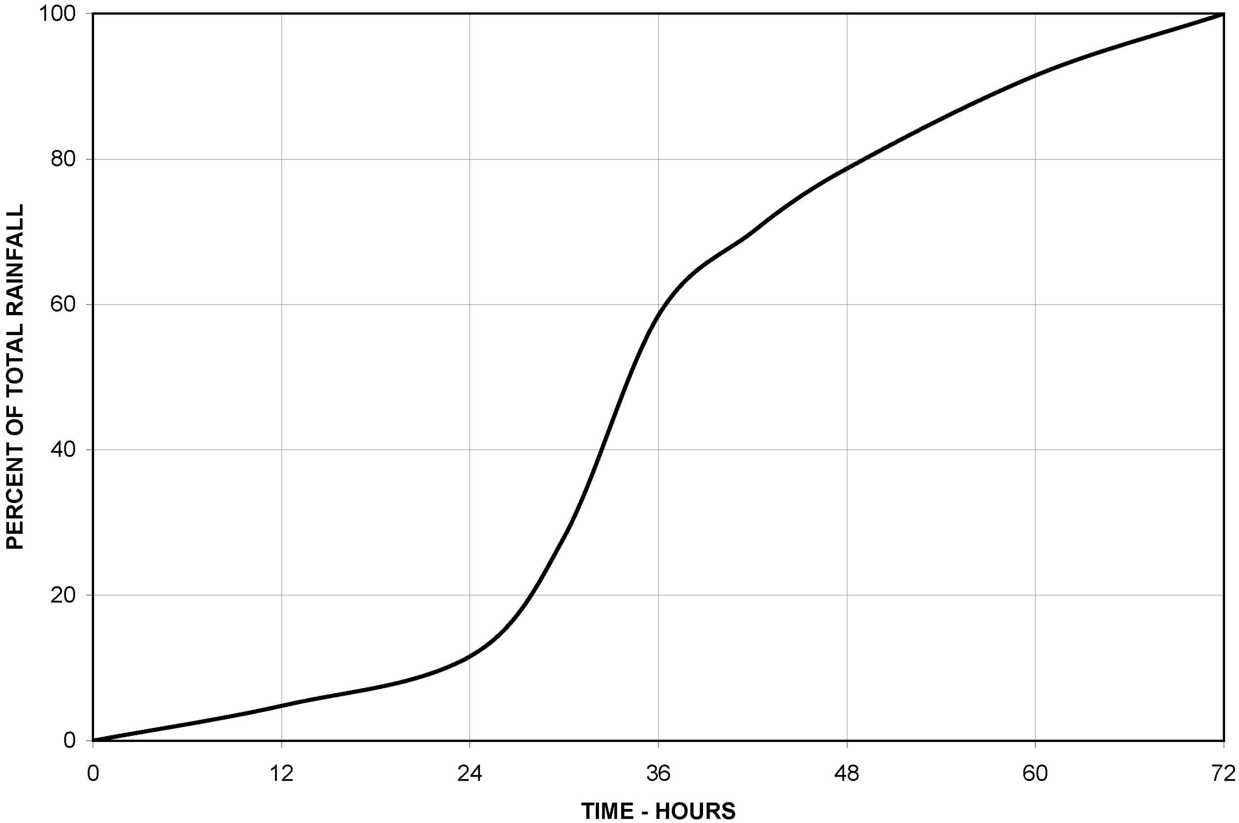
Probable Maximum Precipitation
Isohyets for 7980 Sq. Mi. Event,
Centered at Bulls Gap, TN
Figure 2.4-7

Figure 2.4-7 Probable Maximum Precipitation Isohyets for 7980 Sq. Mi. Event, Centered at Bulls Gap, TN

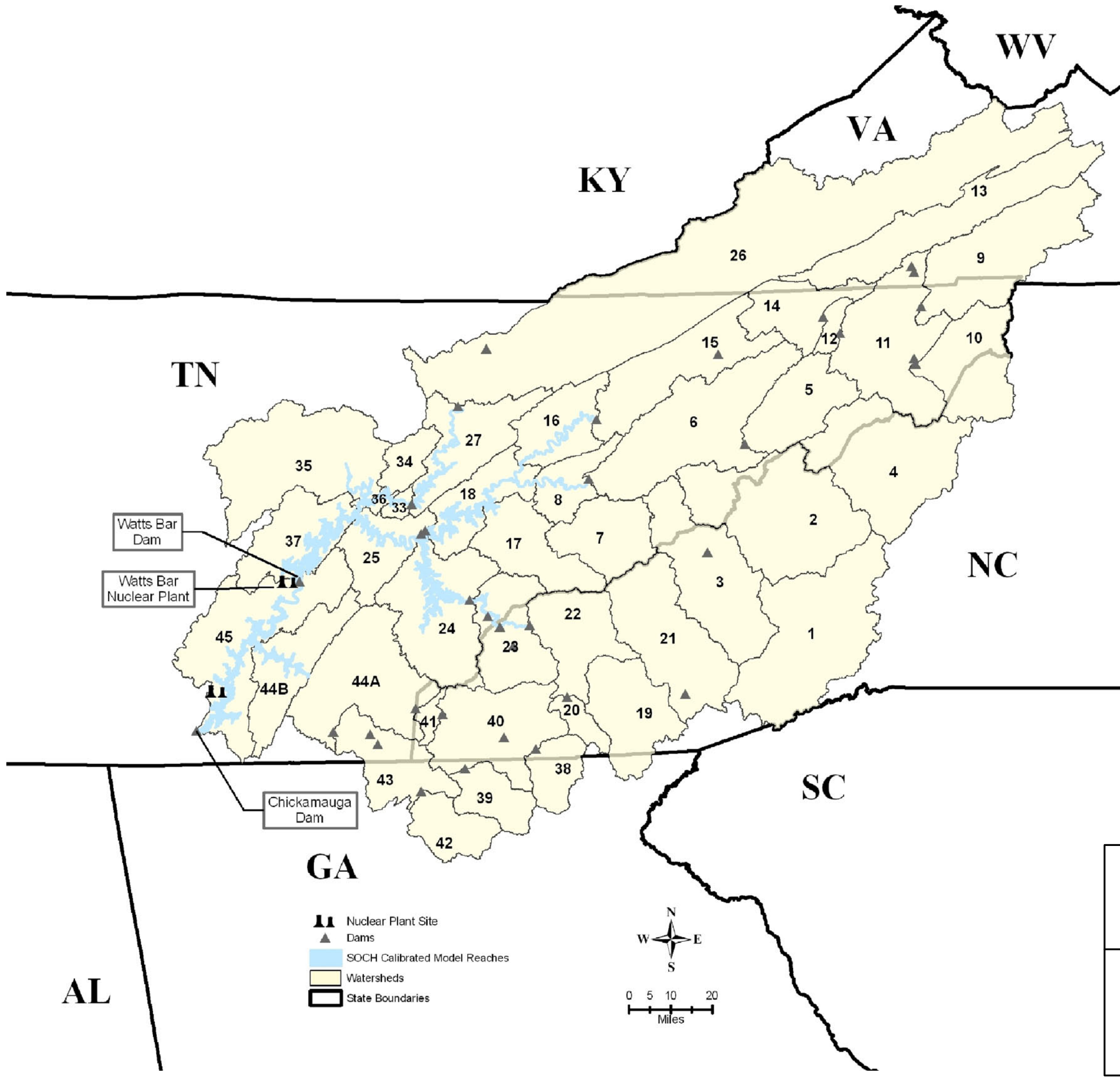


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Rainfall Time Distribution -
Typical Mass Curve

Figure 2.4-8

Figure 2.4-8 Rainfall Time Distribution - Typical Mass Curve

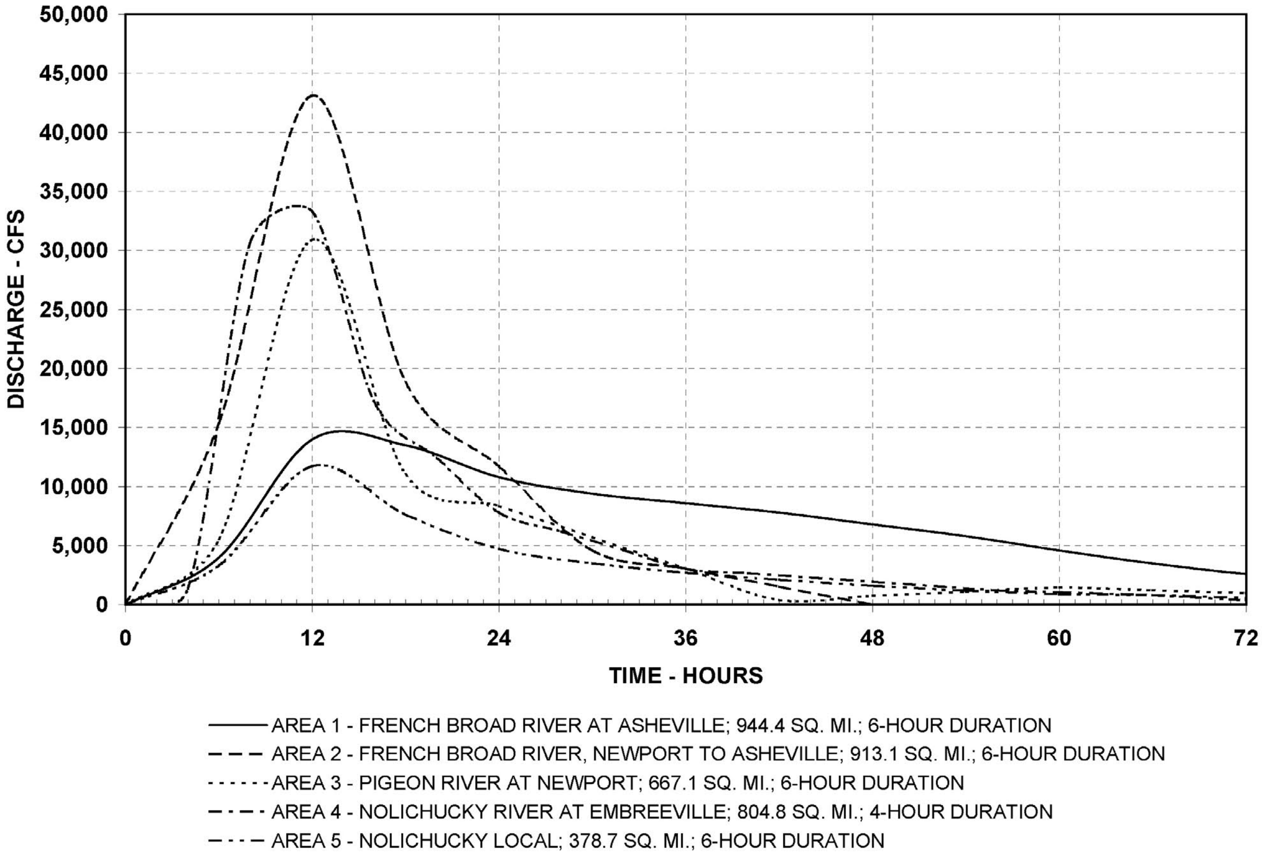


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Drainage Areas above Chickamauga Dam

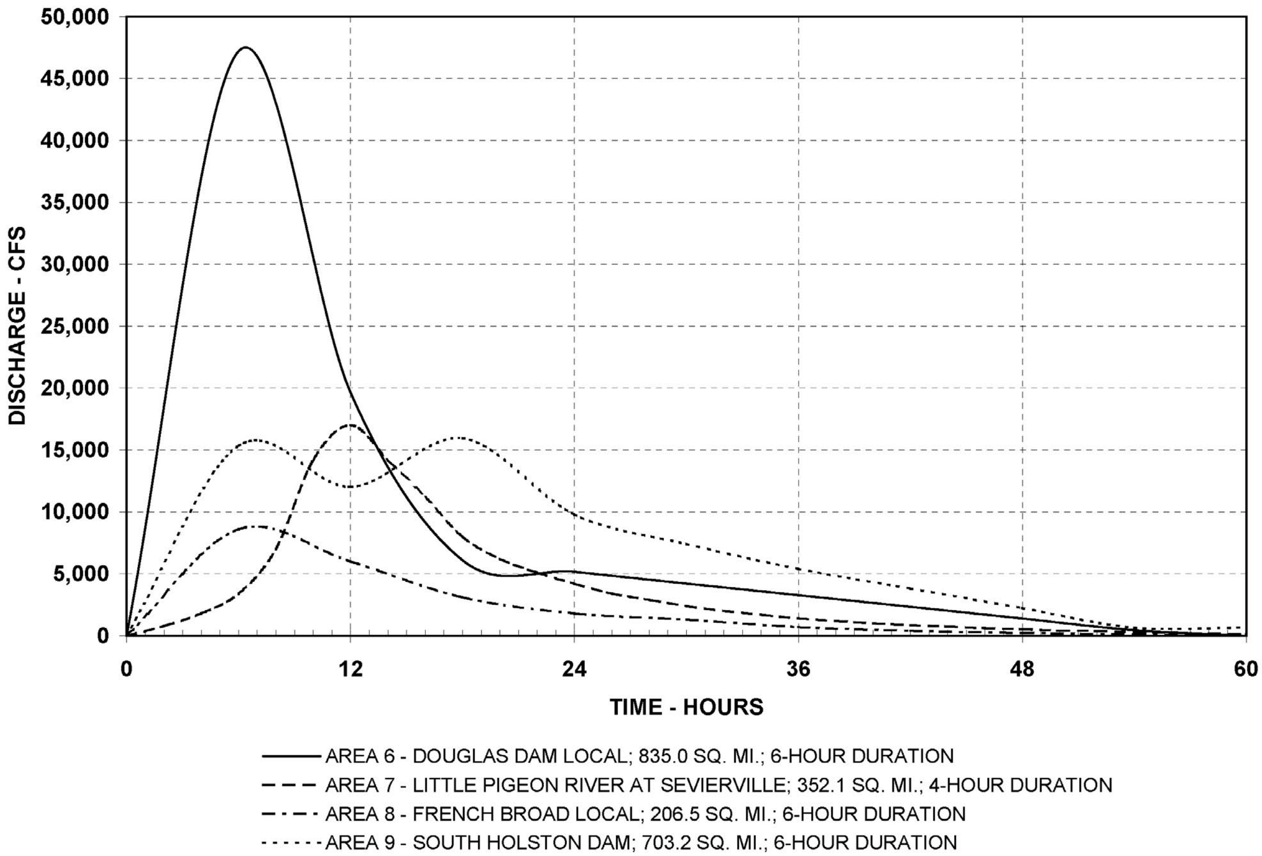
Figure 2.4-9

Figure 2.4-9 Drainage Areas above Chickamauga Dam



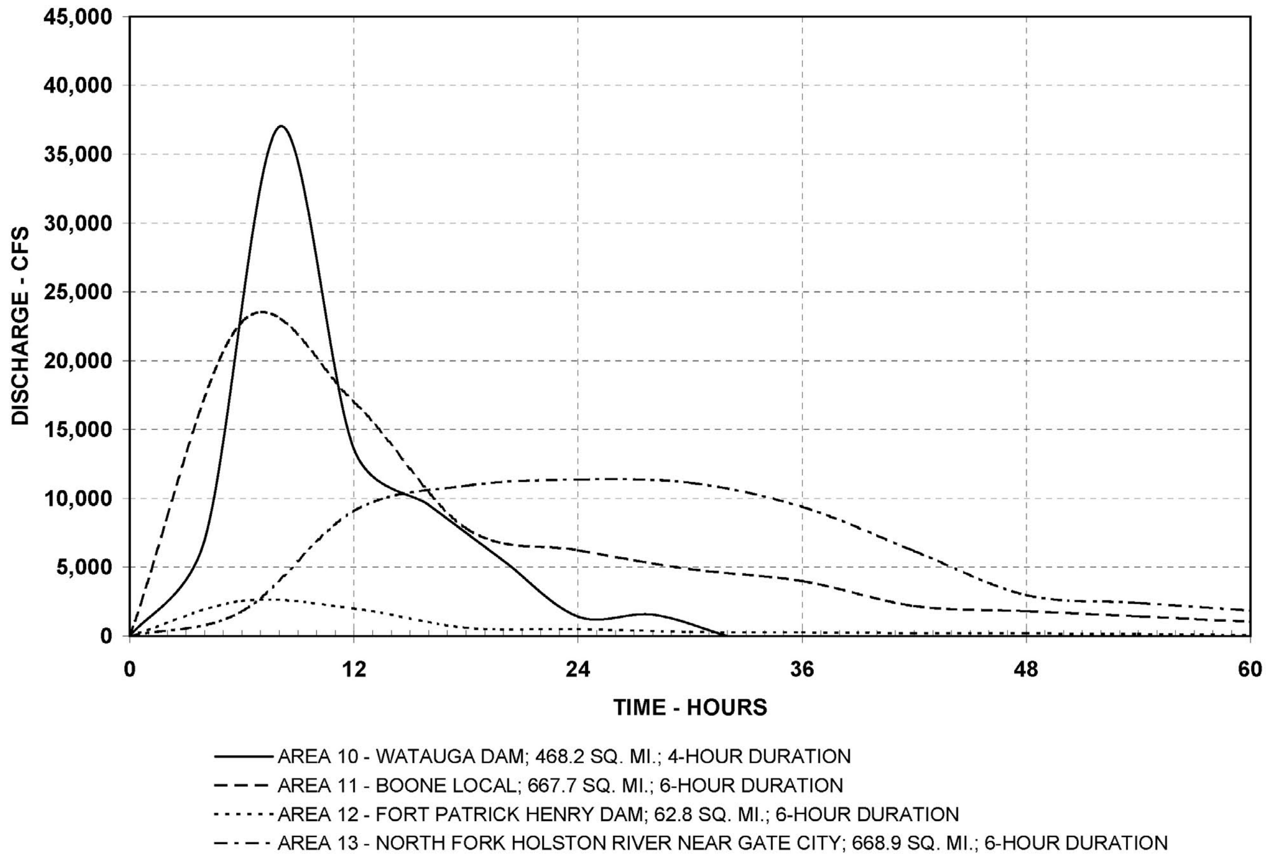
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 1-5 Figure 2.4-10 (Sheet 1 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 1–5 (Sheet 1 of 11)



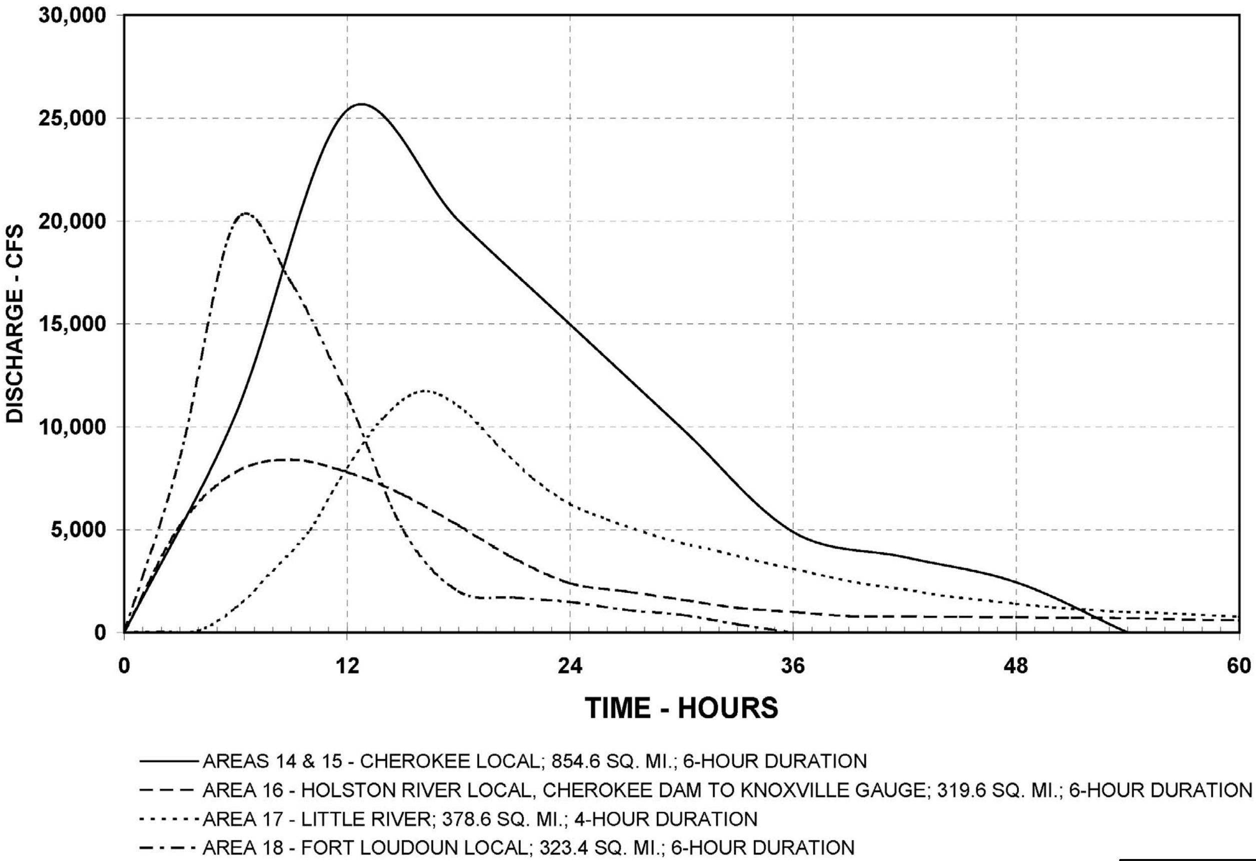
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 6-9 Figure 2.4-10 (Sheet 2 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 6–9 (Sheet 2 of 11)



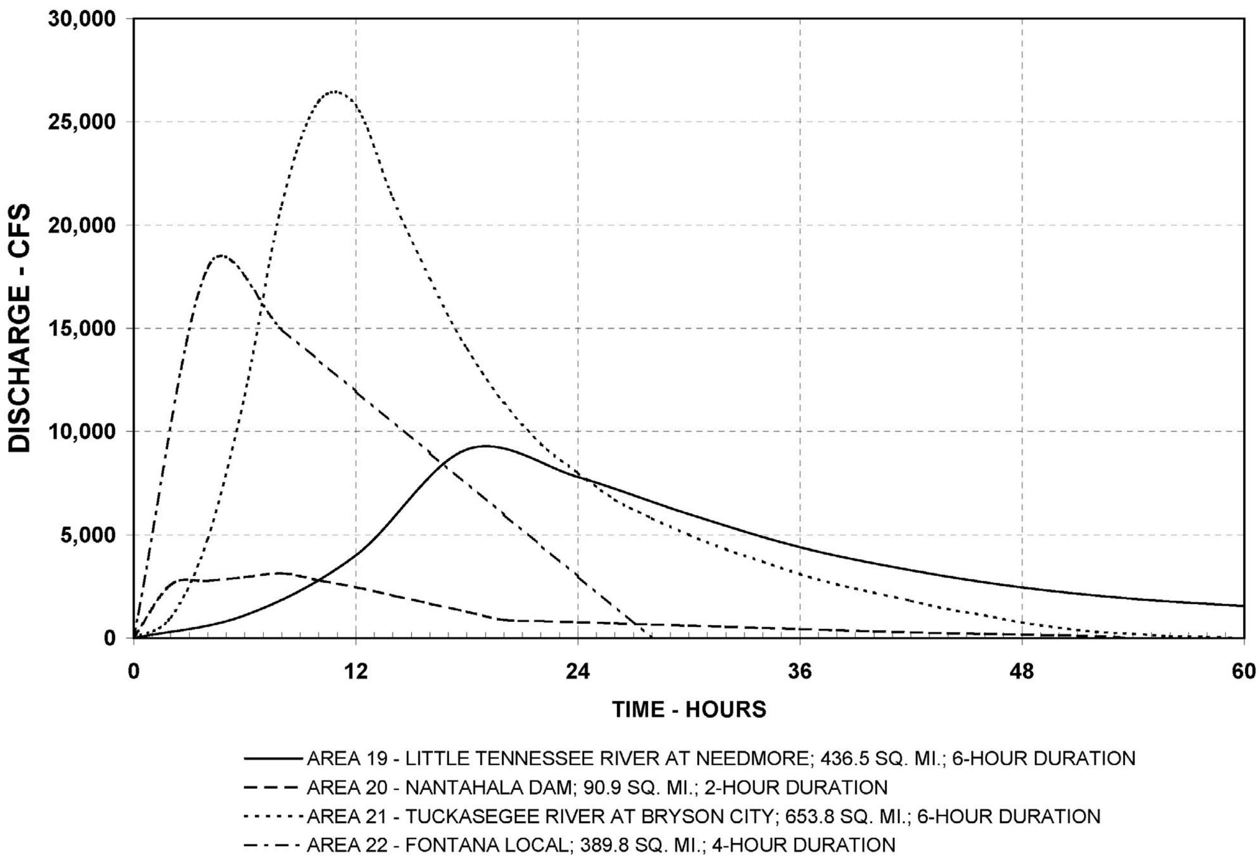
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 10-13 Figure 2.4-10 (Sheet 3 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 10–13 (Sheet 3 of 11)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 14-18 Figure 2.4-10 (Sheet 4 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 14–18 (Sheet 4 of 11)



WATTS BAR NUCLEAR PLANT
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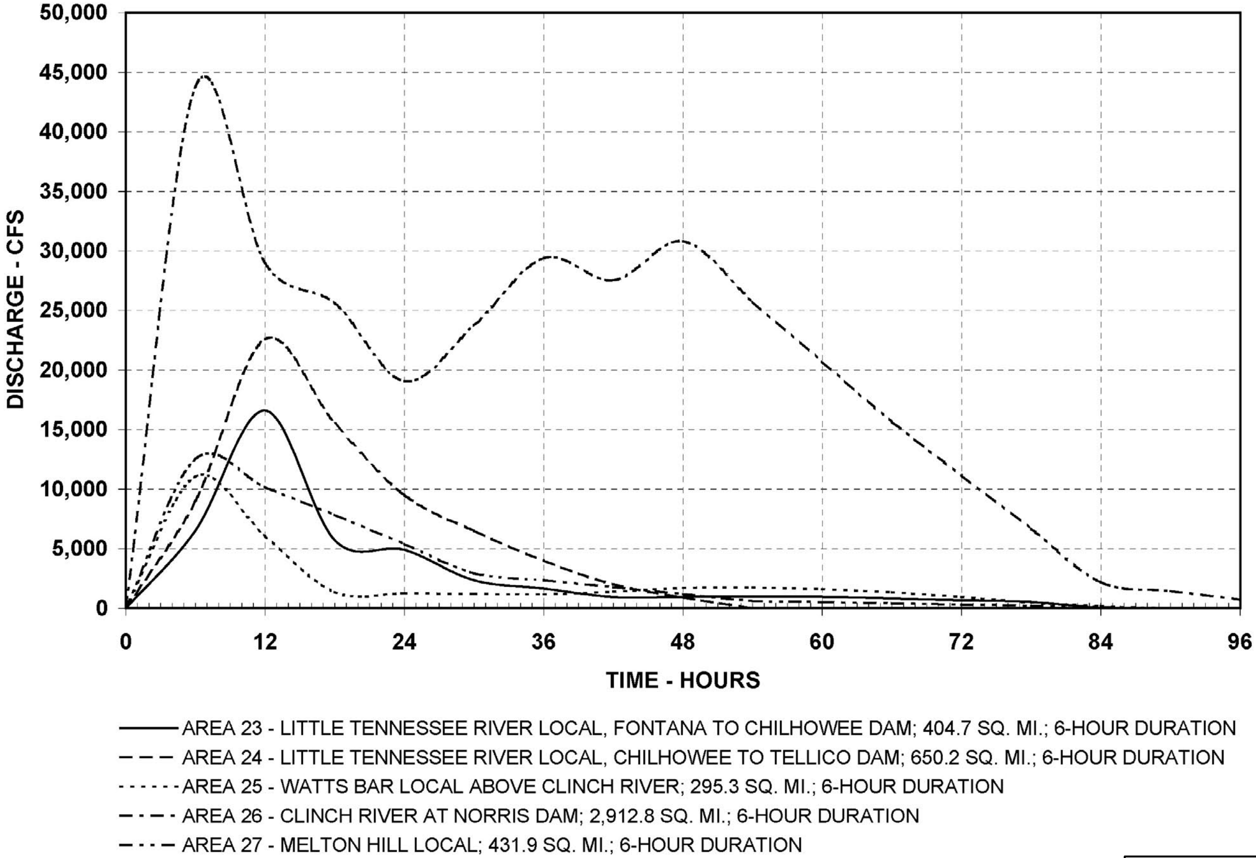
Unit Hydrographs, Areas 19-22

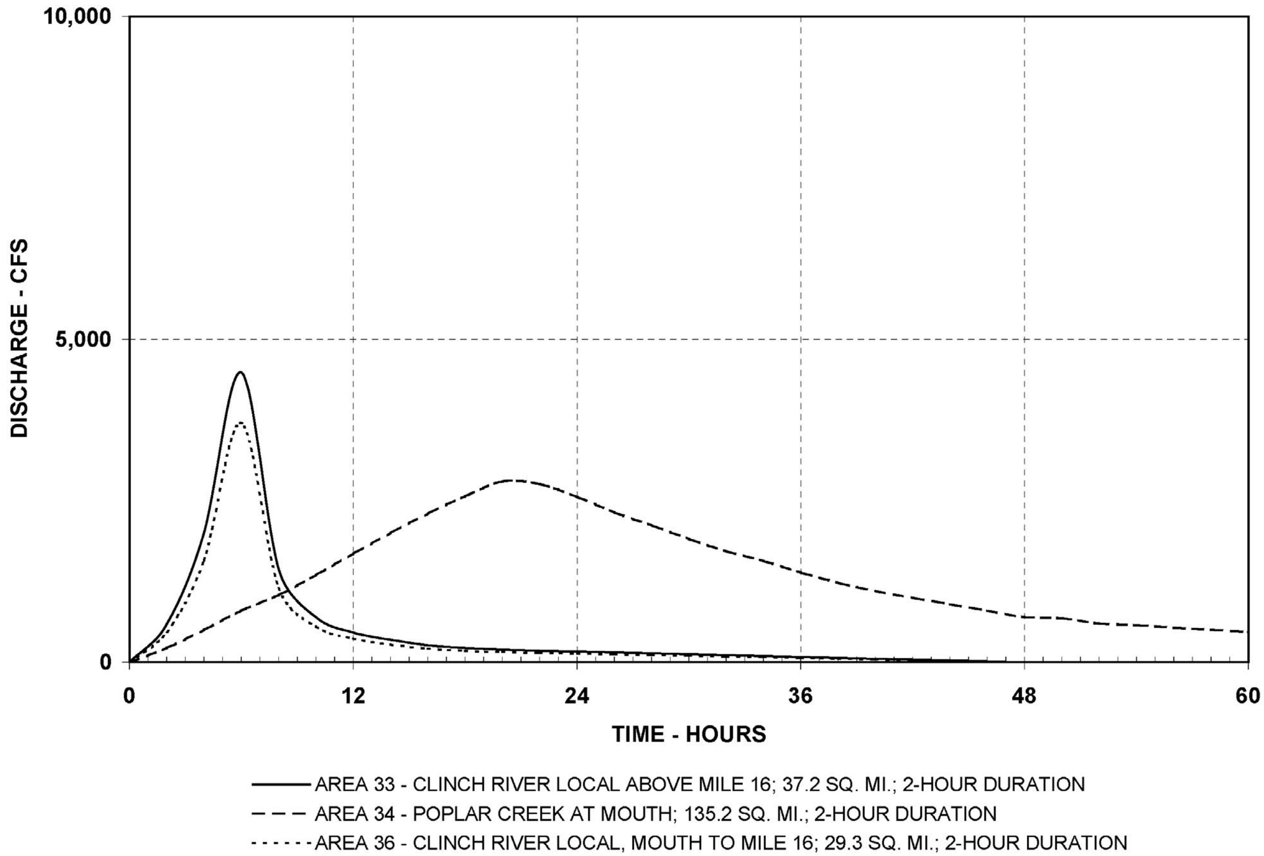
Figure 2.4-10 (Sheet 5 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 19–22 (Sheet 5 of 11)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 23-27 Figure 2.4-10 (Sheet 6 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 23–27 (Sheet 6 of 11)



WATTS BAR NUCLEAR PLANT
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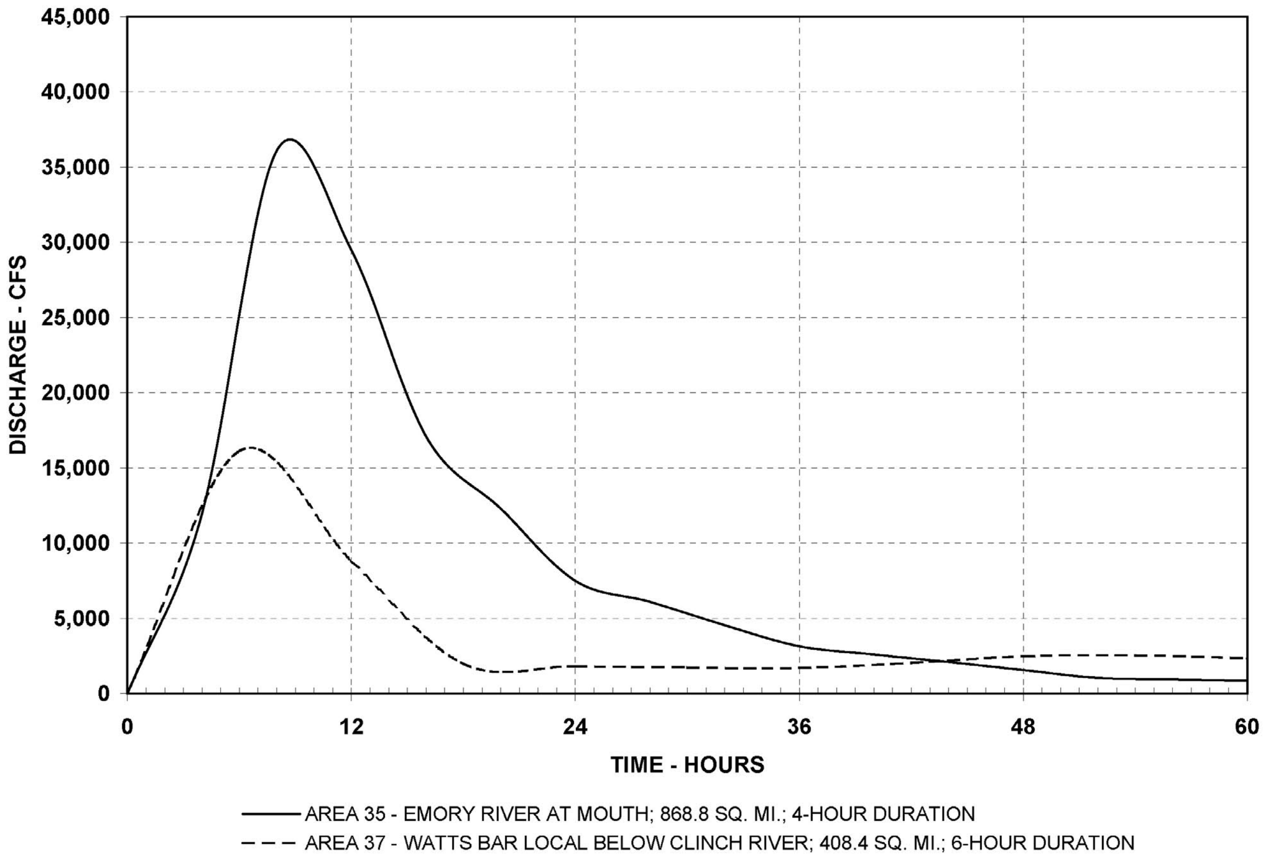
Unit Hydrographs, Areas 33, 34, 36

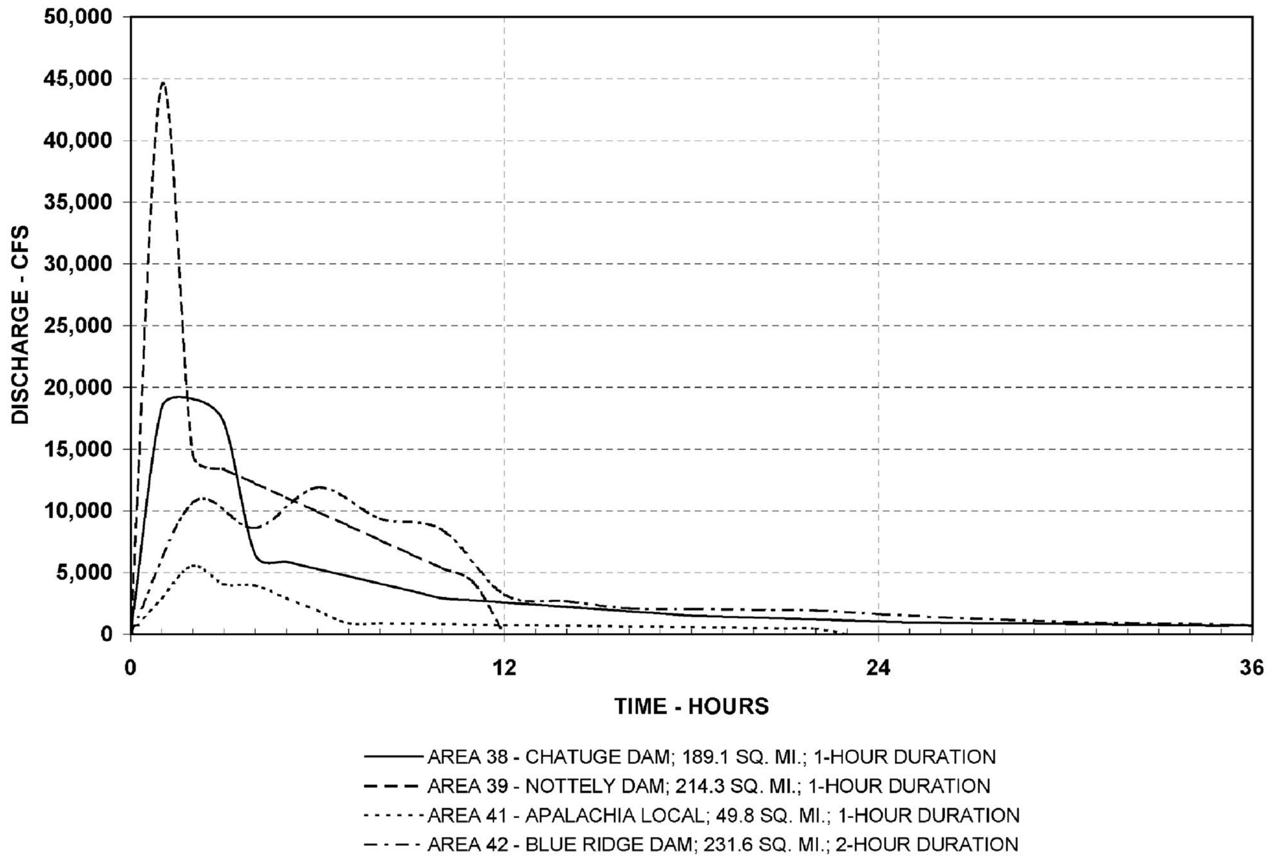
Figure 2.4-10 (Sheet 7 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 33, 34, 36 (Sheet 7 of 11)



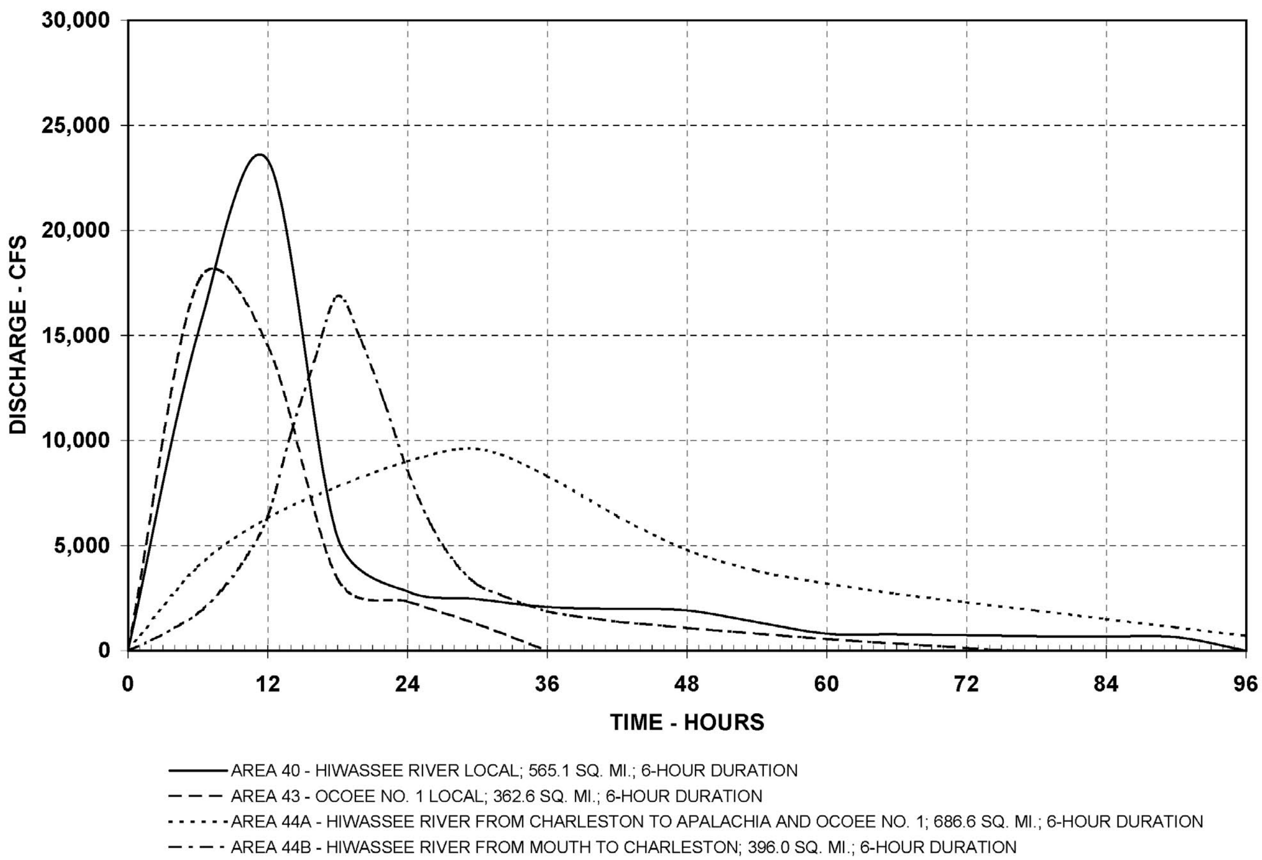
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 35, 37 Figure 2.4-10 (Sheet 8 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 35, 37 (Sheet 8 of 11)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unit Hydrographs, Areas 38, 39, 41, 42 Figure 2.4-10 (Sheet 9 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 38, 39, 41, 42 (Sheet 9 of 11)



WATTS BAR NUCLEAR PLANT
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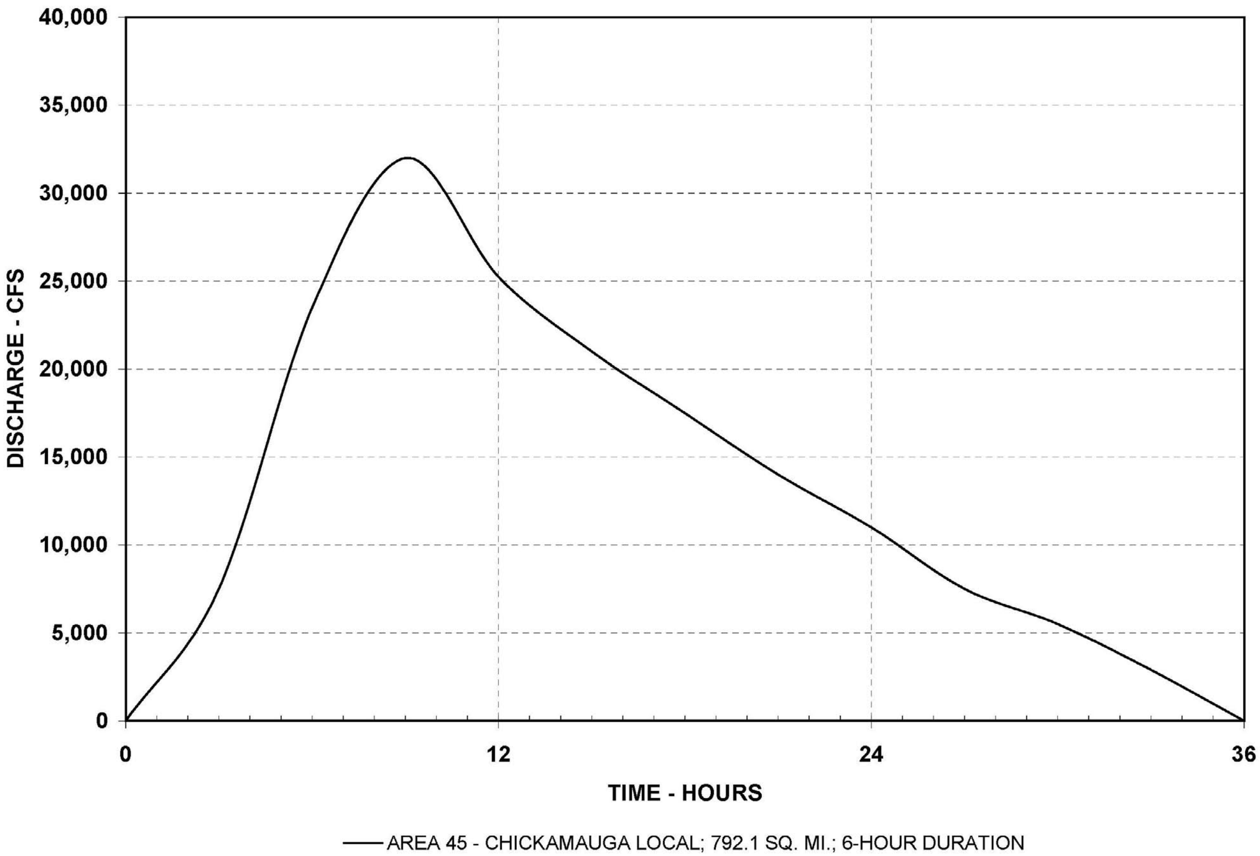
Unit Hydrographs, Areas 40, 43, 44A, 44B

Figure 2.4-10 (Sheet 10 of 11)

Figure 2.4-10 Unit Hydrographs, Areas 40, 43, 44A, 44B (Sheet 10 of 11)

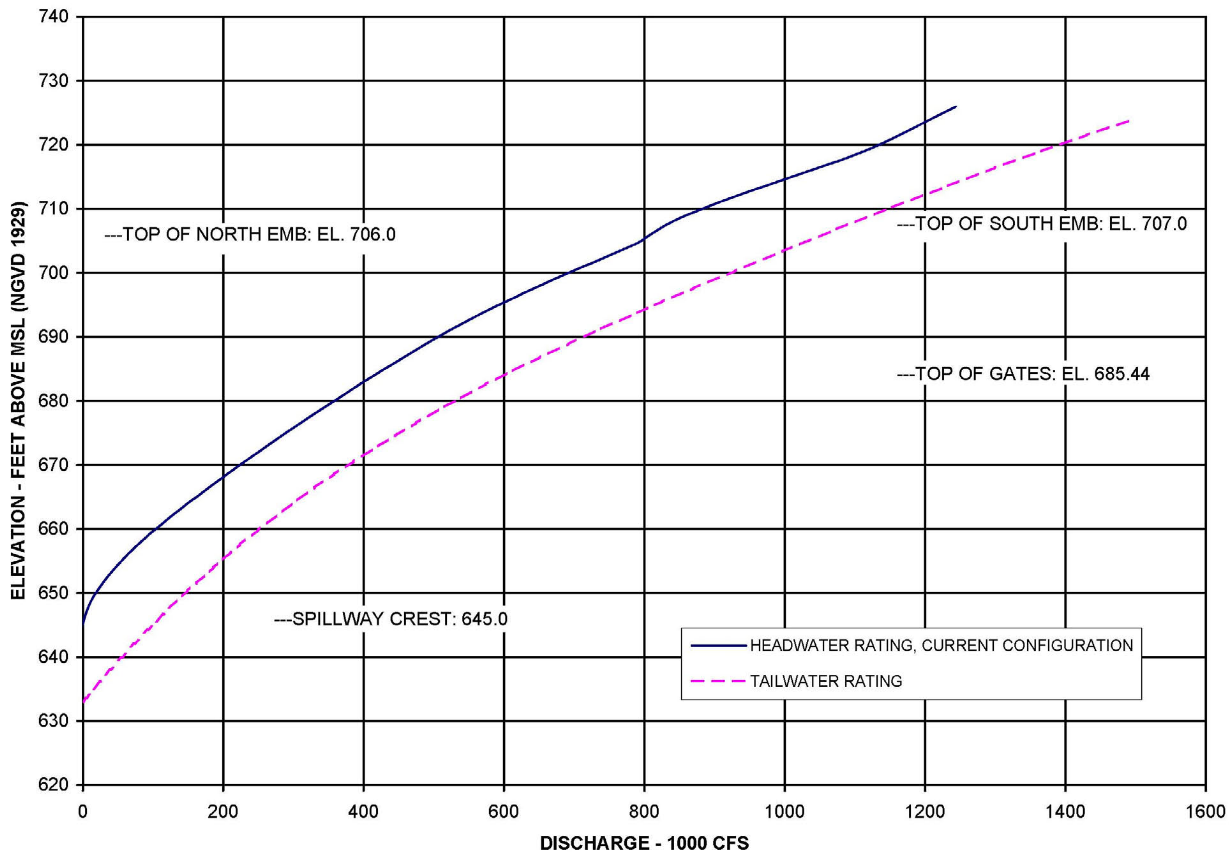


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Unit Hydrographs, Area 45

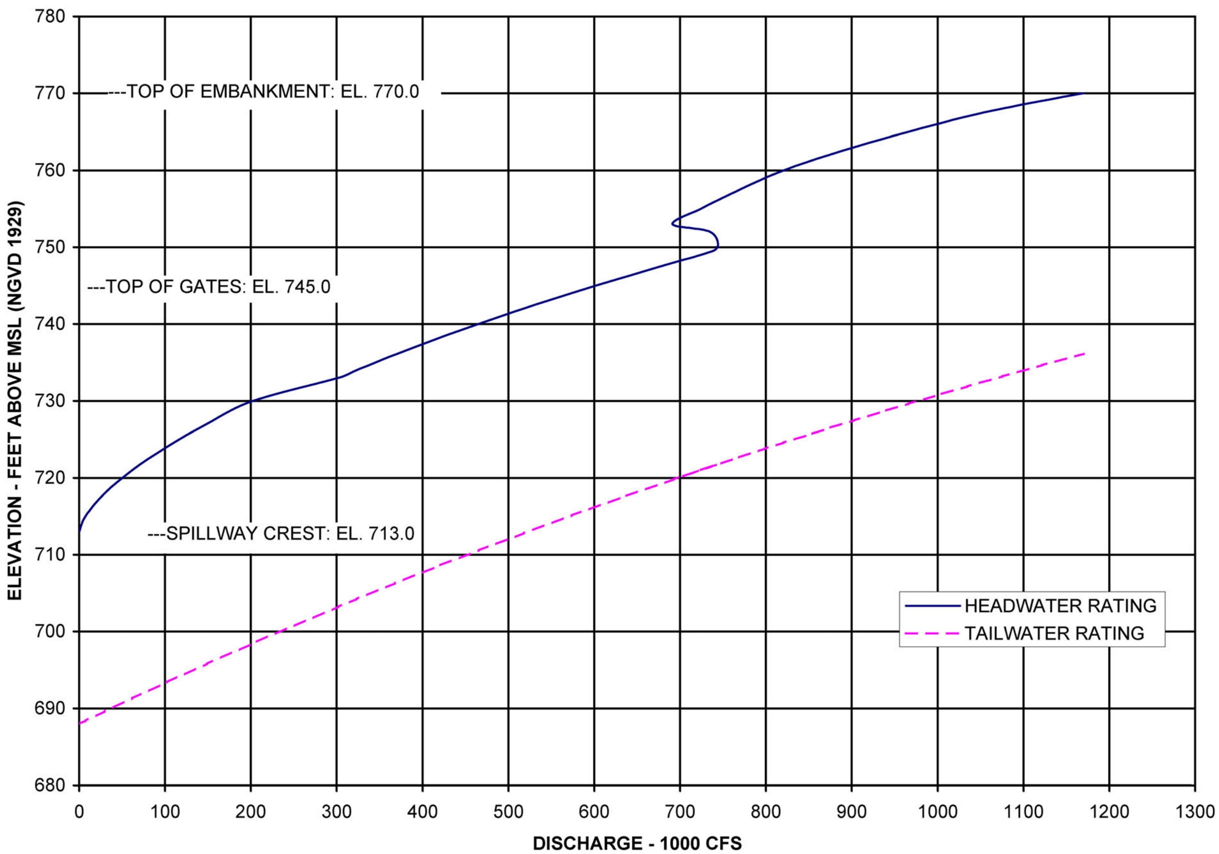
Figure 2.4-10 (Sheet 11 of 11)

Figure 2.4-10 Unit Hydrographs, Area 45 (Sheet 11 of 11)



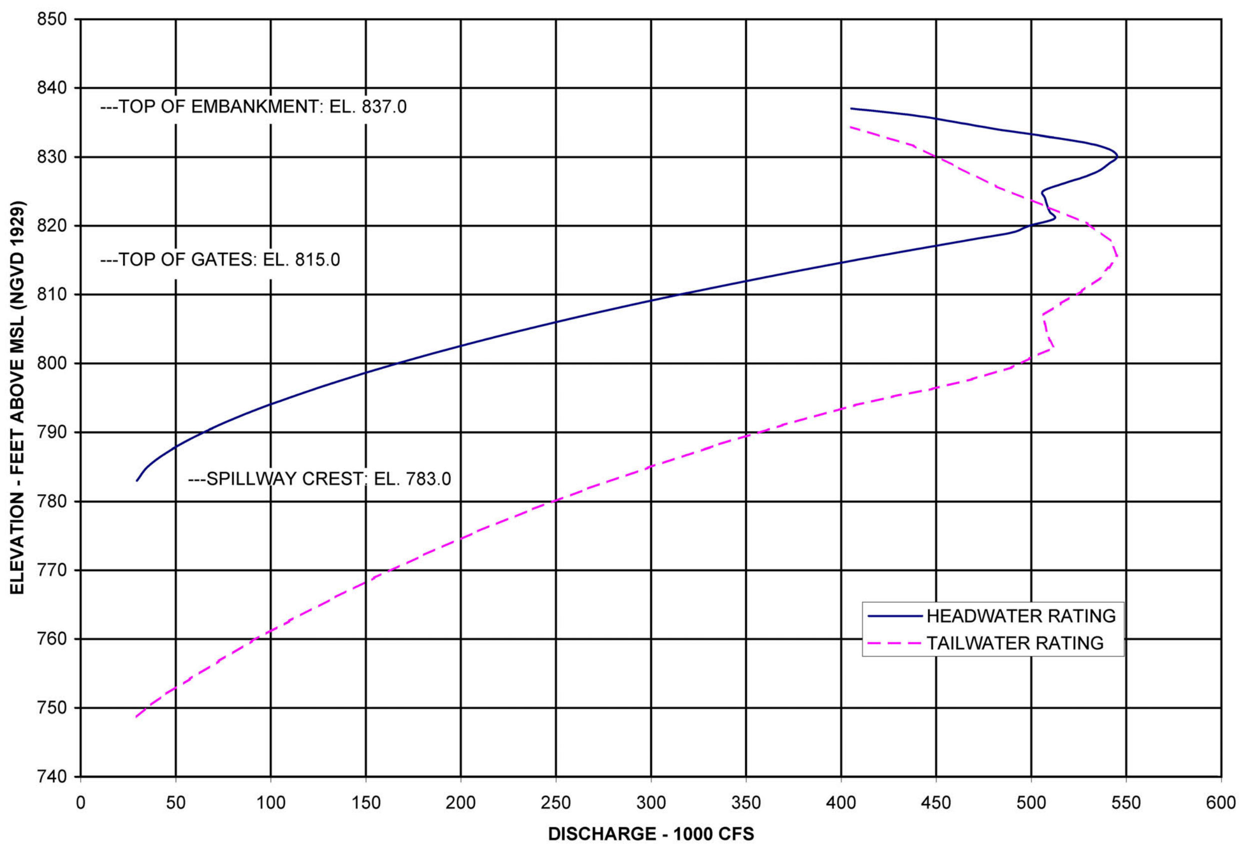
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Chickamauga Dam Figure 2.4-11 (Sheet 1 of 13)

Figure 2.4-11 Discharge Rating Curve, Chickamauga Dam (Sheet 1 of 13)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Watts Bar Dam Figure 2.4-11 (Sheet 2 of 13)

Figure 2.4-11 Discharge Rating Curve, Watts Bar Dam (Sheet 2 of 13)

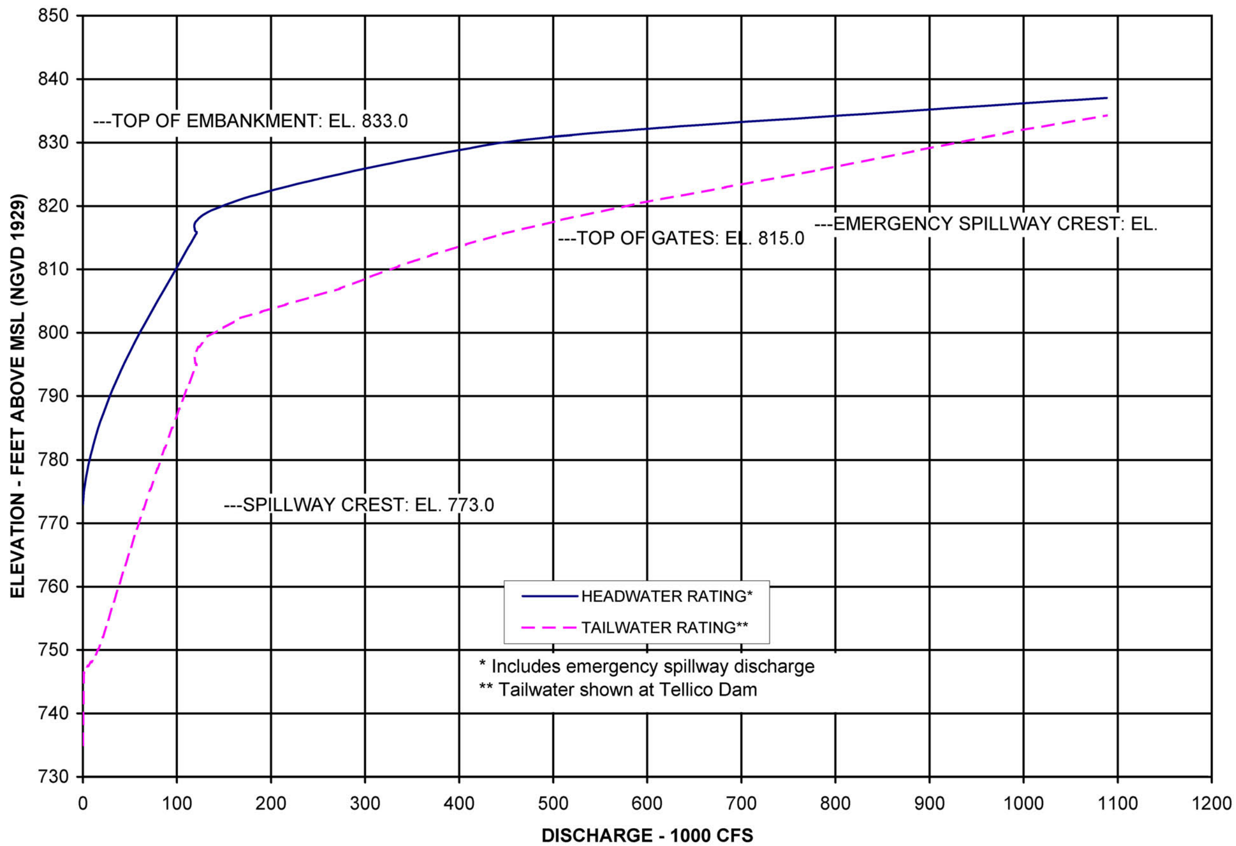


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Discharge Rating Curve,
Fort Loudoun Dam

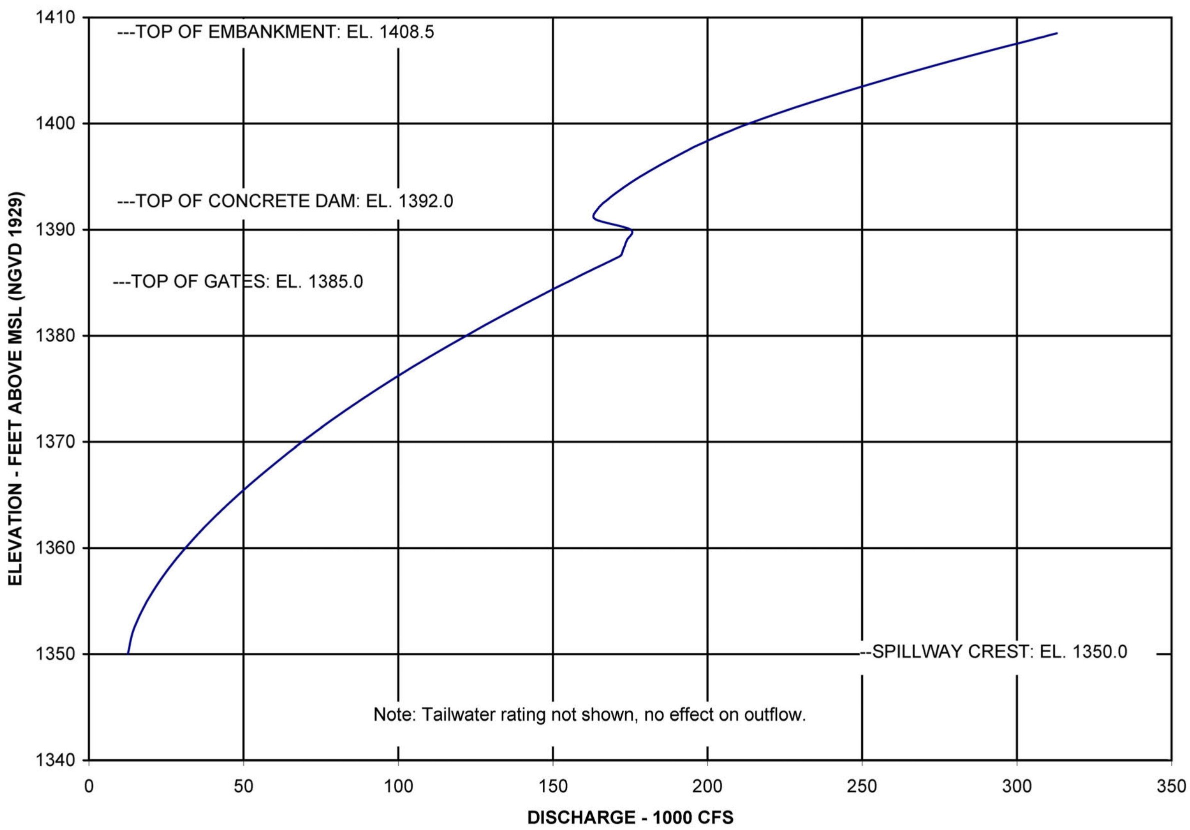
Figure 2.4-11 (Sheet 3 of 13)

Figure 2.4-11 Discharge Rating Curve, Fort Loudoun Dam (Sheet 3 of 13)



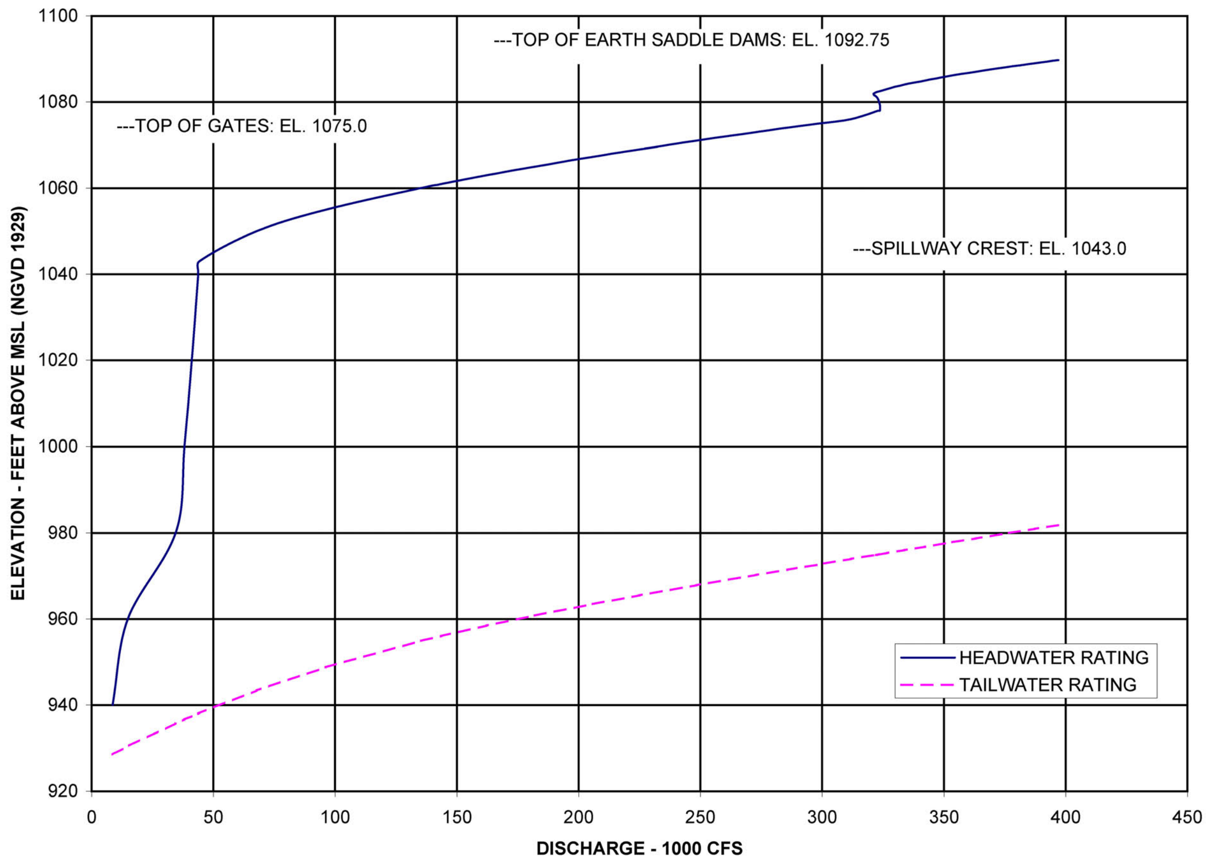
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Tellico Dam Figure 2.4-11 (Sheet 4 of 13)

Figure 2.4-11 Discharge Rating Curve, Tellico Dam (Sheet 4 of 13)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Boone Dam Figure 2.4-11 (Sheet 5 of 13)

Figure 2.4-11 Discharge Rating Curve, Boone Dam (Sheet 5 of 13)

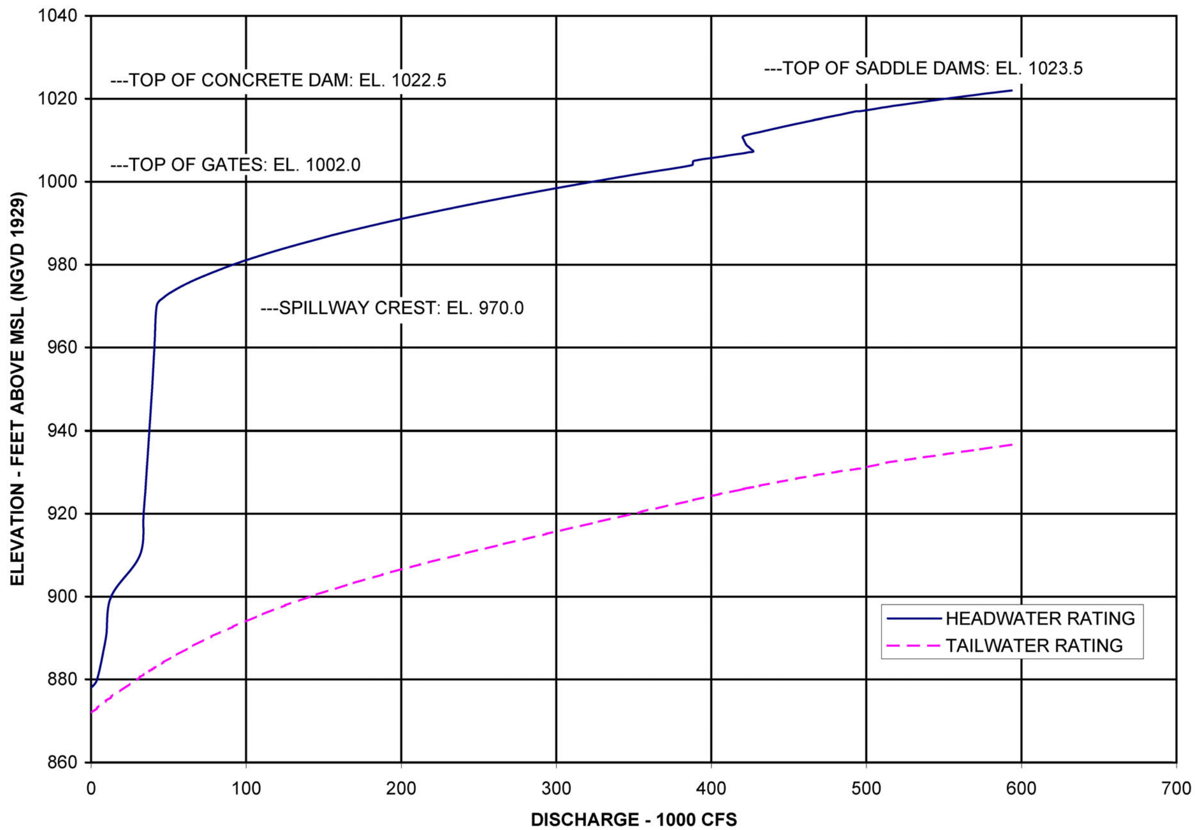


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Discharge Rating Curve, Cherokee Dam

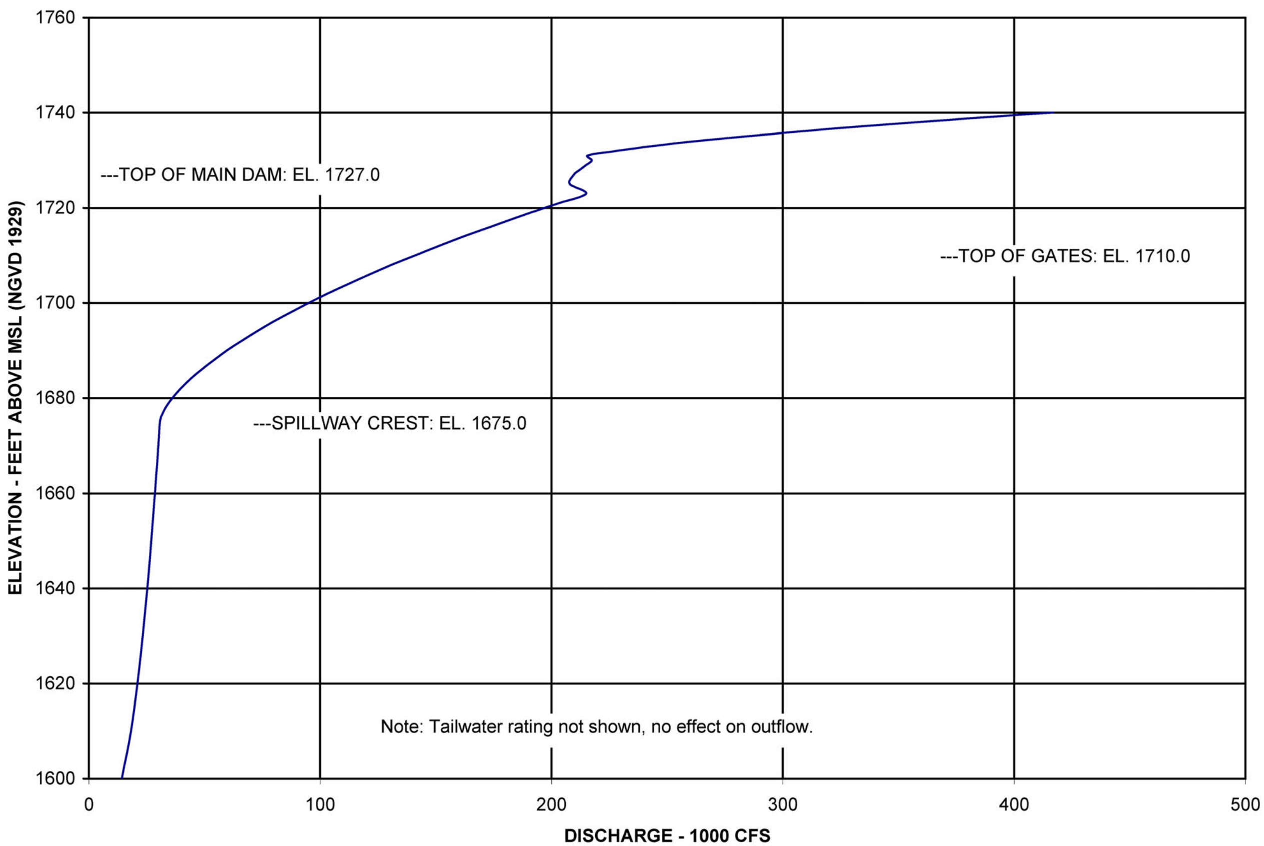
Figure 2.4-11 (Sheet 6 of 13)

Figure 2.4-11 Discharge Rating Curve, Cherokee Dam (Sheet 6 of 13)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Douglas Dam Figure 2.4-11 (Sheet 7 of 13)

Figure 2.4-11 Discharge Rating Curve, Douglas Dam (Sheet 7 of 13)



WATTS BAR NUCLEAR PLANT
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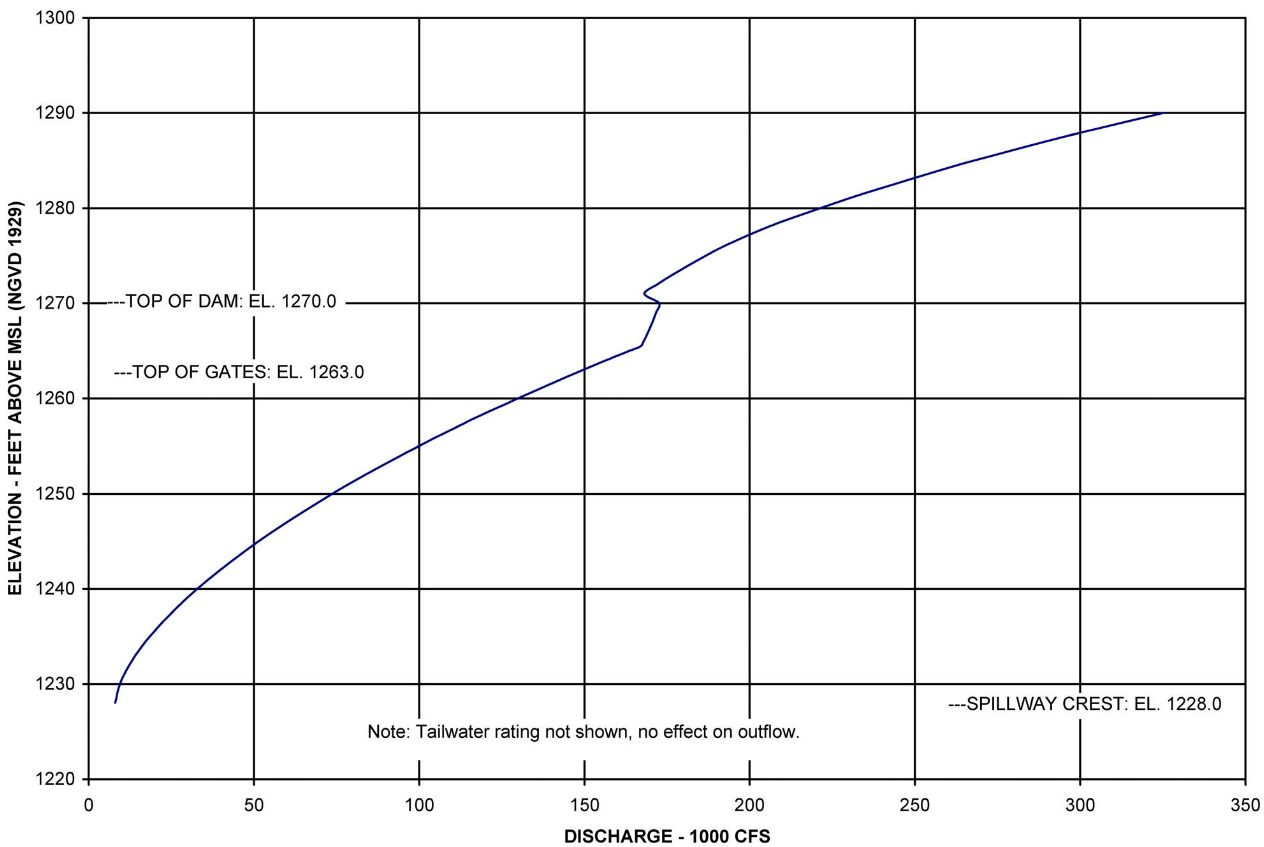
ANALYSIS REPORT

Discharge Rating Curve, Fontana Dam

Figure 2.4-11 (Sheet 8 of 13)

Figure 2.4-11 Discharge Rating Curve, Fontana Dam (Sheet 8 of 13)



WATTS BAR NUCLEAR PLANT
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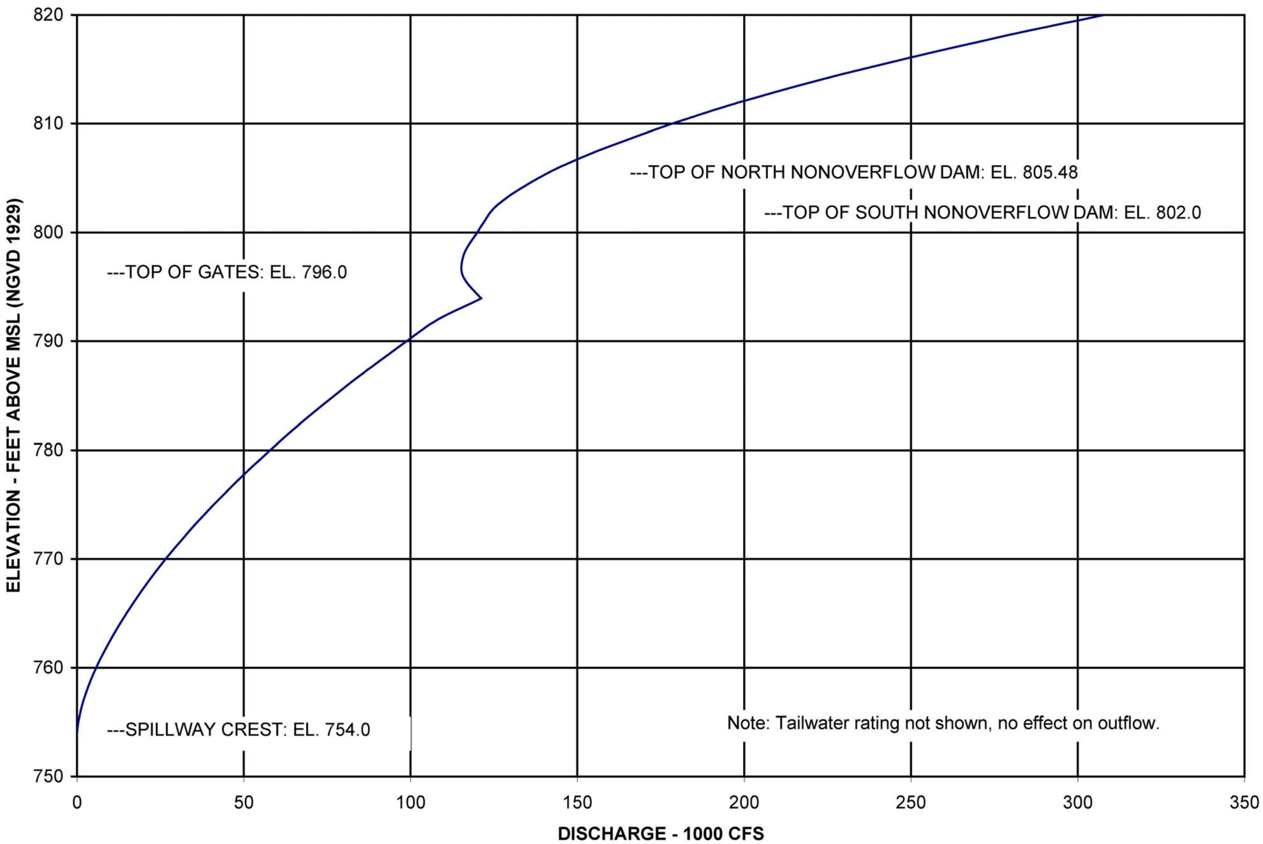
Discharge Rating Curve,
Fort Patrick Henry Dam

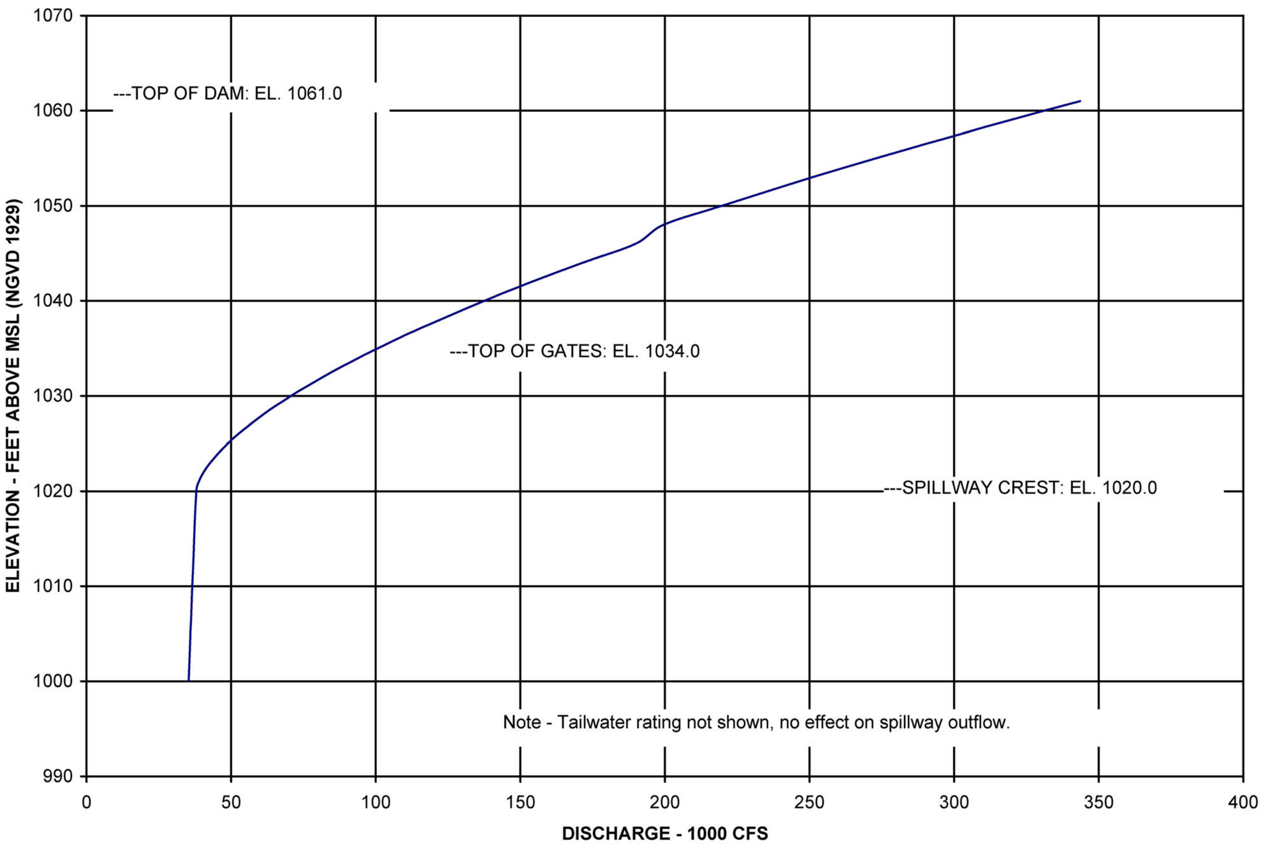
Figure 2.4-11 (Sheet 9 of 13)

Figure 2.4-11 Discharge Rating Curve, Fort Patrick Henry Dam (Sheet 9 of 13)



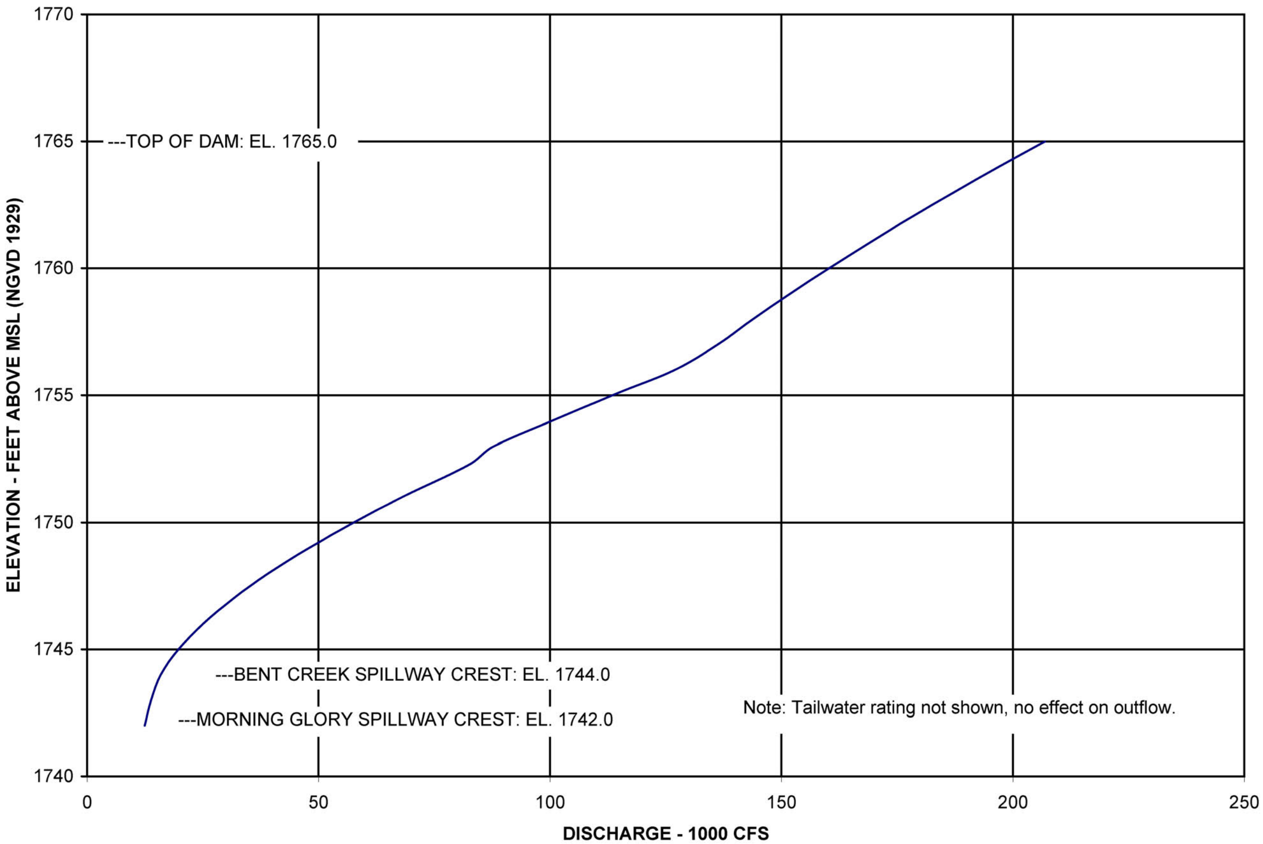
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Melton Hill Dam Figure 2.4-11 (Sheet 10 of 13)

Figure 2.4-11 Discharge Rating Curve, Melton Hill Dam (Sheet 10 of 13)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Norris Dam Figure 2.4-11 (Sheet 11 of 13)

Figure 2.4-11 Discharge Rating Curve, Norris Dam (Sheet 11 of 13)

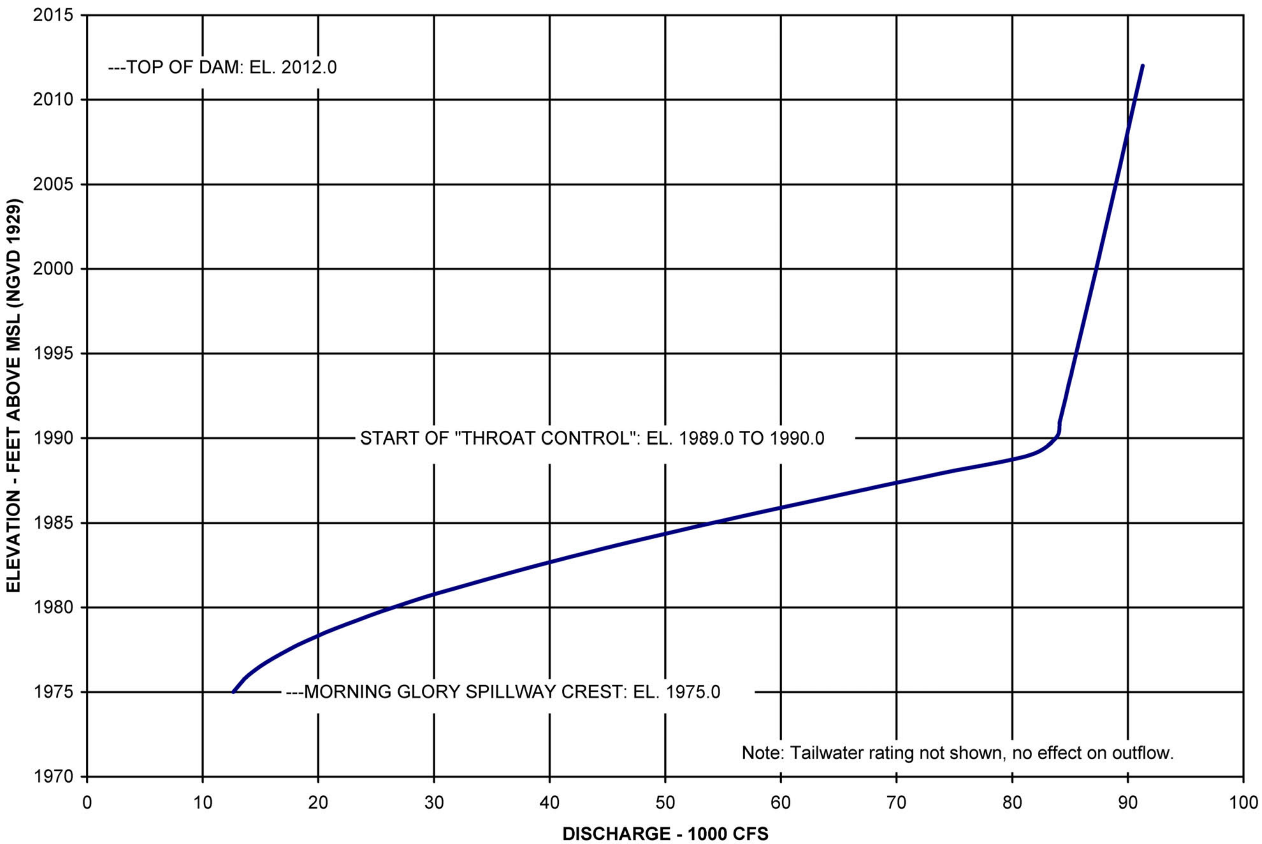


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Discharge Rating Curve,
South Holston Dam

Figure 2.4-11 (Sheet 12 of 13)

Figure 2.4-11 Discharge Rating Curve, South Holston Dam (Sheet 12 of 13)

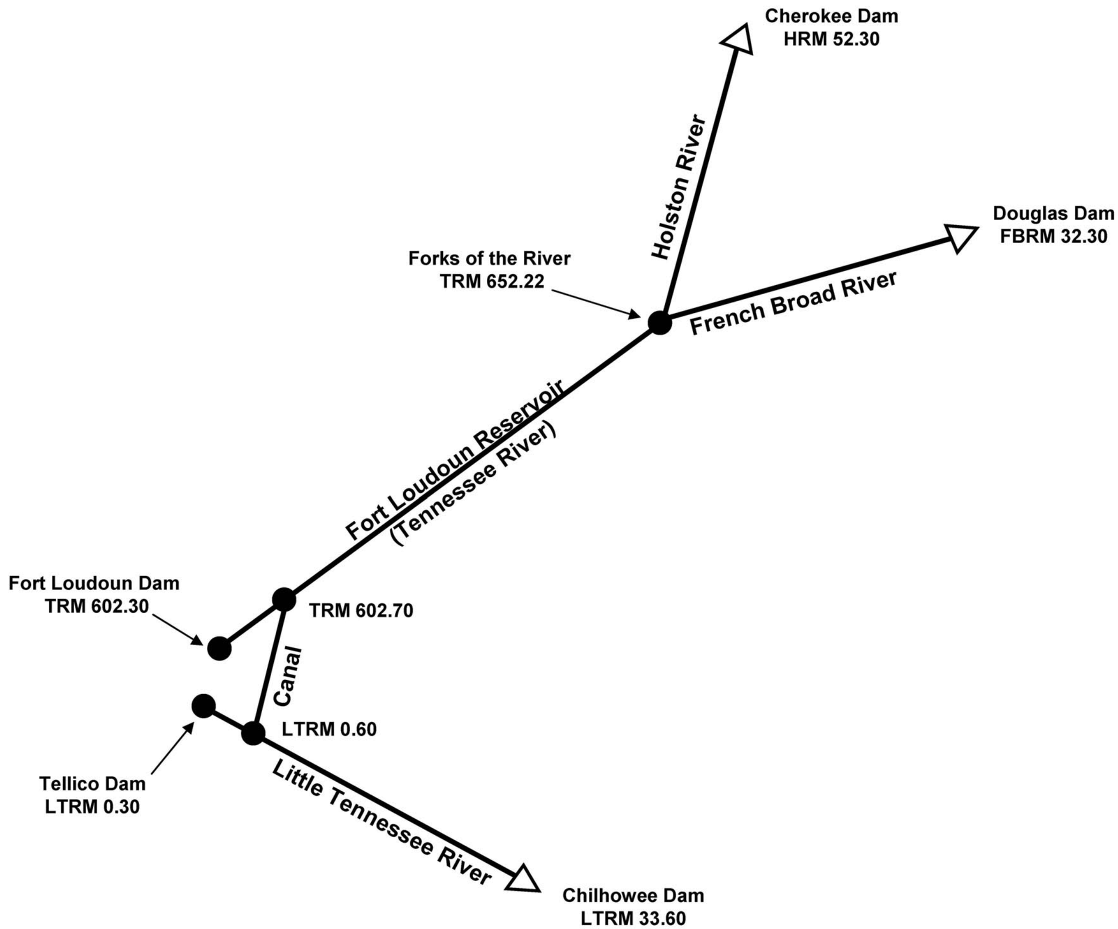


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Discharge Rating Curve, Watauga Dam

Figure 2.4-11 (Sheet 13 of 13)

Figure 2.4-11 Discharge Rating Curve, Watauga Dam (Sheet 13 of 13)

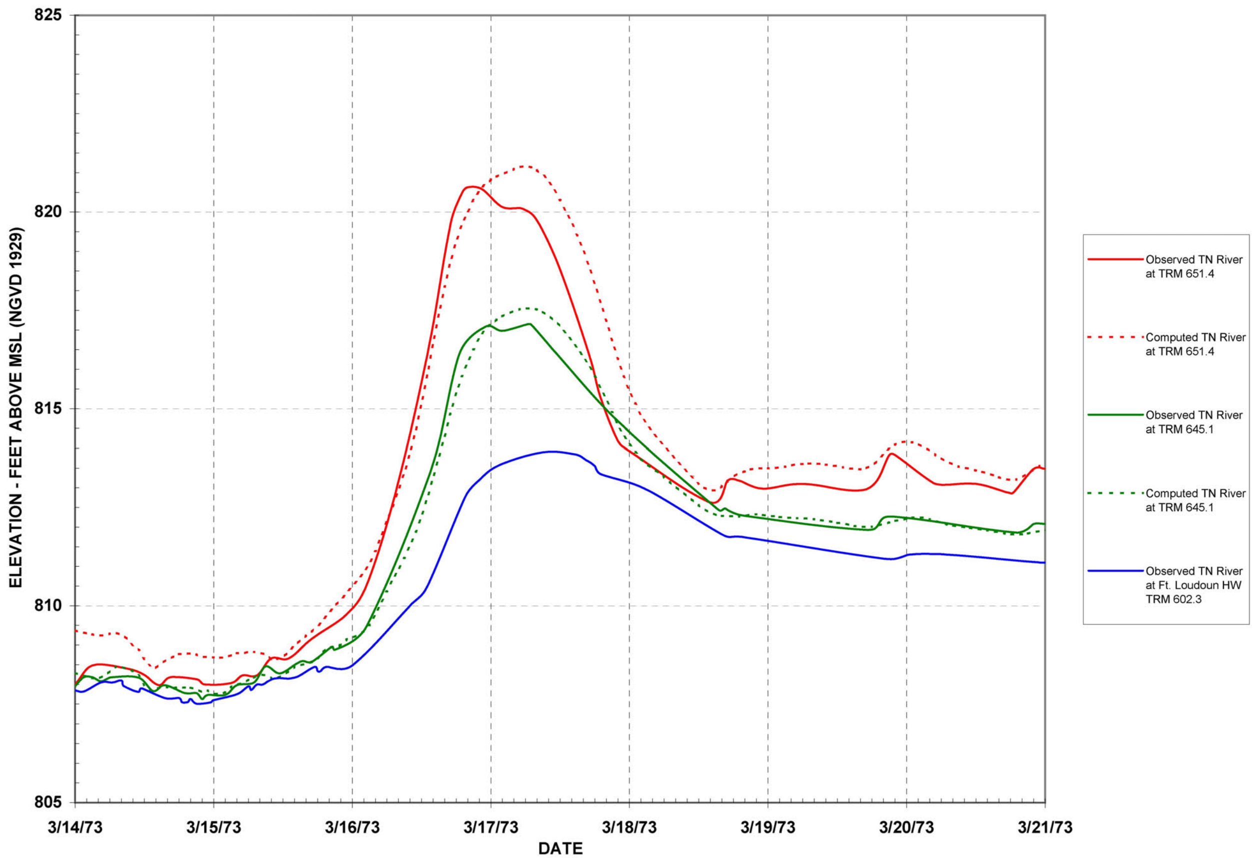


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Fort Loudoun - Tellico SOCH
Unsteady Flow Model Schematic

Figure 2.4-12

Figure 2.4-12 Fort Loudoun - Tellico SOCH Unsteady Flow Model Schematic



WATTS BAR NUCLEAR PLANT
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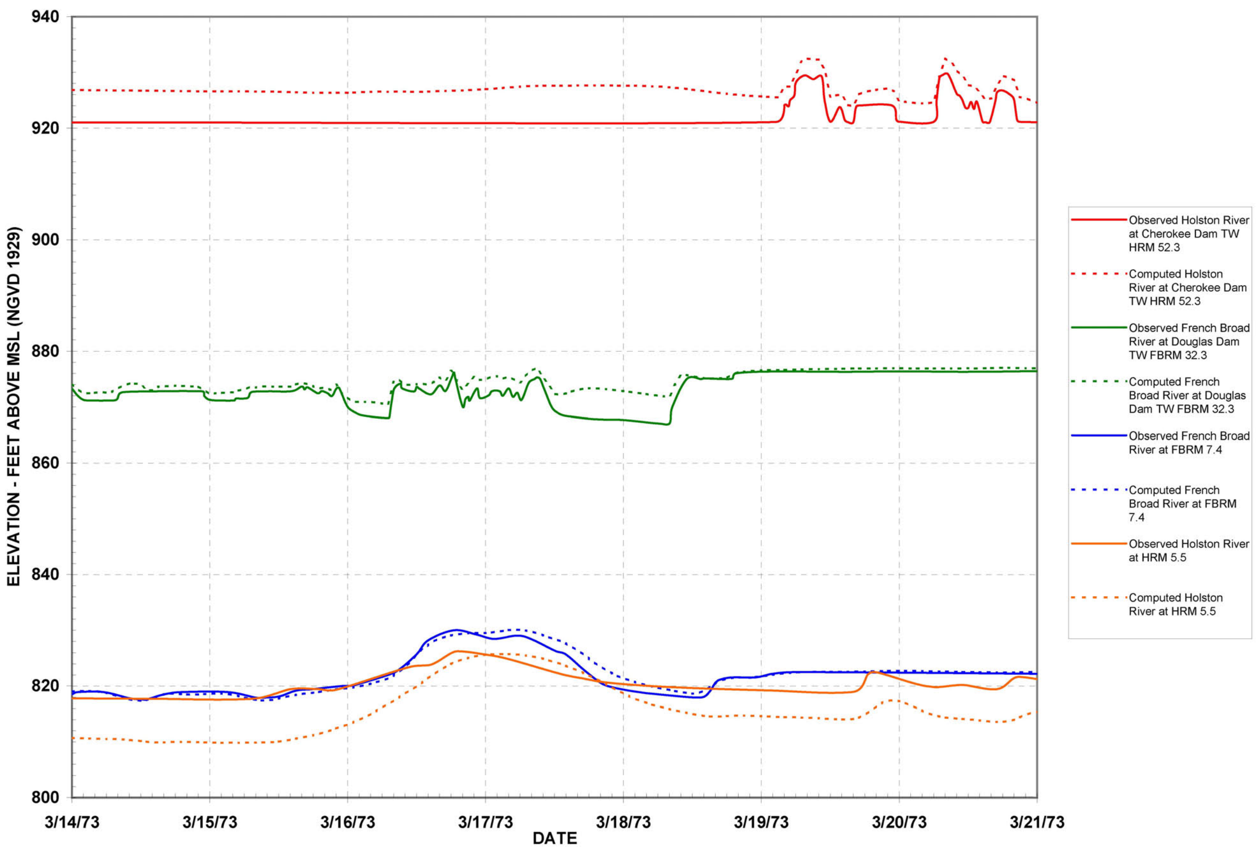
Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood

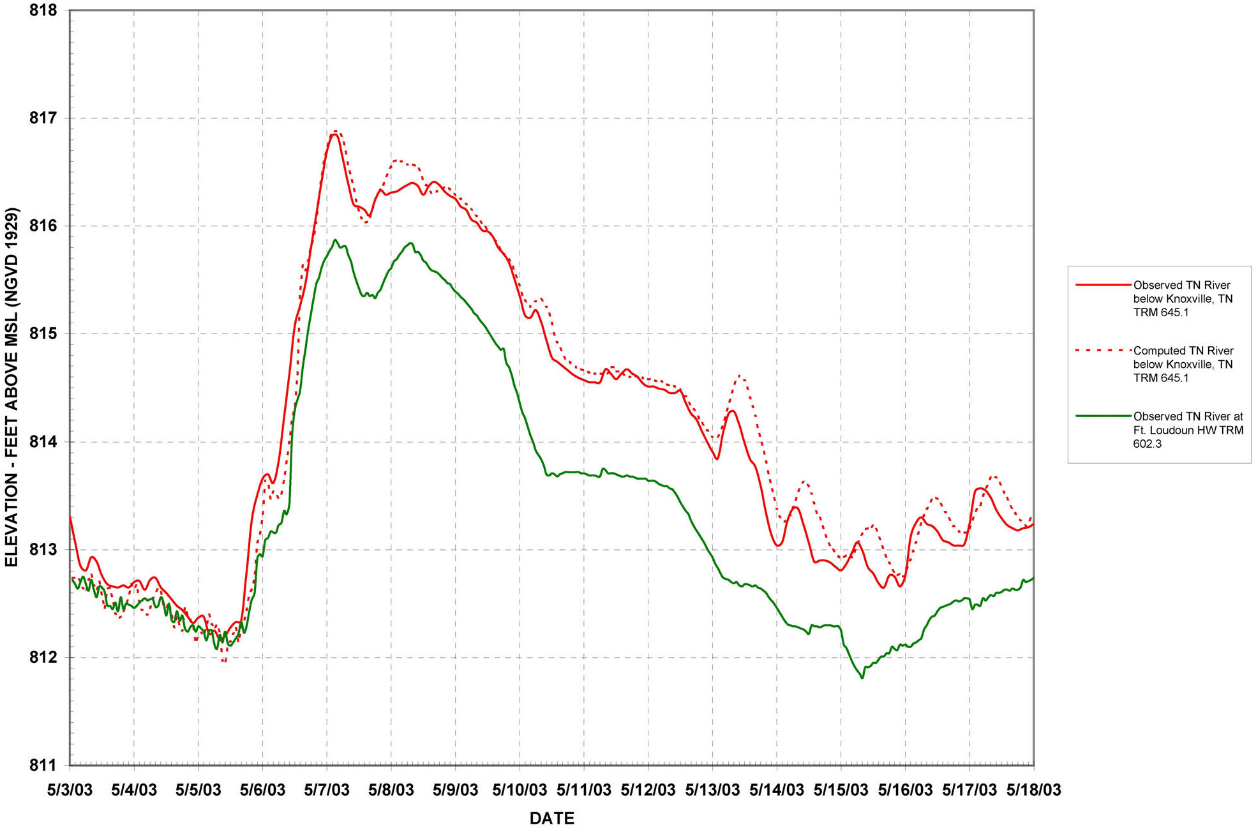
Figure 2.4-13 (Sheet 1 of 2)

Figure 2.4-13 Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood (Sheet 1 of 2)



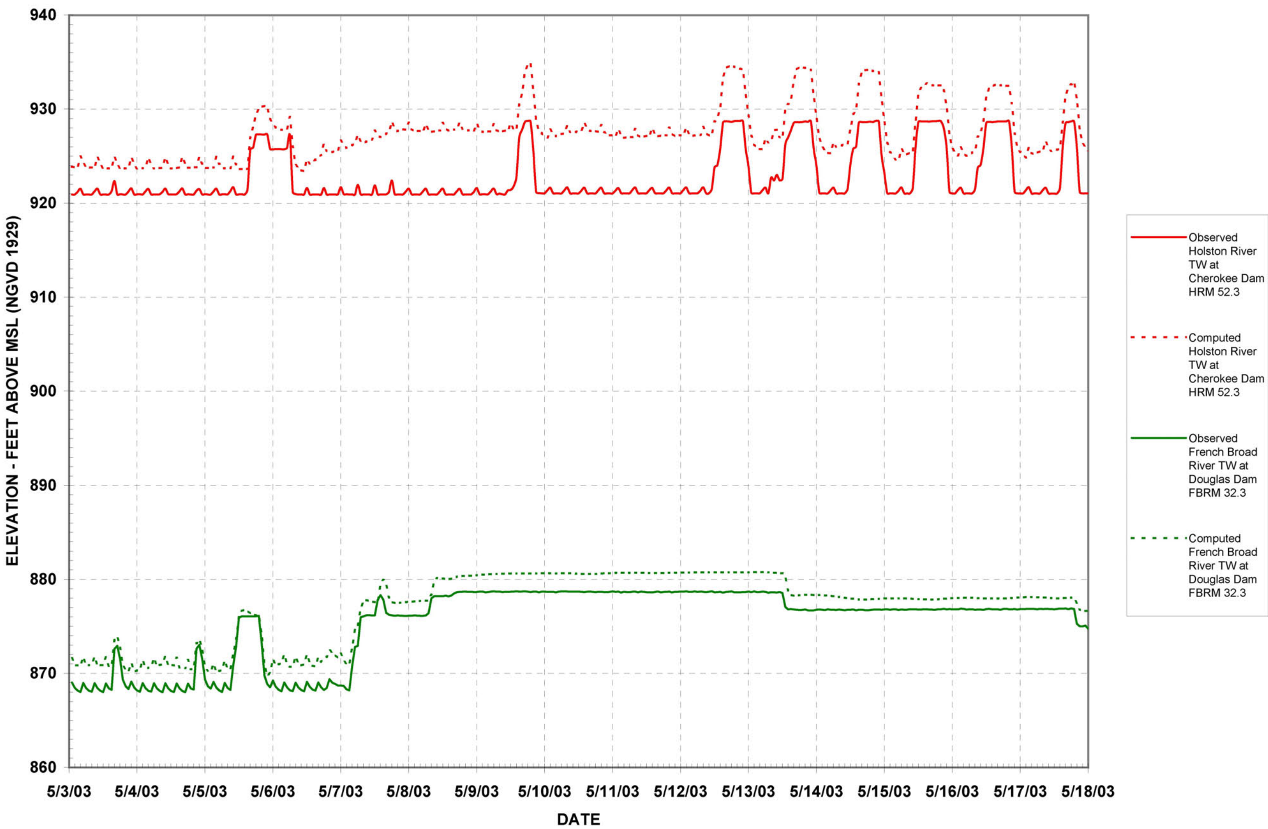
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood Figure 2.4-13 (Sheet 2 of 2)

Figure 2.4-13 Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood (Sheet 2 of 2)



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood Figure 2.4-14 (Sheet 1 of 3)

Figure 2.4-14 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 1 of 3)



WATTS BAR NUCLEAR PLANT
FINAL SAFETY
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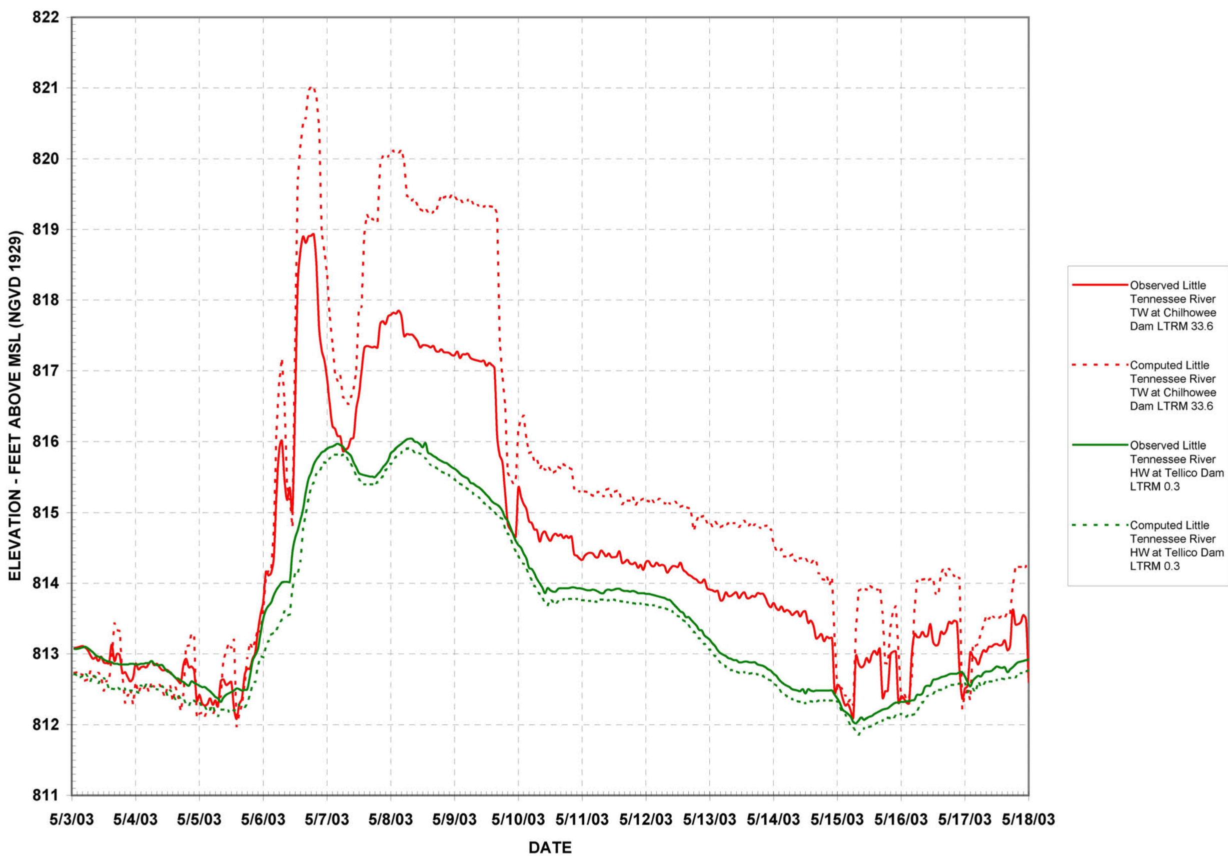
Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood

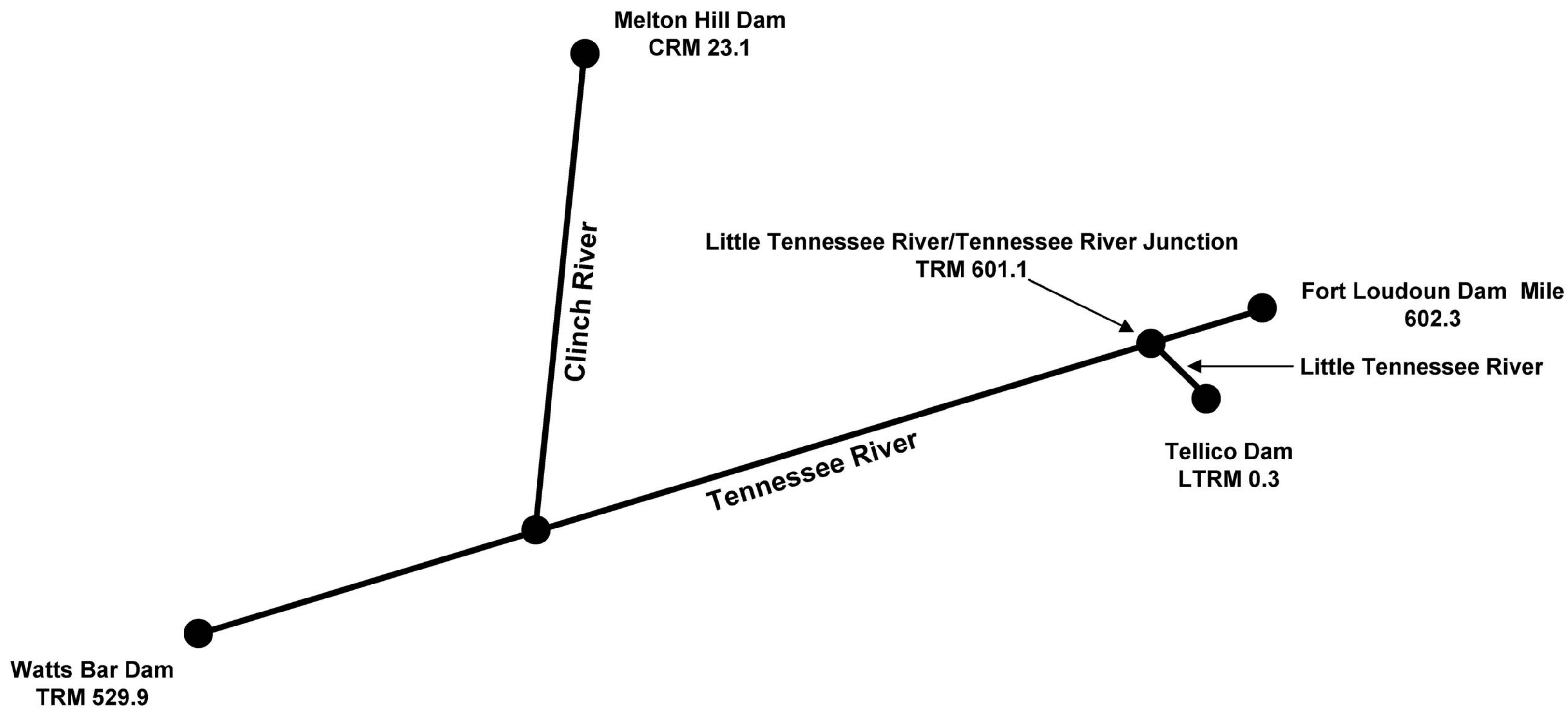
Figure 2.4-14 (Sheet 2 of 3)

Figure 2.4-14 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 2 of 3)



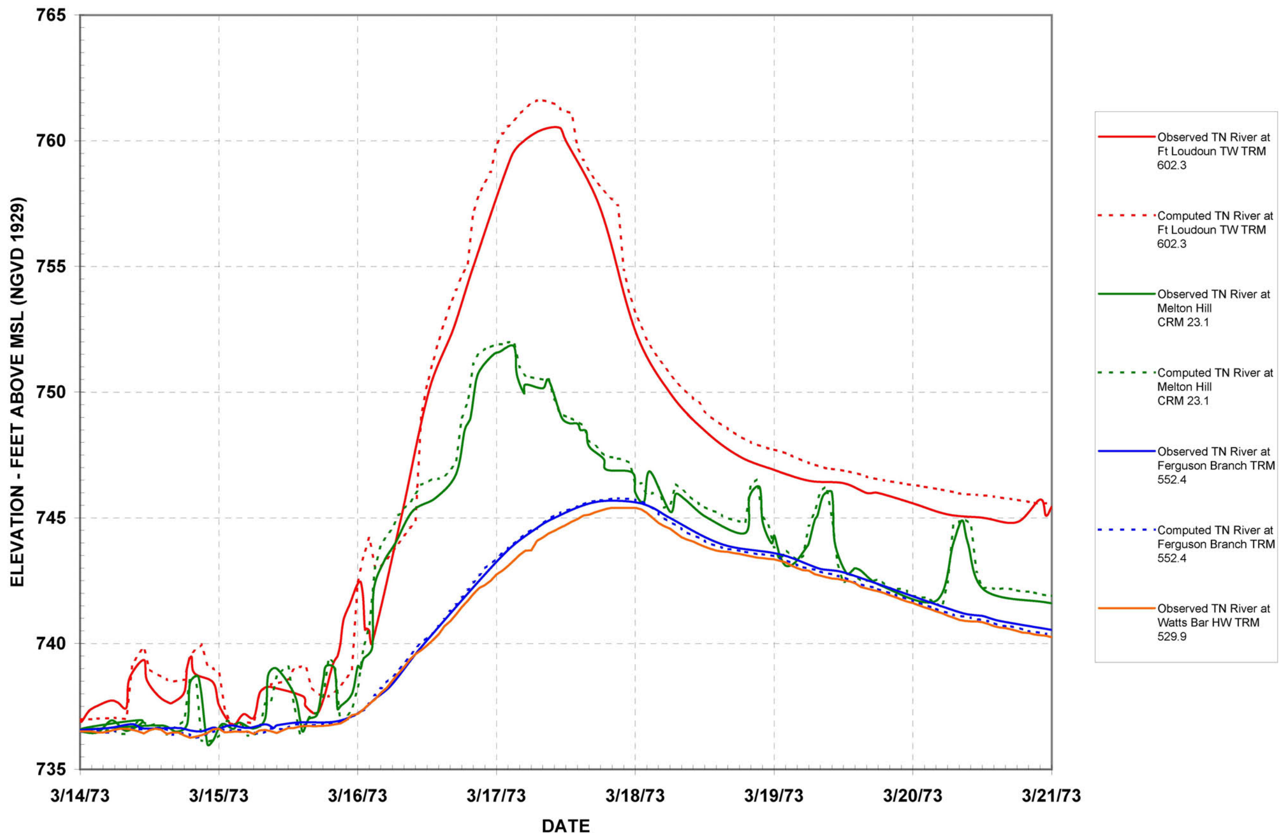
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood Figure 2.4-14 (Sheet 3 of 3)

Figure 2.4-14 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 3 of 3)



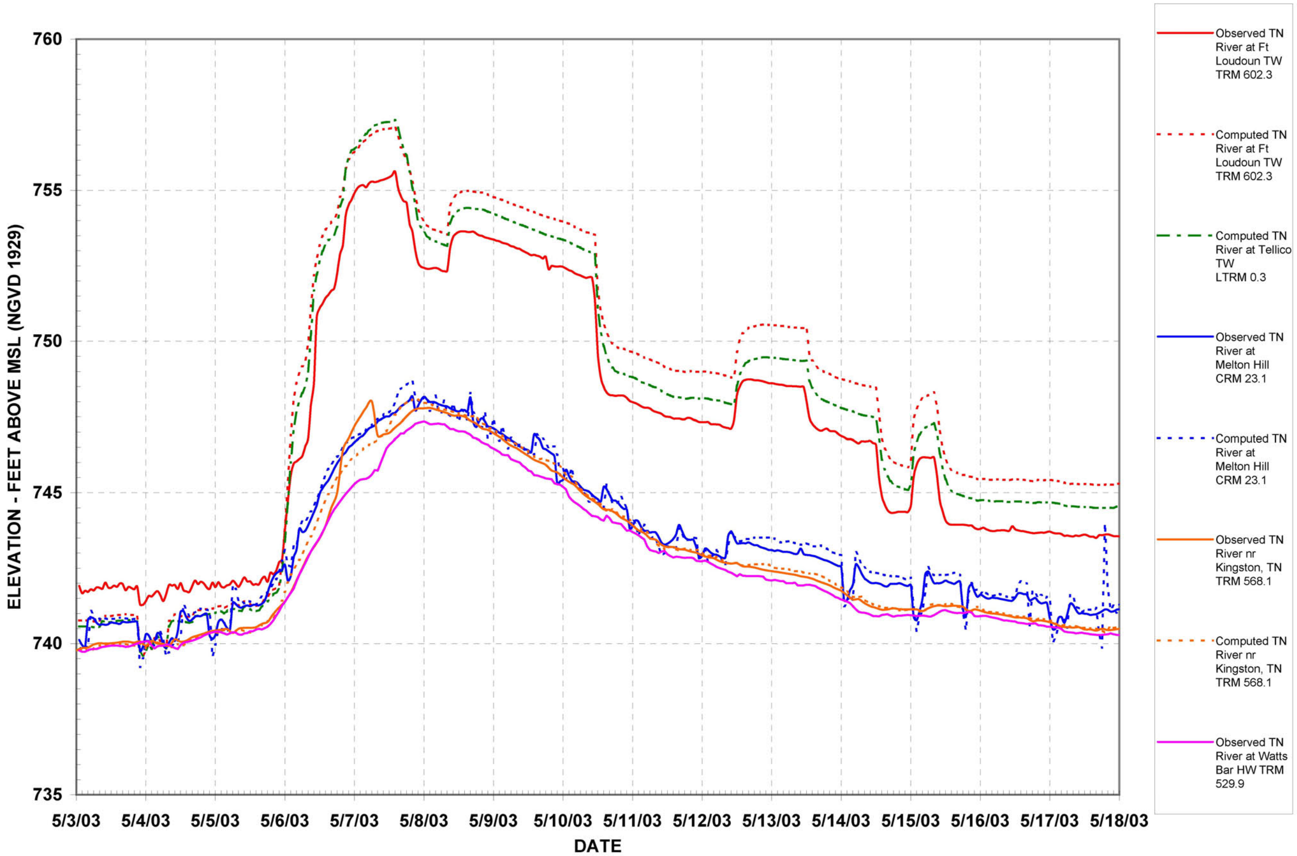
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Watts Bar SOCH Unsteady Flow Model Schematic Figure 2.4-15

Figure 2.4-15 Watts Bar SOCH Unsteady Flow Model Schematic



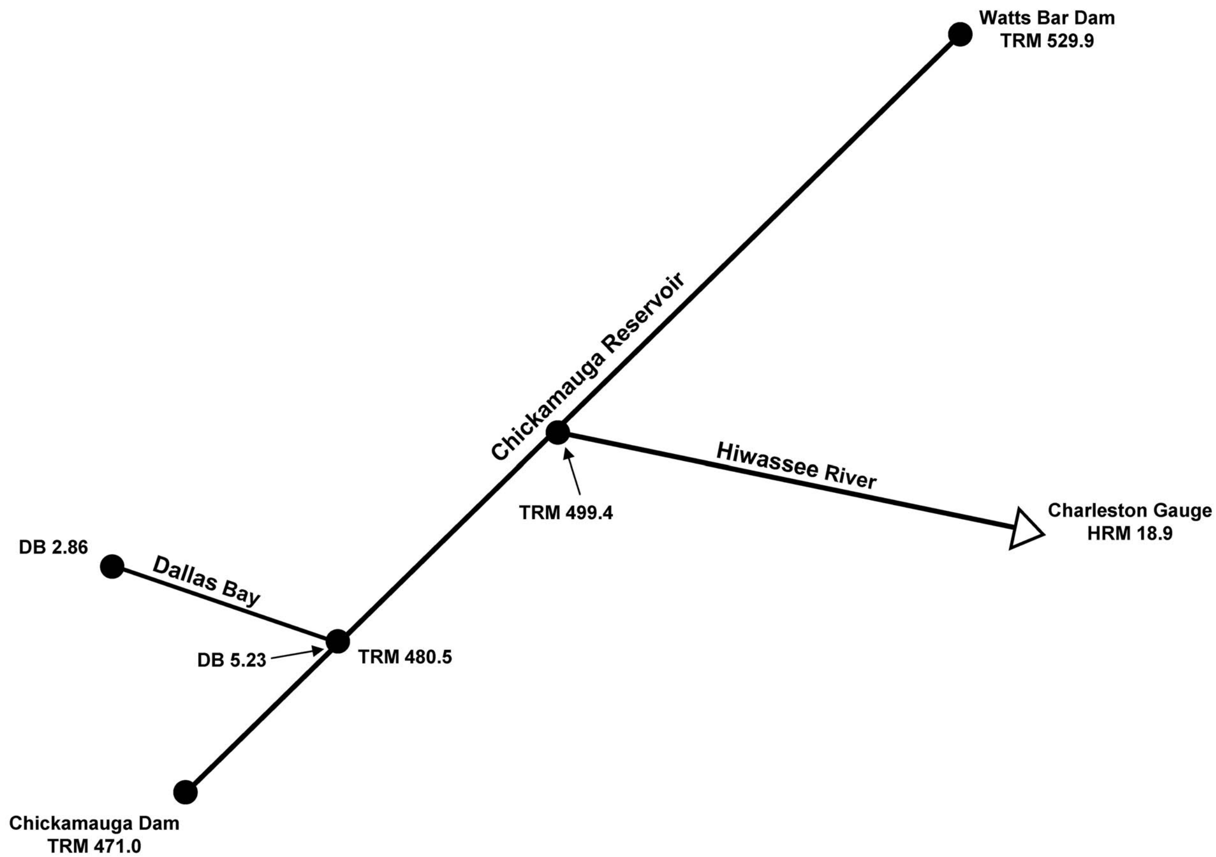
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Watts Bar Reservoir March 1973 Flood Figure 2.4-16

Figure 2.4-16 Unsteady Flow Model Watts Bar Reservoir March 1973 Flood



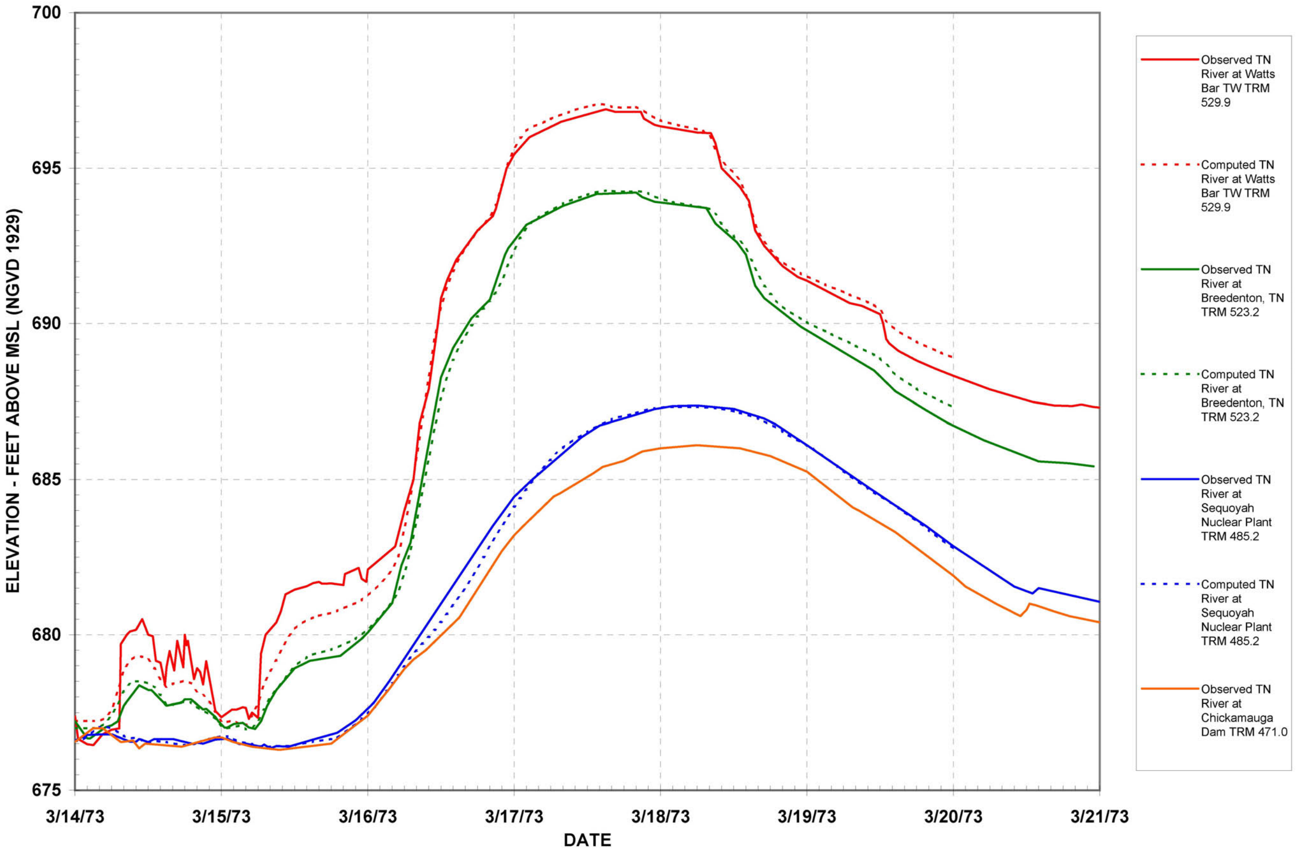
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Watts Bar Reservoir May 2003 Flood Figure 2.4-17

Figure 2.4-17 Unsteady Flow Model Watts Bar Reservoir May 2003 Flood



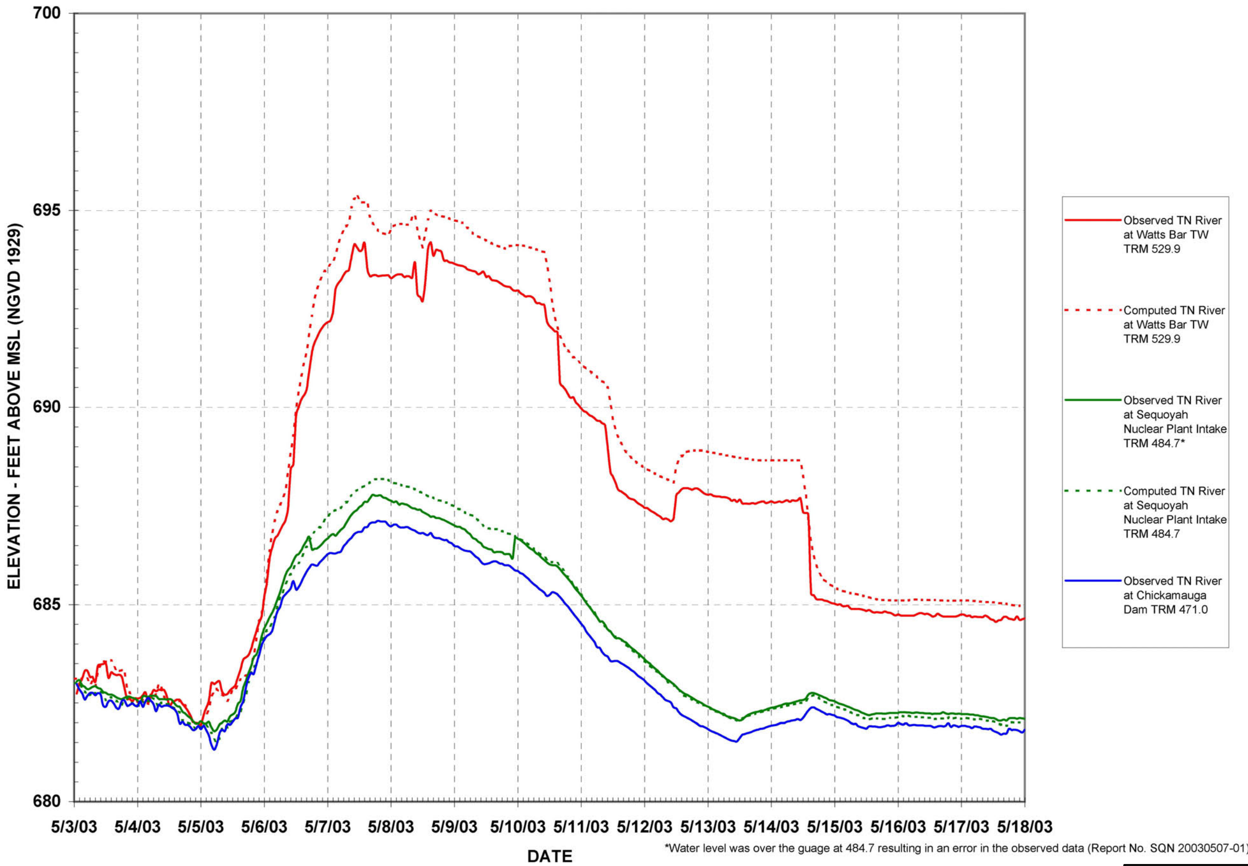
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Chickamauga SOCH Unsteady Flow Model Schematic Figure 2.4-18

Figure 2.4-18 Chickamauga SOCH Unsteady Flow Model Schematic



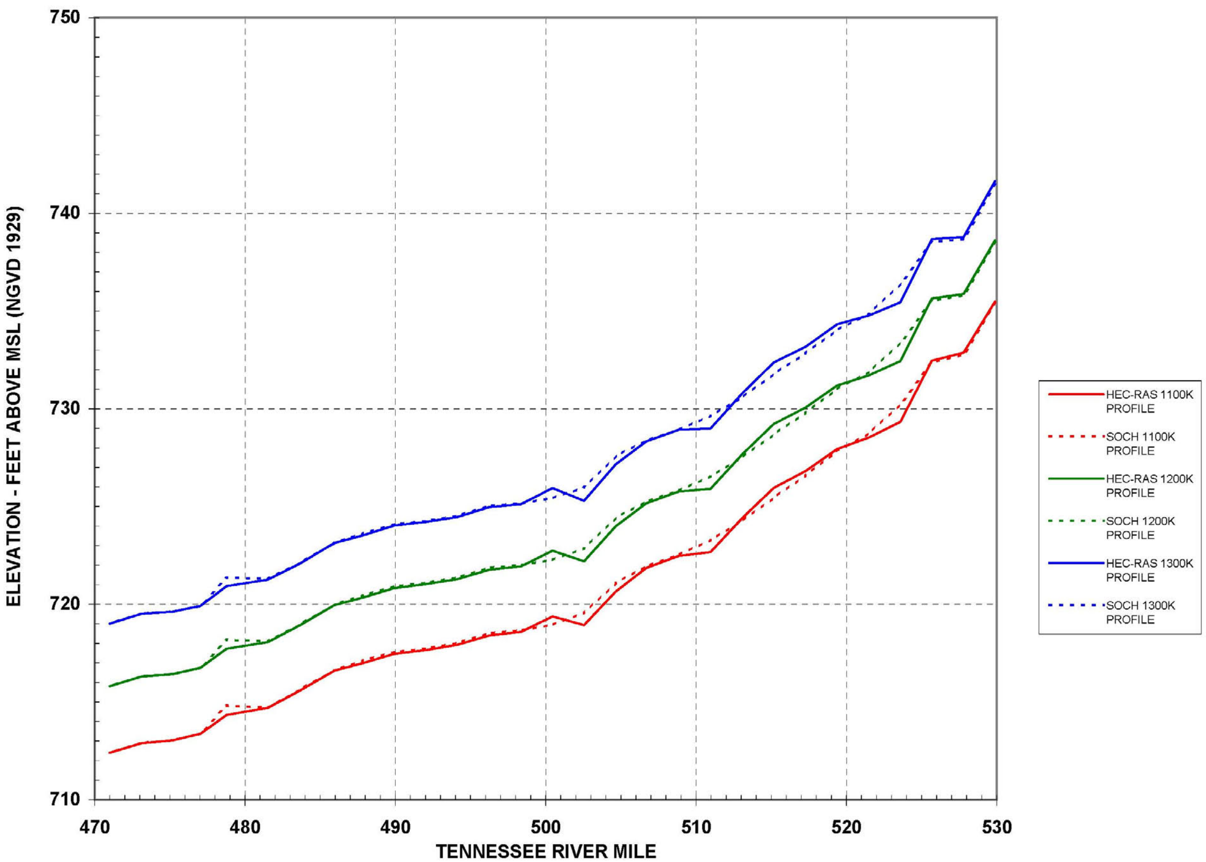
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Chickamauga Reservoir March 1973 Flood Figure 2.4-19

Figure 2.4-19 Unsteady Flow Model Chickamauga Reservoir March 1973 Flood



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Chickamauga Reservoir May 2003 Flood Figure 2.4-20

Figure 2.4-20 Unsteady Flow Model Chickamauga Reservoir May 2003 Flood

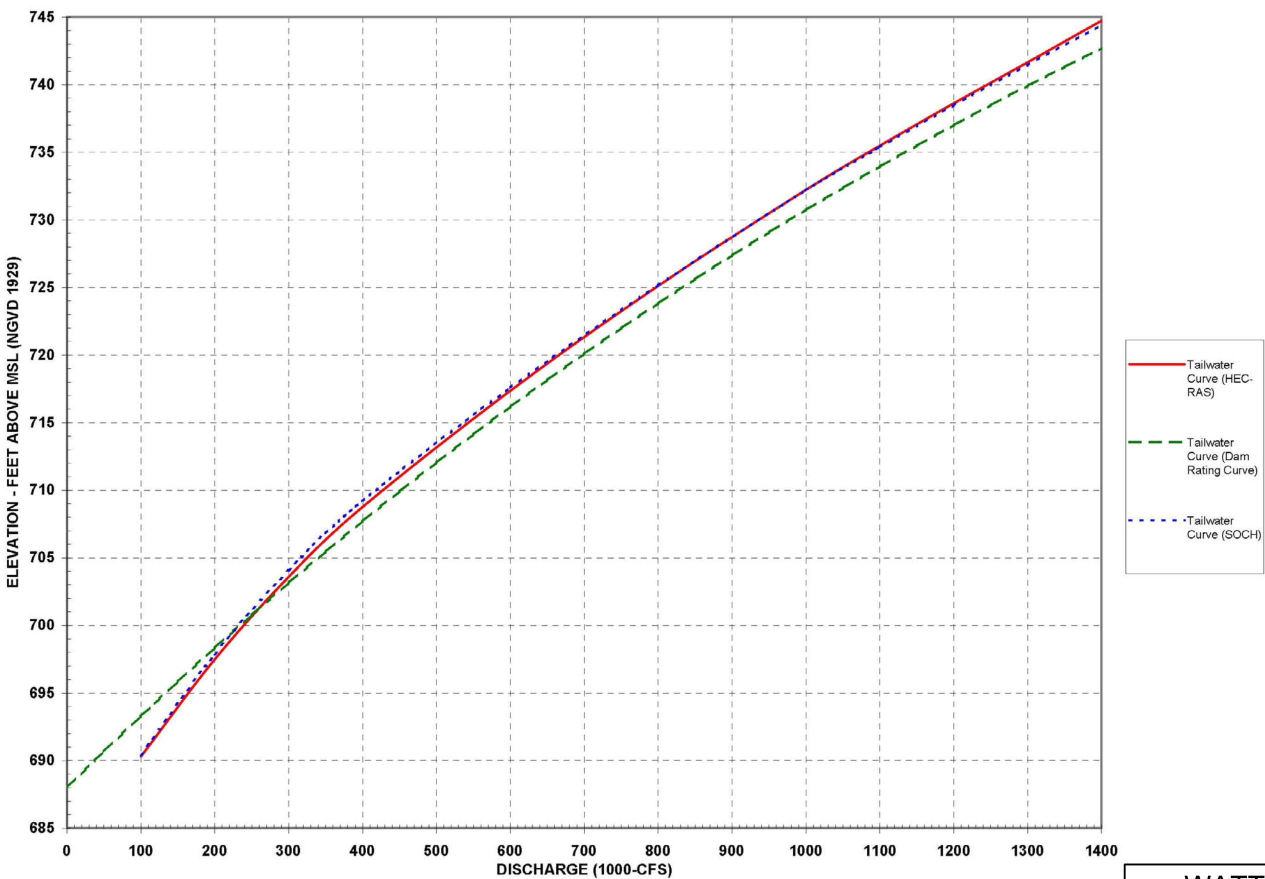


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Chickamauga Steady State
Profile Comparisons

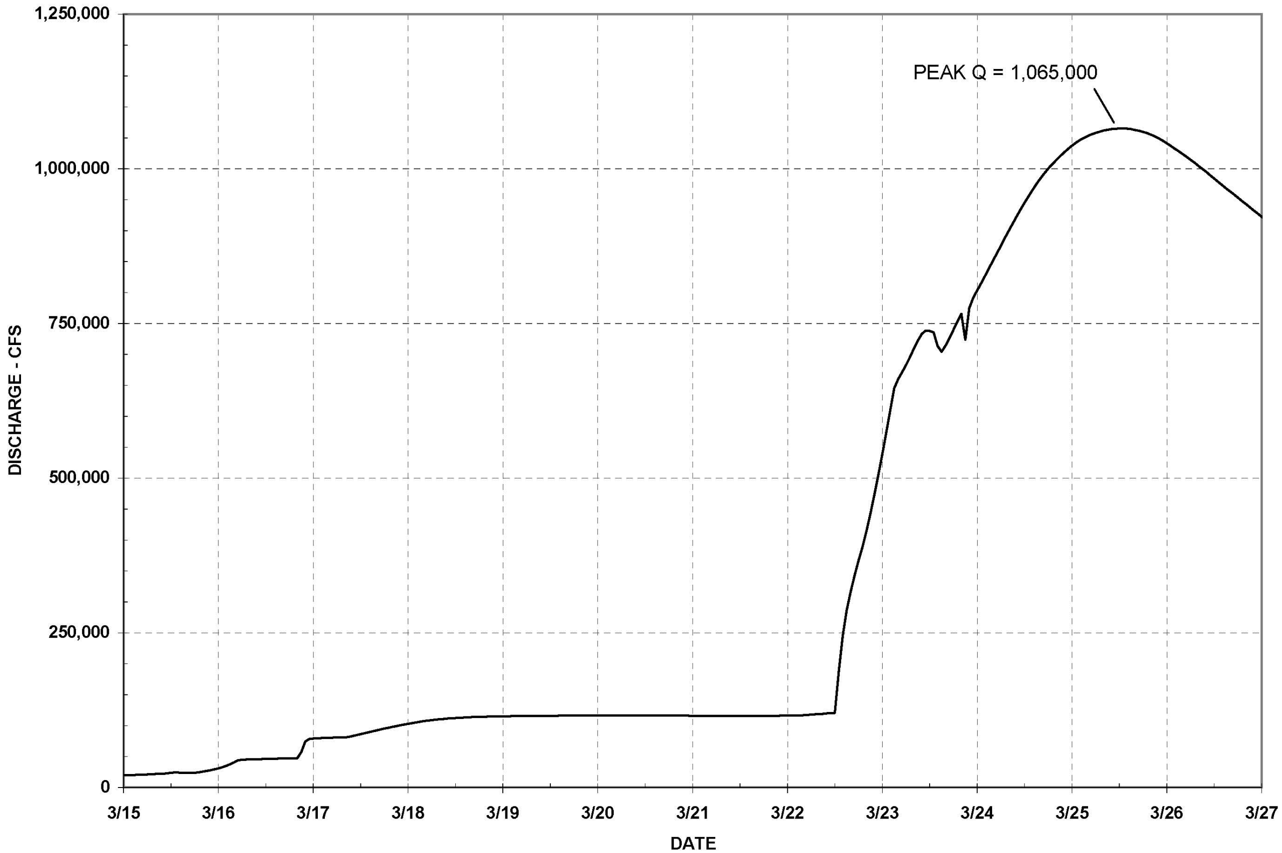
Figure 2.4-21

Figure 2.4-21 Chickamauga Steady State Profile Comparisons



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Tailwater Rating Curve, Watts Bar Dam Figure 2.4-22

Figure 2.4-22 Tailwater Rating Curve, Watts Bar Dam



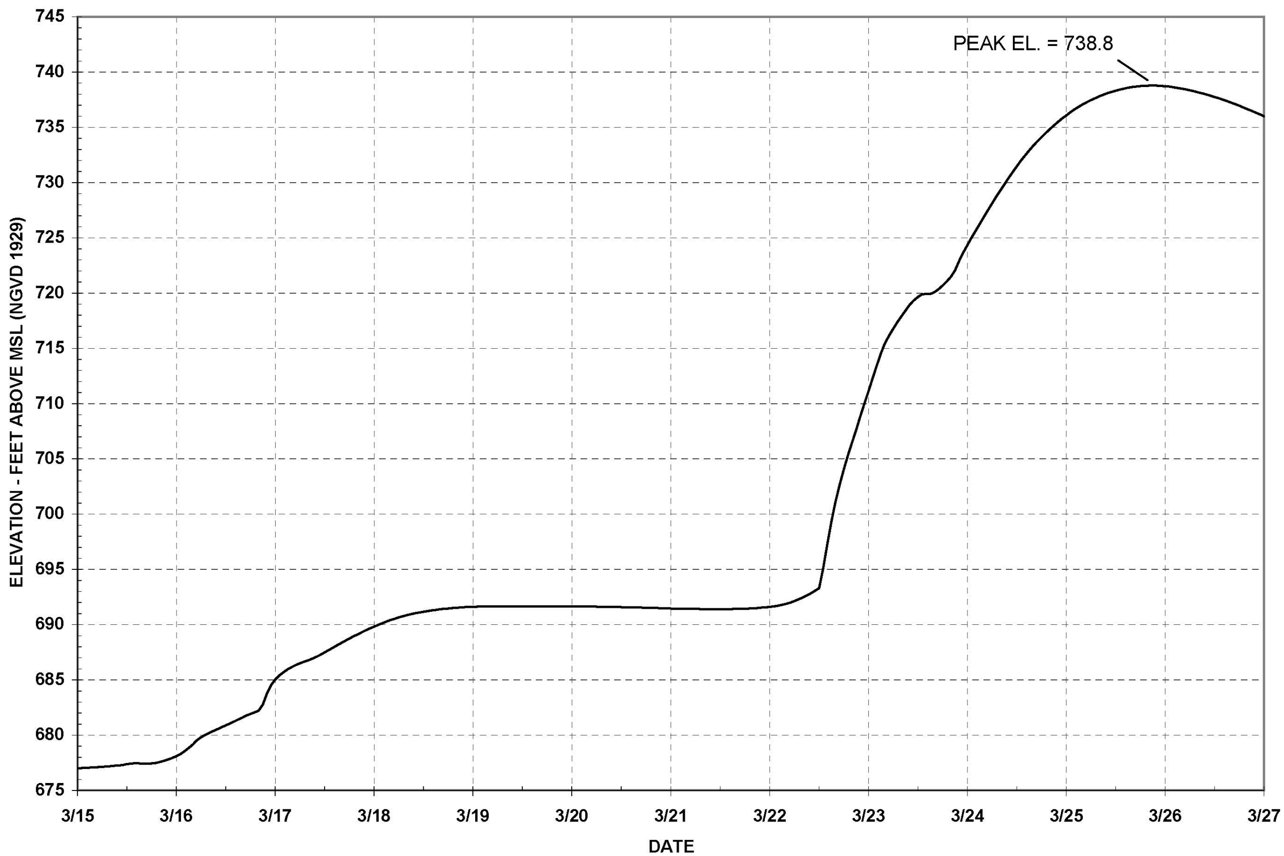
WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

PMF Discharge Hydrograph at
Watts Bar Nuclear Plant

Figure 2.4-23

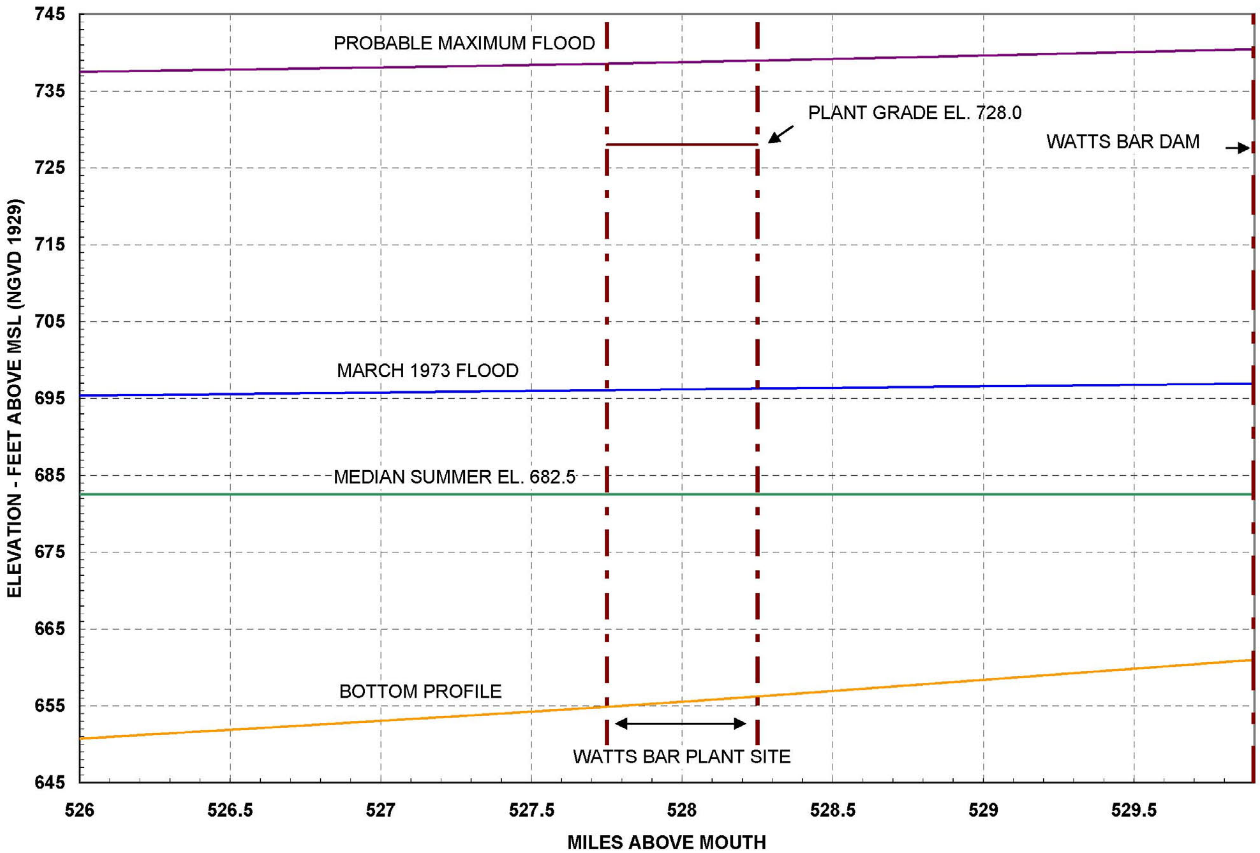
Figure 2.4-23 PMF Discharge Hydrograph at Watts Bar Nuclear Plant

Figure 2.4-24 West Saddle Dike Location Plan and Section



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
PMF Elevation Hydrograph at Watts Bar Nuclear Plant Figure 2.4-25

Figure 2.4-25 PMF Elevation Hydrograph at Watts Bar Nuclear Plant



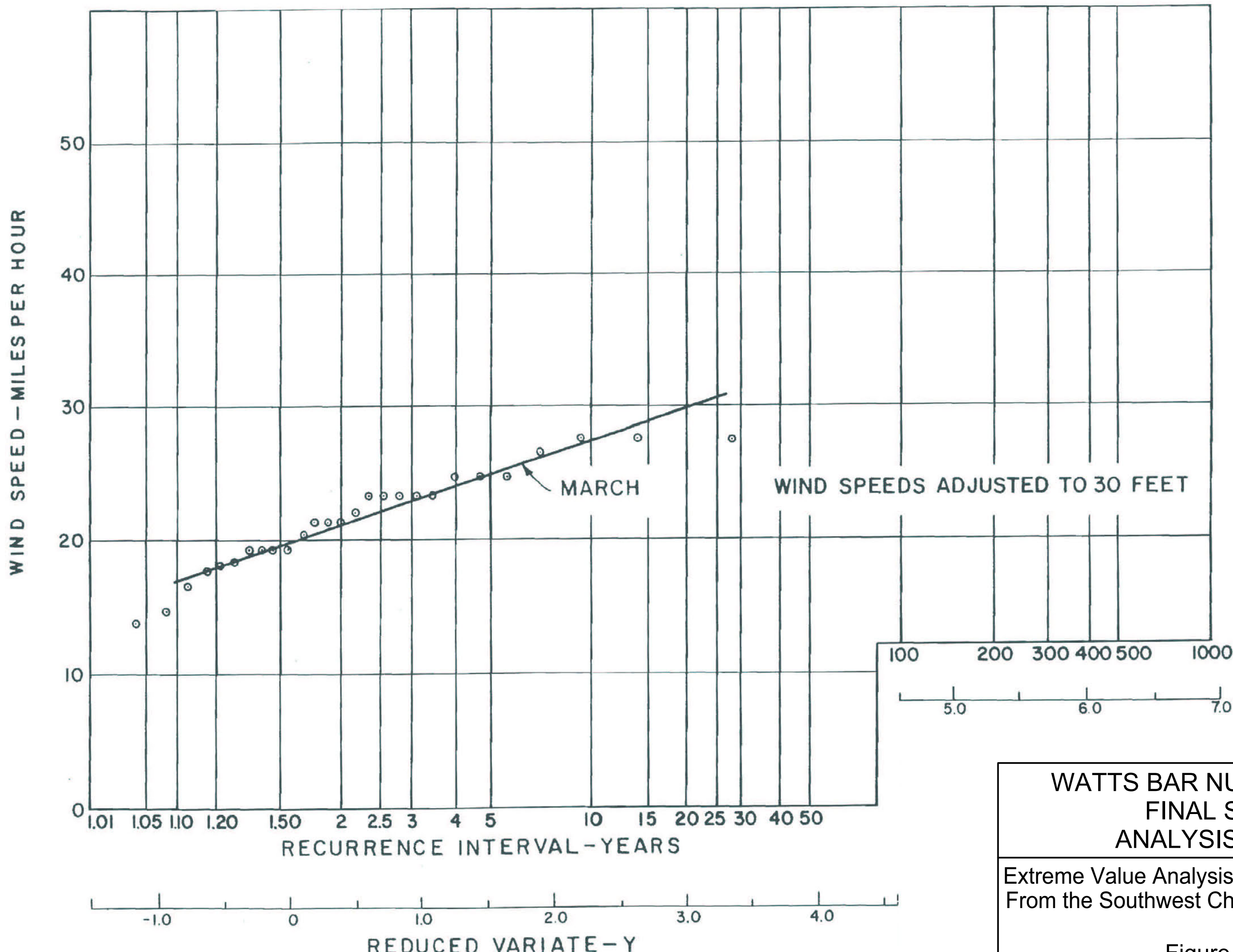
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Probable Maximum Flood and Bottom Profiles Figure 2.4-26

Figure 2.4-26 Probable Maximum Flood and Bottom Profiles

Security-Related Information - Withheld Under 10CFR2.390

Figure 2.4-27 Main Plant General Grading Plan

Figure 2.4-28 Watts Bar Nuclear Plant Wind Wave Fetch



WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Extreme Value Analysis 30-Minute Wind Speed From the Southwest Chattanooga, TN 1948-74

Figure 2.4-29

Figure 2.4-29 Extreme Value Analysis 30-Minute Wind Speed From the Southwest Chattanooga, TN 1948–74

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