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 734.9 would result from an occurrence of the probable maximum storm. Allowances for concurrent wind waves could raise lake levels to Elevation 736.2 with run up on the 4:1 slopes approaching the plant reaching about Elevation 736.9.

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 All surface water supplies withdrawn from the 58.9 mile reach of the mainstream of the Tennessee River between Watts Bar Dam (TRM 529.9) and Chickamauga Dam (TRM 471.0) are listed in Table 2.4-4.

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
	(5) Emergency Exit	1	730.0
	(6) Door to Turbine Bldg.	2	755.0

Structure	Access	Accesses	Elev.
Shield Building			
	(1) Personnel Lock	1	714.0
	(2) Equipment Hatch	1	753.0
	(3) Personnel Lock	1	755.0
Diesel Generator Building	(1) Equipment Access Doors	4	742.0
	(2) Emergency Exits		742.0
	(3) Personnel Access Door	1	742.0
	(4) Emergency Exit	1	760.5
Additional Diesel Generator Building	(1) Equipment Access Door	2	742.0
	(2) Personnel Access Door	1	742.0
	(3) Emergency Exit	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 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. Mean annual precipitation for the Tennessee Valley is shown in Figure 2.4-1. 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 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 35,400 acres, with a volume of 628,000 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 reservoirs in the TVA system upstream Watts Bar Nuclear Plant, ten of which have substantial reserved flood detention capacity during the flood season. Table 2.4-1 lists pertinent data for TVA's major dams prior to modifications made by the Dam Safety Program (See Table 2.4-16). Figures 2.4-2 through 2.4-14 show general plans, elevations, and sections for these dams and Chickamauga Dam downstream. 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 2.4-2 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 4,069,200 acre-feet of detention capacity equivalent to 6.1 inches on the 12,452-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 1.9 inches on the remaining 4,858-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. Figures 2.4-15 through 2.4-24 show the reservoir seasonal operating guides for reservoirs above the plant site. Table 2.4-3 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 6.2 inches on January 1 to 4.9 inches on March 15 and decreasing to 1.2 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 eight major tributary dams in the 3,480-square-mile intervening watershed, of which three have substantial reserved capacity. On March 15, near the end of the flood season, these provide a minimum of 366,700 acre-feet equivalent to 7.1 inches on the 968-square-mile controlled area. Chickamauga Dam contains 347,000 acre-feet of detention capacity on March 15 equivalent to 2.6 inches on the remaining 2,512 square miles. Figure 2.4-26 shows the seasonal operating guide for Chickamauga.

Elevation-storage relationships for the 12 reservoirs above the site and Chickamauga, downstream, are shown in Figures 2.4-25 and 2.4-27 through 2.4-38. Curves determined at selected years as part of TVA's program of monitoring changes due to sedimentation are shown. These show that sedimentation is not significant in these reservoirs.

The original hydrologic design of the dams was based upon a combination of design flood and freeboard which in some cases did not meet probable maximum flood (PMF) criteria imposed by Regulatory Guide 1.70.17, January 1975. The potential consequences of overtopping of those dams not meeting current criteria were evaluated where failure would significantly influence plant site flood levels and were described in Section 2.4.3.4 per the original analysis.

In 1982, TVA officially began a safety review of all its dams. The TVA Dam Safety Program was designed to be consistent with the Federal Emergency Management Agency's (FEMA) Federal Guidelines for Dam Safety and similar efforts by other Federal agencies. Technical studies and engineering analyses were conducted and physical modifications implemented to ensure the hydrologic and seismic integrity of the TVA dams and demonstrate that TVA's dams can be operated in accordance with FEMA guidelines. Table 2.4-16 provides the status of TVA Dam Safety hydrologic modifications as of early 1998. These modifications enable these projects to safely pass the probable maximum flood. The remaining hydrologic modifications planned for Bear Creek Dam and Chickamauga dam will not affect Watts Bar in any manner which might invalidate the reanalysis described below.

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

(1) The computer programs and modeling methods were the same as previosly used and documented in the plant FSAR.

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

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

(4) The planned modification of Chickamauga Dam (armoring the embankment to permit overtopping) was conservatively assumed to have been implemented for the purpose of calculating flood effects. However, the condition of Chickamauga Dam, 57 miles downstream, has negligible impact on flood levels at the plant.

(5) Bear Creek Dam is downstream of the nuclear plant and its planned modification has no relevance to this reassessment.

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 upstream flow in Chickamauga Reservoir.

Based upon Watts Bar Dam discharge records since dam closure in 1942, the average daily streamflow at the plant is 27,800 cfs. The maximum daily discharge was 187,000 cfs on December 30, 1941 prior to present regulation. Daily average releases of zero have been recorded on seven occasions during the past 28 years. Flow data for water years 1960-1987 with regulation essentially equivalent to present conditions indicate an average rate of about 23,700 cfs during the summer months (May-October) and about 31,900 cfs during the winter months (November-April). Flow durations based upon Watts Bar Dam discharge records for the period 1960-1987 are tabulated below:

Average Daily Discharge, cfs	Percent of Time Equalled or Exceeded
5,000	98.9
10,000	93.2
15,000	83.5
20,000	69.3
25,000	50.6
30,000	32.9
35,000	20.1

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 2.4-4. 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 retention capacity above the Watts Bar site is small. Therefore, flood records for the period 1952 to date can be considered representative of prevailing conditions. Figure 2.4-40 shows the known flood experience at Chattanooga in diagram form. The maximum known flood under natural conditions occurred in 1867. This flood reached elevation 716.3 at Watts Bar Nuclear Plant site with a discharge of 440,000 cfs. The maximum flood under present-day regulation reached Elevation 696.18 at the site on March 17, 1973.

The following tabulation lists the highest floods at the Watts Bar Nuclear Plant site under present-day regulation:

Date	Elevation, Feet	Discharge, cfs
February 2, 1957	692.08	157,600
November 19, 1957	693.17	151,600
March 13, 1963	693.85	167,700
January 1, 1970	692.27	167,300
March 17, 1973	696.18	184,800

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 position 2 of Regulatory Guide 1.59.

The types of events evaluated to determine the worst potential flood included (1) Probable Maximum Precipitation (PMP) on the total watershed and critical sub-water sheds 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 plant site flood level from any cause is Elevation 734.9. This elevation would result from the PMP critically centered on the watershed as described in Section 2.4.3. The maximum flood level is 3.2 feet lower than originally determined in the FSAR as a result of dam safety modifications.

Wind waves from a March wind with velocity of 21 miles per hour was assumed to occur coincident with the flood peak. This would create waves 2.0 feet high (trough to crest) and produce maximum lake levels to Elevation 736.2.

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. Flood warning criteria and forecasting techniques have been developed to assure that there will always be adequate time to shut the plant down and be ready for floodwaters above plant grade. 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 approaching plant grade Elevation 728. In all such events there is time for safe plant shutdown after the seismic event before plant grade would be crossed. 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 reaches Elevation 736.9 which is 5.1 feet 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 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 water level for the remaining structure is Elevation 740.1. The Auxiliary and Control Buildings will flood with the water level at Elevation 729. The design basis water level for the Shield Building is Elevation 740.1. The Diesel Generator Building is located above the design basis water level (Elevation 740.1).

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 2.4-14 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 2.4-15 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 total watershed above the plant with consideration given to seasonal and areal variations. The original PMF determination also considered a PMF at upstream tributary dams whose failure has the potential to cause maximum plant site flood levels. Dam safety modifications have eliminated the potential of a PMF at upstrean tributary dams to cause maximum plant site flood levels.

Two basic storm situations were found to have the potential to produce a maximum flood at Watts Bar Nuclear Plant. These are (1) a sequence of March storms producing maximum rainfall on the 21,400-square-mile watershed above Chattanooga and (2) a sequence of March storms centered and producing maximum rains in the basin to the west of the Appalachian Divide and above Chattanooga, hereafter called the 7,980-square-mile storm. The maximum flood level at the plant would be caused by the PMP on the 7,980-square-mile storm. The flood level for the 21,400 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 1,288,000 cfs for the 7,980-square-mile storm. The resulting probable maximum elevation at the plant would be 734.9 for both floods 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.45[1] defines PMP for watersheds above tributary dams. These reports define depth-area-duration characteristics and antecedent storm potentials. Snowmelt is not a factor in generating maximum floods at the site.

Two basic total watershed storm positions have the potential to produce a maximum flood at the Watts Bar plant site. One would produce maximum rainfall over the 21,400-square-mile watershed above Chattanooga. The other would produce maximum rains on the part of the basin downstream from major TVA tributary reservoirs, hereafter referred to as the 7,980-square-mile storm. These storms would occur in March. Depths for other months would be less.

Two possible isohyetal patterns producing the 21,400-square-mile area depths are presented in Report No. 41[4]. The pattern critical to this study is the 'downstream pattern' shown in Figure 2.4-41 along with the maximum 6-hour storm depths. The isohyetal pattern for the 7,980-square-mile storm is shown in Figure 2.4-42 along with the maximum 6-hour storm depths. The pattern is not orographically fixed and can be moved parallel to the long axis northeast and southwest along the valley.

Potential storm amounts differing by seasons were analyzed in sufficient number to make certain that the March storms would be controlling. Enough centerings were investigated to assure that a most critical position was used.

A 3-day storm, 3 days antecedent to the 3-day main storm, was assumed to occur in all total basin PMP situations. Depths equivalent to 40% of the main storm were used for the antecedent storms with uniform areal distribution as recommended in Report No. 41[4].

In the original analysis, storms producing PMP above upstream tributary dams were also evaluated. Storm depths and isohyetal patterns for tributary watersheds with drainage areas less than 3000 square miles are defined in Hydrometeorological Report No. 45[1]. For Douglas Dam, drainage area 4541 square miles, a 72-hour PMP storm depth of 22.3 inches was determined using Hydrometeorological Report No. 45[1] data. Nonorographic PMP was determined from Figure 3-11 from Report No.45[1] and increased slightly for orgraphic effects by the ratio of mean annual precipitation above Douglas to mean annual nonorographic precipitation for the area (Figure 3-16 from Report No. 45[1]). Areal distribution of PMP rainfall was patterned after mean annual precipitation isohyets and isohyets of Hydrometeorological Report No. 45[1] PMP estimates for subbasins above Douglas Dam. Residual rainfall on the area below Douglas and above Watts Bar was such that the total above Watts Bar was 80% of the PMP. These storms would occur in the June to October warm season months. For conservatism, a July date was postulated because reservoirs would be at maximum summer levels.

A 3-day storm, 3 days antecedent to the 3-day main storm, was applied in these small area PMP situations. Depths equal to 30% of the main storm were used for the antecedent storms with uniform areal distribution as recommended in Report No. 45[1].

A standard time distribution pattern was adopted for all storms based upon major observed storms transposable to the Tennessee Valley and in conformance with the usual practice of Federal agencies. The adopted distribution is shown on Figure 2.4-43.

Both the 21,400 square mile total basin storm with downstream orographically fixed pattern (Figure 2.4-41) and the 7,980 square mile storm centered at Bulls Gap, Tennessee (50 miles northeast of Knoxville) produce essentially the same peak stage. These storms would follow an antecedent storm commencing on March 15. For the purpose of this report all subsequent rainfall statistics are for the 21,400-square-mile storm because of equivalent flood levels larger total storm volumes involved and the fact that it produces the controlling storm for the Sequoyah Nuclear Plant downstream. Translation of the PMP from Report No. 41[4] to the basin above Watts Bar results in an antecedent storm producing an average precipitation of 6.44 inches in 3 days, followed by a 3-day dry period, and then by the main storm producing an average precipitation of 16.34 inches in 3 days. Figure 2.4-44 is an isohyetal map of the maximum 3-day PMP. Basin rainfall depths are given in Table 2.4-5.

2.4.3.2 Precipitation Losses

A multi-variable relationship, used in the day-to-day operation of the TVA system, has been applied to determine precipitation excess P_e directly. The relationships were developed from observed data. They relate precipitation excess to the rainfall, week of the year, geographic location, and antecedent precipitation index (API). In their application P_e becomes an increasing fraction of rainfall as the storm progresses in time and becomes equal to rainfall when from 6 to 16 inches have fallen.

For this study, a median API as determined from past records was used at the start of the antecedent storm. The antecedent storm is so large, however, that the main storm is not sensitive to variations in adopted API.

For review purposes, precipitation losses have been determined by subtracting P_e from rainfall. In the critical probable maximum storm losses are 2.24 inches, amounting to 35% of rainfall, for the 3-day antecedent storm, and 1.78 inches, 11% of rainfall, for the 3-day main storm above Watts Bar Dam. Table 2.4-5 displays the API, rain, and precipitation excess for each of the 45 subwatersheds of the hydrologic model for the Watts Bar probable maximum flood.

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 45 unit areas and includes the total watershed above Chickamauga Dam downstream. Unit hydrographs are used to compute flows from the unit areas. The unit area flows are combined with appropriate time sequencing or

channel routing procedures to compute inflows into the most upstream reservoirs which in turn are routed through the reservoirs using standard techniques. Resulting outflows are combined with additional local inflows and carried downstream using appropriate time sequencing or routing procedures including unsteady flow routing. Figure 2.4-45 shows unit areas of the watershed upstream from Chickamauga Dam.

The runoff model differs from that used in the PSAR because of refinements made in some elements of the model during PMF studies for other nuclear plants and those made from information gained from the 1973 flood, the largest that has occurred for present reservoir conditions. Changes are identified when appropriate in the text. They include both additional and revised unit hydrographs and additional and revised unsteady flow stream course models.

Unit hydrographs were developed for each unit area 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]. The number of unit areas has been increased from 34 to 45. The differences include:

(1) Combining the two unit areas for Watauga River (Sugar Grove and Watauga local) into one unit area and dividing the Cherokee to Gate City area into two unit areas (Surgoinsville local and Cherokee local below Surgoinsville);

(2) Increasing the unit areas on the Clinch River from 1 to 11 and the Watts Bar local from 1 to 2;

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

(4) Combining the two unit areas above Fontana (Tuckasegee River at Bryson City and Oconaluftee River at Birdtown) into one unit area (Tuckasegee River at Bryson City) and;

(5) Combining the two unit areas above Ocoee No. 1 (Ocoee No. 1 and Ocoee No. 3) into one unit area (Ocoee No. 1 to Blue Ridge) and dividing the Chickamauga local unit area into two unit areas (Chickamauga local and lower Hiwassee).

In addition, 12 of the unit graphs have been revised. Figure 2.4-46, which contains 11 sheets, shows the unit hydrographs. Table 2.4-6 contains essential dimension data for each unit hydrograph and identification of those hydrographs which are new or revised.

Tributary reservoir routings, except for Tellico, were made using the Goodrich semigraphical method and flat pool storage conditions. Main river reservoir and Tellico routings were made using unsteady flow techniques. This differs from the PSAR in that:

(1) An unsteady flow model has been added for Fort Loudoun Tellico complex, and

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

Unsteady flow routings were computer solved with a mathematical model based on the equations of unsteady flow, Reference [6]. Boundary conditions prescribed were inflow hydrographs at the upstream boundary, local inflows, and headwater discharge relationships at the downstream boundary based upon normal operating rules, or based upon rated curves when geometry controlled.

The unsteady flow mathematical model for the 49.9-mile long Fort Loudoun Reservoir was divided into 24, 2.08-mile reaches. The model was verified at 3 gaged points in Fort Loudoun Reservoir using 1963 and 1973 flood data. The unsteady flow model was extended upstream on the French Broad and Holston Rivers to Douglas and Cherokee Dams, respectively. The French Broad and Holston River unsteady flow models were verified at one gaged point each (mile 7.4 and 5.5, respectively) using 1963 and 1973 flood data.

The Little Tennessee River was modeled from Tellico Dam, mile 0.3, through Tellico Reservoir to Chilhowee Dam at mile 33.6 and upstream to Fontana Dam at mile 61.0. The model for Tellico Reservoir to Chilhowee Dam was tested for adequacy by comparing its results with steady-state profiles at 1,000,000 and 2,000,000 cfs computed by the standard-step method. Minor decreases in conveyance in the unsteady flow model yielded good agreement. The average conveyance correction found necessary in the reach below Chilhowee Dam to make the unsteady flow model agree with the standard-step method was also used in the river reach from Chilhowee to Fontana Dam.

The Fort Loudoun and Tellico unsteady flow models were joined by a canal unsteady flow model. The canal was modeled with five equally-spaced cross-sections at 525-feet intervals for the 2100-foot long canal.

The unsteady flow routing model for the 72.4-mile-long Watts Bar Reservoir was divided into 34, 2.13-mile reaches. The Watts Bar model was verified at two gaged points within the reservoir using 1963 flood data.

The unsteady flow routing model for the total 58.9-mile-long Chickamauga Reservoir was divided into 28, 2.1-mile reaches. The Chickamauga Reservoir unsteady flow model was verified at four gaged points within the reservoir using 1973 flood data. This differs from the PSAR in that the 1973 flood was added for verification replacing the 1963 flood. The 1973 flood is the largest which has occurred since closure of South Holston Dam in 1950. Comparisons between observed and computed stages in Chickamauga Reservoir are shown in Figure 2.4-50.

It is impossible to verify the model with actual data approaching the magnitude of the PMF. The best remaining alternative was to compare the model elevations in a state of steady flow with elevations computed by the standard-step method. This was done

for steady flows ranging up to 1,500,000 cfs. An example shown by the rating curve of Figure 2.4-51 shows the good agreement.

The runoff model was verified by using it to reproduce the March 1963 and March 1973 floods. This differs from the PSAR in that the 1973 flood was added for verification replacing the 1957 flood. The 1973 flood is the largest which has occurred since closure of South Holston Dam in 1950. Observed volumes of precipitation excess were used in the verification. Comparisons between observed and computed outflows from Watts Bar and Chickamauga Dams for the 1963 and 1973 floods are shown in Figures 2.4-48 and 2.4-49, respectively.

From a study of the basic units of the predicting system and the systems response to alterations in various basic elements, it is concluded that the runoff model serves adequately and conservatively to determine maximum flood levels. This conclusion is based in part upon studies by others[7,8,9] and unpublished work of TVA which indicate the assumption of linearity in unit hydrographs, an important element of the system, is valid when they are developed from large, out-of-bank floods produced by major, basin-wide storms or will duplicate such floods.

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

(1) The total project period, or

(2) The 5-year period, 1972-1976, for those projects whose operating guides were changed in 1971.

Because of the wet years of 1972-1975 and the operating guide changes, median elevations were higher for 7 of the 13 tributary reservoirs where routing is involved.

Normal reservoir operating procedures were used in the antecedent storm. These used turbine and sluice discharge in the tributary reservoirs. Turbine discharges are not used in the main river reservoirs after large flood flows develop because head differentials are too small. Normal operating procedures were used in the principal storm except that turbine discharge was not used in either the tributary or main river dams. All gates were determined to be operable without failures during the flood. Gates on main river dams would be fully raised, thus requiring no additional operations, by the last day of the storm which is before the structures and access roads would be inundated.

Median initial reservoir elevations were used at the start of the storm sequence used to define the PMF to be consistent with statistical experience and to avoid unreasonable combinations of extreme events. As a result 55% of the total reserved system flood detention capacity above the plant was occupied at the start of the main flood. This is considered to he amply conservative. Neither the initial reservoir levels nor the operating rules would have significant effect on maximum flood discharges and elevations at the plant site because spillway capacities, and hence uncontrolled conditions, were reached early in the flood.

2.4.3.4 Probable Maximum Flood Flow

The analysis to determine the probable maximum flood (PMF) flow included evaluation of PMP over the total watershed with consideration of critical seasonal and areal variations. A comparison of candidate events is provided by Table 2.4-7.

The PMF discharge was determined to be 1,288,000 cfs resulting from the 7,980-square-mile storm centered at Bulls Gap. The peak discharge of 1,230,000 cfs resulting from the total basin, downstream centered, orographically fixed storm produced an equivalent peak stage.

The dam safety modifications to Fort Loudon - Tellico and Watts Bar enable these projects to safely pass the PMF. The West Saddle Dike at Watts Bar Dam would be overtopped and breached. Chickamauga Dam would be overtopped but was assumed not to fail (failure would reduce the flood level at the site). In the original analysis, Fort Loudoun-Tellico and Watts Bar upstream and Chickamauga Dam downstream would be overtopped and the earth embankments breached. The PMF hydrograph is shown in Figure 2.4-52. Rainfall and precipitation shown in Figure 2.4-52 are for the total basin storm.

Following is a more complete description of candidate situations examined.

The storm producing the PMF discharge is the 7,980-square-mile storm centered at Bulls Gap, Tennessee, 50 miles northeast of Knoxville, shown in Figure 2.4-42. The flood from this storm would overtop and breach the West Saddle Dike at Watts Bar Dam (Figure 2.4-53).

The storm producing the next highest discharge and the equivalent level as the PMF was the March storm producing the PMP on the 21,400-square-mile watershed above Chattanooga with downstream centered and orographically fixed isohyetal pattern shown in Figure 2.4-44, and is more completely described in Section 2.4.3.1. The flood would overtop and breach the earth West saddle dike at Watts Bar Dam upstream. Chickamauga Dam, downstream,would be over topped but was conservatively assumed not to fail.

The previous PMF evaluations also considered candidate situations involving upstream tributary dams, Douglas and Watauga. These two dams were overtopped and breached in their respective PMFs. These two situations were shown at that time to be non-governing. Dam safety modifications have since eliminated the potential failures of these dams; therefore, these two candidate situations have been eliminated.

Following is a more complete description of dam stability and earth embankment breaching analysis made to define PMF conditions.

Concrete Section Analysis

For concrete dam sections, comparisons were made between the original design headwater and tailwater levels and those that would prevail in the PMF. If the overturning moments and horizontal forces were not increased by more than about 20%, the structures were considered safe against failure. All upstream dams passed

this test except Douglas, Fort Loudoun, and Watts Bar. Original designs showed the spillway sections of these dams to be most vulnerable. These spillway sections were examined in further detail and judged to be stable.

Spillway Gates

During peak PMF conditions the radial spillway gates of Fort Loudoun and Watts Bar Dams will be wide open with flow over the gates and under the gates. For this condition both the static and dynamic load stresses in the main structural members of the gate will be less than the yield stress by a factor of 3. The stress in the trunnion pin is less than the allowable design stress by a factor of greater than 10. The trunnion pin is prevented from dislodgement by a key into the gate anchorage assembly and fitting into a slot in the pin.

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

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

Waterborne Objects

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

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

Should the barge approach the spillway portion broadside, two and possibly three bridge bents could be failed. For this condition the bridge would collapse on the barge and the barge would be grounded on the tops of the spillway piers. This would be probable because the approach velocity of the barge would be from 4 to 7 miles per hour and the bottom of the barge would be about 6 inches above the tops of the piers. For this condition the barge would be grounded before striking the spillway gates

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

Lock Gates

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

Embankment Breaching

The potential for embankment breaching was examined for all PMF candidate situations. In the 1998 reanalysis the only embankment failure would be the West Saddle Dike at Watts Bar Dam. Chickamauga Dam, 57 miles downstream of the plant, would be overtopped but was assumed not to fail. This is conservative as failure would slightly lower flood elevations at the plant.

Figure 2.4-47b shows the headwater and tailwater discharge relationships for Fort Loudon Dam as modified. Figures 2.4-47c and 2.4-55 show these relationships for Tellico and Watts Bar Dams, respectively.

The adopted relationship to compute the rate of erosion in an earth dam failure is that developed and used by the Bureau of Reclamation in connection with its safety of dams program[11]. The expression below relates the volume of eroded fill material to the volume of water flowing through the breach. The equation is:

$$\frac{Qsoil}{Qwater} = Ke^{-x}$$

where

Q soil = Volume of soil eroded in each time period

Q water = Volume of water discharged each time period

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

e = Base of natural logarithm system

$$X = \frac{b}{H}\tan\phi d$$

where

b = Base length of overflow channel at any given time

H = Hydraulic head at any given time

φ_d = Developed angle of friction of soil material. A conservative value of 13 degrees was adopted for materials in the dams investigated.

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

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

Watts Bar West Saddle Dike Failure

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

Given the hour of failure, the peak discharge was determined based upon the headwater and tailwater depths at that time. Unsteady flow routing techniques were used to define the rest of the outflow hydrograph. This was accomplished using headwater discharge relationships at the dam treated as the downstream boundary to the Watts Bar Reservoir routing reach.

Some verification for the breaching computational procedures illustrated above was obtained by comparison and actual failures reported in the literature and in informal discussion with hydrologic engineers. These reports show that overtopped earth embankments do not necessarily fail. Earth embankments have sustained overtopping of several feet for several hours before failure occurred. An extreme example is Oros earth dam in Brazil[12] which was overtopped to a depth of approximately 2.6 feet along a 2,000-foot length for 12 hours before breaching began. Once an earth embankment is breached, failure tends to progress rapidly, however. How rapidly depends upon the material and headwater depths during failure. Complete failures computed in this and other studies have varied from about one half to 6 hours after initial breaching. This is consistent with actual failures.

Chickamauga Embankment Failure

In the original analysis the failure of earth embankments at Chickamauga Dam, 57 miles downstream from Watts Bar Nuclear Plant reduced flood levels at the plant by 0.2 foot. Future embankment improvements are planned for Chickamauga Dam which might eventually invalidate assumption of failure. Therefore, the dam was assumed not to fail in determining flood elevations for the plant. This assumption is conservative.

Wave Front From Watts Bar Breaching

This subsection describes analysis of the wave from resulting from the embankment failure of Watts Bar Dam. Embankment improvements made by the Dam Safety Program ensure that failure in the PMF will not occur. This subsection is retained for historical purposes.

Because Watts Bar Nuclear Plant is located only 1.9 miles downstream from Watts Bar Dam, the magnitude and shape of the wave front resulting from the sudden failure of the Watts Bar embankment during the PMF was examined in detail. The analysis consisted of determining the magnitude of the wave resulting from the embankment failure (1) immediately after failure, (2) after traveling downstream 8500 feet and striking a ridge on the left bank, and (3) reflected from the ridge directed toward the plant on the right bank.

The water levels upstream and downstream from the earth embankment for the conditions which exist just prior to and immediately following embankment failure are shown schematically on Figure 2.4-61. Immediately after failure of the 750-foot wide portion of the embankment a bore will develop. The relative height of the bore has been defined by Stoker[31] by the following functional relationship, which is shown graphically on Figure 2.4-62.

$$\frac{h_2 - h_0}{h_1} = \psi(h_1, h_0)$$

In which, h_0, h_1, and h_2, are defined in Figure 2.4-61.

For the initial conditions shown schematically on Figure 2.4-61.

$$\frac{h_0}{h_1} = \frac{727 - 700}{762 - 700} = \frac{27}{62} = 0.435$$

Entering Figure 2.4-62 with h_0/h_1, = 0.435

$$\frac{h_2 - h_0}{h_1} = 0.250$$

Solving this equation for the bore height above tailwater, (h_2-h_0) gives:

$$h_2 - h_0 = 0.250h_1 = 0.250x62 = 15.5feet$$

Solving for the depth behind the bore, h_2, gives:

$$h_2 = 0.250h_1 + h_0 = 0.250x62 + 27 = 42.5feet$$

The initial bore velocity per Reference [31], can be computed from:

$$\varepsilon' = V_0 + \frac{gh_2}{h_0}\left(\frac{h_0 + h_2}{2}\right)$$

$$\varepsilon' = 12.0 + (32.2)\left(\frac{42.5}{27}\right)\left(\frac{27 + 42.5}{2}\right) = 12.0 + 42.0 = 54.0fps$$

In which,

$$V_0 = \frac{Qbefore\ failure}{Tailwater\ area} = \frac{971000}{3000 \times 27} = 12fps$$

Therefore, immediately after the failure of the embankment, the height of the bore, will be 15.5 feet above the initial tailwater depth, its width will be 750 feet at the dam and it will move toward Blalock Ridge with an initial velocity of 54 feet per second.

The bore is conservatively estimated to spread laterally at 10° as it moves out from the dam, as shown on Figure 2.4-63. The height of the bore would be reduced by the 10° expansion from 15.5 feet at the dam to 3.1 feet at Blalock Ridge. The ridge is sloped and heavily wooded and would absorb a large percentage of the incident bore energy. Since wave energy is proportional to the square of the wave amplitude A,[32] the following expression can be written:

$$A_i^2 = A_r^2 + A_a^2$$

in which subscripts i, r, and a, refer to incident, reflected and absorbed components, respectively. This equation may be written as:

$$1 = \frac{(A_r)^2}{A_i} + \frac{(A_a)^2}{A_i}$$

Using $C_r = A_r/A_i$ and $C_a = A_a/A_i$ where C_r is the reflection coefficient and C_a is the absorption coefficient, the expression becomes:

$$C_r = \sqrt{1 - C_a^2}$$

The following table was computed using this expression.

$^*C_a^2$	$^*C_r^2$	$C_a=A_a/A_i$	$C_r=A_r/A_i$
0	1	0	1
0.1	0.9	0.32	0.95
0.2	0.8	0.45	0.89
0.3	0.7	0.55	0.84
0.4	0.6	0.63	0.77
0.5	O.5	0.71	0.71
0.6	0.4	0.77	0.63
0.7	0.3	0.84	0.55
0.8	0.2	0.89	0.45
0.9	0.1	0.95	0.32
1.0	0	1	0

Where:

$^*C_a^2$ is the fraction of wave energy absorbed

$^*C_r^2$ is the fraction of wave energy reflected

Values of C ranging from about 0.16 up to 0.72 have been reported in the literature for tree-like materials, with most values being nearer the 0.16 value[33]. Assuming 50% of the bore energy would be absorbed by the ridge and 50% reflected, gives from the preceding table

$$C_r = \frac{A_r}{A_i} = 0.71$$

with A_i = 3.1 feet, the reflected bore height would be:

$$A_r = 2.2 \text{ feet}$$

In addition to the conservative assumption regarding the absorption of bore energy, the following additional factors which would further reduce the reflected wave height have been neglected. These factors are:

(1) The shape of the ridge is such that it does not parallel the wave front, therefore only a portion of the wave front would be directed toward the plant site. Considerable dispersion of the wave front would result from the curved shape of Blalock ridge.

(2) The ridge is sloped, therefore run-off would further reduce the reflected wave height.

(3) After striking Blalock Ridge no further wave expansion has been considered in this analysis.

Thus, a wave height of about 2.2 feet is conservatively estimated to be reflected from Blalock Ridge and directed toward the plant site. This compares with the computed wave height in the PSAR of 2.0 feet. This wave would travel at about 30 feet per second on the flood plain as it approaches the plant site.

The reflected wave would arrive at the plant site in about 5 to 10 minutes following embankment failure (embankment failure was postulated to occur at 11:30 on March 24). At the time the wave arrives at the plant site, the water level is about elevation 728 as shown in FSAR Figure 2.4-64 producing a water level at the plant site of El 730.2. The maximum PMF level occurs some one and one-half days later.

Forces resulting from the 2.2-foot bore were evaluated for each of the safety-related structures required to remain functional and maintain their structural integrity during the probable maximum flood. The design criteria for the structures described in Section 3.8 were not exceeded.

2.4.3.5 Water Level Determinations

The maximum plant site flood elevation 734.9 is produced by the 7980 square mile storm. The less critical 21,400 square mile storm would produce elevation 734.7 at the plant site. The flood elevation hydrographis shown in Figure 2.4-57. Elevations were computed concurrently with discharges for the site using the unsteady flow reservoir model. Figure 2.4-65 shows the PMF profile together with the regulated maximum known flood and thalweg profiles along a 16-mile reach of the Chickamauga Reservoir which encompasses the plant location.

The third candidate situation considered, failure of Douglas Dam during its PMF, was shown in the original analysis to produce a flood crest at the plant site approximately 4.4 feet below the controlling PMF event. Dam safety modifications to Douglas Dam involved raising the height of the dam to prevent overtopping in the PMF and this eliminates failure of Douglas as a candidate situation.

The elevations from the potential controlling PMF events are compared in Table 2.4-7.

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 2.4-57). The day of occurrence would be in the month of March or possibly the first week in April.

Figure 2.4-98 shows the main plant general grading plan. The diesel generator buildings to the north and the pumping station to the east 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 well above the maximum computed elevation including wind wave runup. The pumping station is shielded from direct wave action on all sides except to the south by either buildings, earth embankments, or the cooling towers. The maximum effective fetch of 1.3 miles occurs from both the southwest and northeast directions (Figure 2.4-67). This allows for the sheltering effect of several hills on the south riverbank which become islands at maximum flood levels.

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 March 30-minute wind data which was used directly in subsequent wind wave calculations were adjusted to 30 feet by the equation:

$$V_{30} = V_z(30/Z)^{1/7}$$

where:

V_{30} = wind speed at 30 feet

V_Z = wind speed at height Z above the ground

The adjusted 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 2.4-66 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.

Wind wave calculations call for a 28-minute sustained wind. There is, however, no significant difference between a 2-year, 30-minute wind velocity and a 2-year, 28-minute wind velocity. Thus, the 2-year, 30-minute, 21 mile per hour wind velocity was used to compute wind waves.

Computation of wind waves used the procedures of the Corps Of Engineers[14]. The critical direction for the PMF elevations is from the southwest with an effective fetch of 1.3 miles as shown in Figure 2.4-67. For a 28-minute sustained 21-mile-per-hour wind, 99.6% of the waves approaching the plant would be less than 2.0 feet high, crest to trough, resulting in maximum water elevation of 736.2.

At the diesel generator building, corresponding runup on the earth embankment with a 4:1 slope would be 2.0 feet, reaching elevation 736.9. The runup on the south wall of the pumping station would be to elevation 736.9.

Wind wave setup is not a problem since the wind direction is opposite to the flow of the river. 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 Rainflow 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 Loudon 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 Loudon, the postulated SSE failure of Fontana and Douglas, the postulated SSE failure of Norris, and the postulated SSE failure of Fontana, Fort Loudon, 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 nonoverflow 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 nonoverflow 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 over topped 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 Figures 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 Loudon 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 Loudon 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 Loudon 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 .14 g at Douglas, .09 g at Fontana, .07 g at Cherokee, .05 g at Norris, .06 g at Fort Loudoun and Tellico, and .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 2.4-8, 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 watermilfoil, 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 Management 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]

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

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 x 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 2.4-10 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].

$$v = \frac{K\ dh/dl}{0}$$

where v = mean velocity, ft/day;

K = hydraulic conductivity = 48 ft/day;

dh/dl = hydraulicgradient = .01

0 = porosity = 0.15 (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 caused by a 21 mph overland wind; this is 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)	734.9
DBF on lake	736.2
DBF Runup on 4:1 sloped surfaces	736.9
DBF Runup on vertical walls with base elevation 728.0	736.9
DBF Surge level within flooded structures	735.4

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 Section 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	
	for	for
Season	Stage I shutdown	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 2.4-8 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 criticaldams 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 Sage 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.

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Table 2.4-1 Facts About Major TVA Dams and Reservoirs

TABLE 2.4-1

FACTS ABOUT MAJOR TVA DAMS AND RESERVOIRS

County	Nearby City	Type of Dam (g)	Concrete in dam, lock, & pwrhse (cu. yds.)	Earth and/or Rock Fill cu. yds.	Max. Height (Feet) (a)	Length (Feet)	Drainage area above dam (sq. mi.)	Length of Lake (miles)	Max. Width of Lake (miles)	Area of Lake at Full Pool (acres)	Shore line at Full Pool (miles)	Lake Elevation (feet above sea level) Ordinary Minimum	Top of Gates	Full Pool (m)	Lake Volume (acre-feet) Ordinary Minimum Elevation	Top of Gates Elevation	Useful Controlled Storage (Ac.-FL)	Construction Started	Closure	Constr. Completed (1st unit on line)	Cost of Plant in Service 6-30-70 (d) (millions)	Ultimate Generating Capacity KW and No. of Units ( )
Marshall (b) Livingston	Paducah	CGE	1,356,000	5,582,100	206	8,422	40,200	184.3	2.5	160,300	2,380	354	375	359	2,121,000	6,129,000	4,008,000	7-1-38	8-30-44	9-14-44	$118.0	170,000(5)
Hardin	Savannah	"	679,100	3,081,000	113	7,715	32,820	52.7	1.5	43,100	496	408	418	414	688,000	1,195,000	417,000	3-8-35	2-8-38	6-23-38	45.7	216,000(6)
auderdale (b) Colbert	Sheffield Florence	CG	1,729,400	0	137	4,535	30,750	15.5	1.6	15,500	154	504.5	507.88	507.5	582,000	641,000	59,000	4-11-18	4-14-24	9-12-25	107.6	629,810(21)
auderdale (b) Lawrence	Town Creek	"	1,100,000	0	72	6,342	29,590	74.1	2.8	67,100	1,063	550	556.3	556	720,000	1,071,000	351,000	11-21-33	10-3-36	11-9-36	87.5	356,400(11)
Marshall	Guntersville	CGE	308,640	874,900	94	3,979	24,450	75.7	2.5	67,900	949	593	595.44	595	879,700	1,052,000	172,300	12-4-35	1-16-39	8-1-39	51.2	97,200(4)
Marion	So. Pittsburg	"	546,900	989,200	83	3,767	21,870	46.3	2.7	10,900	192	632	635	634	221,600	254,600	33,000	4-1-64	12-14-67	2-20-68	74.9	97,200(4)
Hamilton	Chattanooga	"	506,390	2,793,500	129	5,800	20,790	58.9	1.7	35,400	810	675	685.44	682.5	392,000	739,000	347,000	1-13-36	1-15-40	3-4-40	42.1	108,000(4)
Meigs (b) Rhea	Spring City	"	480,200	1,210,000	112	2,960	17,310	72.4	1.3	39,000	783	735	745	741	796,000	1,175,000	379,000	7-1-39	1-1-42	2-11-42	35.6	150,000(5)
Loudon	Lenoir City	"	586,700	3,594,000	122	4,190	9,550	55.0	0.7	14,600	360	807	815	813	282,000	393,000	111,000	7-8-40	8-2-43	11-9-43	42.4	131,190(4)
Franklin	Winchester	E & R	95,450	3,216,000	170	1,470	529	34	—	10,700	246	860	895	888	294,000	617,000	323,000	3-28-66	12- -70	11- -71	49.5	45,000(1)
Cherokee (c)	Farner	CG	237,806	0	150	1,308	1,018	9.8	0.3	1,100	31	1,272	1,280	1,280	48,600	57,500	8,900	7-17-41	2-14-43	9-22-43	24.0	75,000(2)
Cherokee	Murphy	"	800,556	0	307	1,376	968	22	0.6	6,090	180	1,415	1,526.5	1,524.5	71,800	434,000	362,200	7-15-36	2-8-40	5-21-40	24.4	117,100(2)(h)
Clay	Hayesville	E	25,700	2,348,000	144	2,850	189	13	1.5	7,050	132	1,860	1,928	1,927	18,400	240,500	222,100	7-17-41	2-12-42	12-9-54	9.1	10,000(1)
Polk	Benton	CG	160,000	0	135	840	595	7.5	—	1,890	18	816.9	837.65	837.65	53,500	87,300	33,800	8- -10	12-15-11	1-10-12	3.0	18,000(3)
Polk	Benton	RFT	0	0	30	450	516	—	—	—	—	—	1,115	1,115	—	—	—	5- -12	—	10- -13	3.0	21,000(2)
Polk	Ducktown	CG	82,500	82,000	110	612	496	7	0.3	621	24	1,113	1,435	1,435	790	4,650	3,860	7-17-41	8-15-42	4-30-43	9.0	27,000(1)
Fannin	Blue Ridge	E	—	1,500,000	167	1,000	232	10	—	3,290	60	1,590	1,691	1,690	12,500	196,500	184,000	11- -25(e)	12-6-30	7- -31	5.5	20,000(1)
Union	Blairsville	E & R	21,700	1,552,200	184	2,300	214	20	1.1	4,180	106	1,690	1,779	1,779	12,700	174,300	161,600	7-17-41	1-24-42	1-10-56	8.1	15,000(1)
Loudon (b) Roane	Oak Ridge	CG	246,800	0	103	1,020	3,343	44	0.8	5,690	173	790	796	795	94,500	126,000	31,500	9-6-60	5-1-63	7-3-64	36.2	72,000(2)
Anderson (b) Campbell	Knoxville	CGE	1,002,300	181,700	265	1,860	2,912	72	1.2	34,200	800	930	1,034	1,020	290,000	2,555,000	2,265,000	10-1-33	3-4-36	7-28-36	33.3	108,800(2)
Loudon	Lenoir City	CGE	80,000	2,000,000	108	3,238	2,627	33.2	1.3	16,500	310	807	815	813	321,300	447,300	126,000	3-15-67	1- -77	(i)	69.0	(i)
Graham (b) Swain	Robbinsville	CG	2,815,500	760,600	480	2,365	1,571	29	0.6	10,610	248	1,525	1,710	1,708	295,000	1,448,000	1,153,000	1-1-42	11-7-44	1-20-45	78.6	225,000(3)
Sevier	Sevierville	CGE	556,390	127,900	202	1,705	4,541	43.1	1.5	30,400	555	920	1,002	1,000	84,500	1,490,000	1,405,500	2-2-42	2-19-43	3-21-43	16.9	112,000(4)
Jefferson (b) Grainger	Jefferson City	"	694,200	3,384,100	175	6,760	3,428	59	1.5	30,300	414	980	1,075	1,073	83,600	1,544,000	1,460,400	8-1-40	12-5-41	1-16-42	36.6	120,000(4)
Sullivan	Kingsport	CG	72,500	0	95	737	1,903	10.3	0.25	872	37	1,258	1,263	1,263	22,700	26,900	4,200	5-14-51	10-27-53	12-5-53	12.3	36,000(2)
Sullivan (b) Washington	Johnson City Kingsport	CGE	198,400	714,000	160	1,532	1,810	17.3	0.5	4,400	130	1,330	1,385	1,385	45,000	193,400	148,400	8-29-50	12-16-52	5-16-53	27.8	75,000(3)
Sullivan	Bristol, Va.-Tenn.	E & R	97,500	5,897,100	285	1,600	703	24.3	1.3	7,580	168	1,616	1,712	1,729	121,100	764,000	642,600	8-4-47(f)	11-20-50	2-13-51	31.4	35,000(1)
Carter	Elizabethton	"	80,400	3,197,800	318	900	468	16.7	0.8	6,430	106	1,815	1,975	1,959	52,300	677,000	624,700	7-22-16(f)	12-1-48	5-30-49	32.5	50,000(2)
Warren (b) White	Rock Island	CG	—	—	92	800	1,675	22	—	2,100	120	780	805.30	805.30	11,600	51,600	37,000	-15	12-8-16	-16	8.2	31,860(2)
										638,353	11,070				8,621,190	23,732,350	15,110,860					3,138,090(105)
Marion	Chattanooga	E & R	125,000	8,200,000	230	8,000	—	—	—	520	—	1,530	—	1,672	2,000	37,800	35,800	7-8-70	1- -71	1- -71	155.0	1,530,000(1)

a. Foundation to operating deck.
b. River is county line.
c. Powerhouse is in Polk County, Tennessee.
d. Original construction or acquisition cost, including switchyard, as adjusted by subsequent additions, retirements, and reclassifications. Includes estimated costs of projects under construction.
e. Construction discontinued early in 1926; resumed in March 1929.
f. Initial construction started February 16, 1912; temporarily discontinued to conserve critical materials during war.
g. Abbreviations: CG-Concrete gravity dams. CGE-Concrete gravity with earth embankments. E-Earth fill. E&R-Earth and rock fill. RFT-Rock-filled timber.
h. Unit 2 is a reversible pump-turbine.
i. Tellico project has no lock or powerhouse. Streamflow through navigable to Fort Loudoun Reservoir will increase average annual energy throu Loudoun powerhouse by 200 million kwh.
j. cost and quantity data estimated.
k. Nickajack Dam replaced the old Hales Bar Dam 6 miles upstream.
l. Acquired: Wilson by transfer from U. S. Corps of Engineers in 193 No. 1, Ocoee No. 2, Blue Ridge, and Great Falls by purchase from TE 1939. Subsequent to acquisition, TVA heightened and installed additio at Wilson.
m. Full Pool Elevation is the normal upper level to which the reservoirs filled. Where storage space is available above this level, additional fil be made as needed for flood control.
n. Construction of Nickajack main lock limited to underwater portion; pletion later.

at TVA dams and several steam plant projects are available
r list.

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Table 2.4-2 Facts About Non-TVA Dam and Reservoir Projects

ALCOA Projects	River	Drainage Area, Sq. Miles	Miles Above Mouth	Maximum Height, Feet	Length, Feet	Area of Lake, Acres	Length of Lake Miles	Total[a] Storage, Acre-Feet	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
Natahala	Natahala	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 Tuckasegee	East Fork Tuckasegee	24.9	10.9	140	385	39	1.4	1,797	1952
	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

a. Volume at top of gates.

Table 2.4-3 Flood Detention Capacity TVA Projects Above Watts Bar Nuclear Plant

Storage Reserved for Flood Control - Acre Feet

Project	January 1	March 15	Summer
Tributary			
Douglas	1,252,000	1,019,000	67,000
Watauga	223,000	154,800	108,300
South Holston	289,800	221,000	106,500
Boone	91,700	40,400	4,300
Cherokee	1,014,000	812,000	59,000
Fontana	737,000	583,000	23,000
Tellico	126,000	126,000	32,700
Norris	1,470,000	1,113,000	512,000
Main River			
Fort Loudoun	111,000	111,000	30,000
Watts Bar	379,000	379,000	165,000
Total	5,693,500	4,559,200	1,107,800

Table 2.4-4 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 271.0)

Plant Name	Use (MGD)	Location	(Bank)	Approximate Distance From Site (River Miles)	Type Supply
Watts Bar Dam	#	TRM 529.9		1.1 (Upstream)	Industrial
Watts Bar Steam Plant	##	TRM 529.9	R	1.1 (Upstream)	Industrial
Watts Bar Nuclear Plant	###	TRM 528.8	R	0	Industrial
City of Dayton	1.78	TRM 503.8	R	25 (Downstream)	Municipal
Sequoyah Nuclear Plant	1615.68	TRM 483.6	R	45.2 (Downstream)	Industrial
East Side Utility	5.00	TRM 473	L	55.8 (Downstream)	Municipal
Chickamagua Dam	#	TRM 471		57.8 (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 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 2.4-5 Probable Maximum Storm Rainfall and Precipitation Excess
(Page 1 of 2)

Index No.	Area	Antecedent Storm Rain, Inches	Antecedent Storm P_e,[a] Inches	Main Storm Rain, Inches	Main Storm P_e,[b] Inches
1	Asheville	6.44	2.99	17.40	14.72
2	Newport, French Broad	6.44	4.04	18.50	16.51
3	Newport, Pigeon	6.44	4.04	19.30	17.31
4	Embreeville	6.44	4.04	15.10	13.11
5	Nolichucky Local	6.44	4.04	15.50	13.51
6	Douglas Local	6.44	4.86	17.10	15.88
7	Little Pigeon River	6.44	4.04	20.90	18.91
8	French Broad Local	6.44	4.19	18.60	16.81
9	South Holston	6.44	4.52	12.30	10.70
10	Watauga	6.44	4.04	13.30	11.31
11	Boone Local	6.44	4.04	14.10	12.11
12	Fort Patrick Henry	6.44	4.86	14.40	13.18
13	Gate City	6.44	4.83	12.30	11.08
14	Surgoinsville Local	6.44	4.86	14.60	13.38
15	Cherokee Local below Surgoinsville	6.44	4.86	15.80	14.58
16	Holston River Local	6.44	4.52	17.10	15.50
17	Little River	6.44	4.04	21.50	19.51
18	Fort Loudoun Local	6.44	4.04	17.60	15.61
19	Needmore	6.44	2.99	21.20	18.52
20	Nantahala	6.44	2.99	21.50	18.82
21	Bryson City	6.44	2.99	19.10	16.42
22	Fontana Local	6.44	2.99	20.70	18.02
23	Little Tennessee Local - Fontana to Chilhowee Dam	6.44	2.99	24.00	21.32
24	Little Tennessee Local - Chilhowee to Tellico Dam	6.44	4.04	21.00	19.01
25	Watts Bar Local above Clinch River	6.44	4.04	15.80	13.81
26	Norris Dam	6.44	4.86	13.80	12.58
27	Coal Creek	6.44	4.52	14.60	13.19
28	Clinch Local	6.44	4.52	14.90	13.49
29	Hinds Creek	6.44	4.52	15.30	13.89
30	Bullrun Creek	6.44	4.68	15.70	14.29

Table 2.4-5 Probable Maximum Storm Rainfall and Precipitation Excess (Continued)
(Page 2 of 2)

Index No.	Area	Antecedent Storm Rain, Inches	Antecedent Storm P_e,[a] Inches	Main Storm Rain, Inches	Main Storm P_e,[b] Inches
31	Beaver Creek	6.44	4.52	16.10	14.69
32	Clinch Local (5 areas)	6.44	4.52	15.30	13.89
33	Local above mile 16	6.44	4.52	15.30	13.89
34	Poplar Creek	6.44	4.52	14.90	13.49
35	Emory River	6.44	4.52	13.10	11.69
36	Local Area at Mouth	6.44	4.52	14.90	13.49
37	Watts Bar Local below Clinch River	6.44	4.52	14.40	12.99
38	Chatuge	6.44	2.99	21.40	18.72
39	Nottely	6.44	2.99	19.10	16.42
40	Hiwassee Local	6.44	2.99	18.90	16.22
41	Apalachia	6.44	2.99	17.90	15.22
42	Blue Ridge	6.44	2.99	22.10	19.42
43	Ocoee No. 1, Blue Ridge to Ocoee No. 1	6.44	4.04	18.30	16.31
44	Lower Hiwassee	6.44	4.19	15.20	13.41
45	Chickamauga Local	6.44	4.52	14.50	13.09
	Average above Watts Bar Dam	6.44	4.20	16.34	14.56
	Chickamauga Dam	6.44	4.14	16.46	14.63

a. Adopted API prior to antecedent storm, 1.0 inch.

b. Computed API prior to main storm, 3.65 inches.

Table 2.4-6 Unit Hydrograph Data
(Page 1 of 2)

Unit Area No.	Name	Drainage Area, Sq. Miles	Duration, Hours	Q_p	C_p	T_p	W_{50}	W_{75}	T_B
1	French Broad River at Asheville	945	6	15,000	.27	14	35	12	166
2	French Broad River Newport to Asheville[a]	913	6	35,000	.53	12	12	7	108
3	Pigeon River at Newport[a]	666	6	26,600	.56	12	11	6	78
4	Nolichucky River at Embreeville	805	6	27,300	.58	14	14	9	82
5	Nolichucky Local	378	6	10,600	.40	12	16	9	87
6	Douglas Local[a]	832	6	47,930	.27	6	8	6	60
7	Little Pigeon River at Sevierville	353	6	15,600	.62	12	10	6	102
8	French Broad River Local[b]	207	6	7,500	.51	12	11	8	60
9	South Holston	703	6	16,000	.53	18	24	17	100
10	Watauga[b]	468	6	17,700	.53	12	13	7	84
11	Boone Local[a]	669	6	22,890	.16	6	13	8	90
12	Fort Patrick Henry	63	6	3,200	.40	8	8	6	64
13	North Fork Holston River	672	6	12,260	.60	24	33	25	108
14	Surgoinsville Local[b]	299	6	10,280	.48	12	13	9	66
15	Cherokee Local below Surgoinsville[b]	554	6	18,750	.48	12	14	7	66
16	Holston River Local[b]	289	6	6,800	.55	18	22	15	96
17	Little River at Mouth[b]	379	4	11,730	.68	16	14	8	96
18	Fort Loudoun Local[b]	323	6	20,000	.29	6	10	6	36
19	Little Tennessee River at Needmore[a]	436	6	9,130	.49	18	23	12	126
20	Nantahala[a]	91	6	3,770	.45	10	12	7	70
21	Tuckasegee River at Bryson City [a]	655	6	26,000	.43	10	12	7	58
22	Fontana local[a]	389	6	16,350	.46	10	9	5	94
23	Little Tennessee River Local, Fontana-Chilhowee[b]	406	6	16,900	.58	12	9	5	84
24	Little Tennessee River Local, Chilhowee-Tellico Dam[b]	650	6	17,000	.61	18	21	11	72

Table 2.4-6 Unit Hydrograph Data (Continued)
(Page 2 of 2)

Unit Area No.	Name	Drainage Area, Sq. Miles	Duration, Hours	Q_p	C_p	T_p	W_{50}	W_{75}	T_B
25	Watts Bar Local above Clinch River[b]	293	6	11,300	.30	8	9	7	84
26	Norris Dam	2912	6	43,300	.07	6	15	8	118
27	Coal Creek[b]	36.6	2	2,150	.64	8	9	5	40
28	Clinch Local[b]	22.25	2	1,350	.10	2	8	5	34
29	Winds Creek[b]	66.4	2	2,620	.68	9	7	5	54
30	Bullrun Creek[b]	104	2	2,400	.47	14	21	14	84
31	Beaver Creek[b]	90.5	2	2,600	.58	14	14	10	88
32	Clinch Locals (5 areas)[b]	111.25	2	1,350	.10	2	8	5	34
33	Local above mi. 16[b]	37	2	4,490	.95	6	4	3	46
34	Poplar Creek[b]	136	2	2,800	.61	20	25	13	88
35	Emory River at Mouth[b]	865	6	34,000	.37	9	13	8	87
36	Local area at Mouth[b]	32	2	3,870	.95	6	3	2	46
37	Watts Bar Local below Clinch River[b]	427	6	16,300	.36	9	9	7	84
38	Chatuge Dam[a]	189	6	13,570	.34	6	6	5	54
39	Nottely Dam[a]	215	6	13,500	.29	6	5	4	80
40	Hiwassee Local	564	6	13,800	.36	12	18	12	124
41	Applachia Local	50	6	2,900	.54	9	6	4	90
42	Blue Ridge Dam[a]	232	6	11,920	.24	6	7	4	54
43	Ocoee No. 1 to Blue Ridge[b]	363	6	17,000	.37	8	11	7	36
44	Lower Hiwassee[b]	1087	6	32,500	.93	23	16	10	136
45	Chickamauga Local[b]	780	6	32,000	.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

a. Revised
b. New

Table 2.4-7 Flood Flow And Elevation Summary

		Discharge, CFS	Elevation
1.	Storm producing PMP depths on the 7,890-square-mile watershed with center at Bulls Gap[a]	1,288,000	734.9
2.	Storm producing PMP depths on the 21,400-square-mile watershed above Chattanooga[a]	1,230,000	734.7

a. Includes failure of the West Saddle Dike at Watts Bar Dam.

Table 2.4-8 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 2.4-9 Sheets 1 and 2 Deleted By Amendment 63

Table 2.4-10 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 2.4-10 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 2.4-10 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	Longitude	Depth	Estimated Elevation Ground (feet)	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 2.4-10 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	Location Longitude	Depth	Estimated Elevation Ground (feet)	Estimated Elevation Water Surface (feet)	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 2.4-11 Deleted by Amendment 83

Table 2.4-12 Deleted by Amendment 92

Table 2.4-13 Deleted by Amendment 92

Table 2.4-14 Weir Length Description and Coefficients of Discharge For Areas 3 and 4
(Sheet 1 of 1)

Watershed Area	Weir Parameters: 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 2.4-15 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 2.4-16 Dam Safety Modification Status (Hydrologic)

Dam	*Dam Modification	Year Completed
Main River Dams		
Fort Loudon-Tellico	Fort Loudon Dam was raised 3.25 feet with a concrete wall to elevation 833.25. A 2000-foot uncontrolled spillway with crest at Elevation 817 was added at Tellico Dam.	1989
Watts Bar	Embankment of main dam was raised 10 feet with earthfill/concrete wall to Elevation 767. West Saddle Dike was not modified. Top of Saddle Dike remains at Elevation 757.	1997
Nickajack	South embankment was raised 5 feet with earthfill/concrete wall to Elevation 657. A 1900-foot roller-compacted concrete overflow dam with top at Elevation 634 was added below the north embankment.	1992
Guntersville	Embankments were raised 7.5 feet with earthfill and concrete walls to Elevation 617.5.	1996
Tributary Dams		
Little Bear Creek	Embankment was raised 4.5 feet.	1998
Beech	Embankment was raised 4.5 feet with earthfill to Elevation 475.5.	1992
Blue Ridge	Three (3) additional spillway bays were added in 1982. Embankment was raised 7 feet with earthfill/concrete wall to Elevation 1713, and a 320-foot uncontrolled spillway with crest at Elevation 1691 was added in 1995.	1995
Boone	Embankment was raised 8.5 feet with earthfill to Elevation 1408.5.	1984
Cedar Creek	Embankment was raised 5.5 feet with concrete wall to Elevation 605.	1997
Chatuge	Embankment was raised 6.5 feet with earthfill to Elevation 1946.5.	1986
Cherokee	A portion (600 feet) of the non-overflow dam was raised 7.75 feet to Elevation 1089.75.	1982
Douglas	A portion of the non-overflow dam was raised 13.5 feet to Elevation 1022.5, and eight saddle dams were raised 6.5 feet with earthfill to Elevation 1023.5.	1988
Nottely	Embankment was raised 13.5 feet with rockfill to Elevation 1807.5.	1988
Upper Bear Creek	Embankment was raised 4 feet with concrete wall to Elevation 817.	1997
Watauga	Embankment was raised 10 feet with rockfill to Elevation 2012.	1983
Fontana	Dam post-tensioned	1988
Melton Hill	Dam post-tensioned	1988

*These dam safety modifications enable these projects to safely pass the probable maximum flood (PMF).
Note: Plans are to armor the embankment at Chickamauga and Bear Creek Dams to permit overtopping.

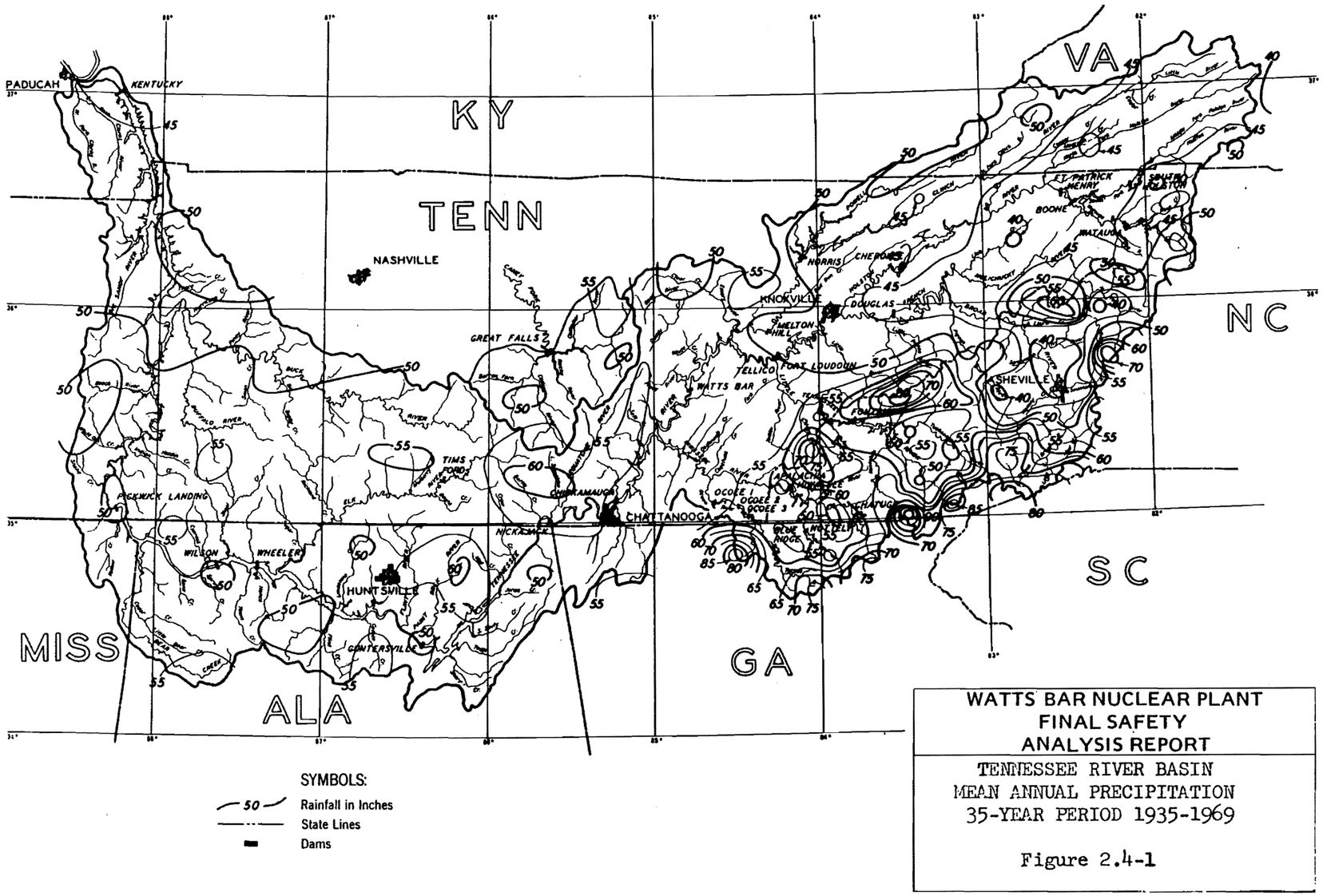


Figure 2.4-1 Tennessee River Basin Mean Annual Precipitation 35-Year Period 1935-1969

Figure 2.4-2 General Plan Elevation & Sections Watts Bar Hydro Project

Figure 2.4-3 General Plan Elevations & Sections Fort Loudon Project

Figure 2.4-3a General Plan Elevations and Sections Fort Loudon Project

Figure 2.4-4 Norris Dam Plan-Elevations and Sections

Figure 2.4-5 General Plan Elevations & Sections - Melton Hill Project

Figure 2.4-6 General Plan Elevation And Sections - Fontana Project

Figure 2.4-7 General Plan Elevation & Sections - Douglas Project

Figure 2.4-8 General Plan Elevations & Sections - Cherokee Project

Figure 2.4-9 General Plan Elevation & Sections - Fort Patrick Henry Project

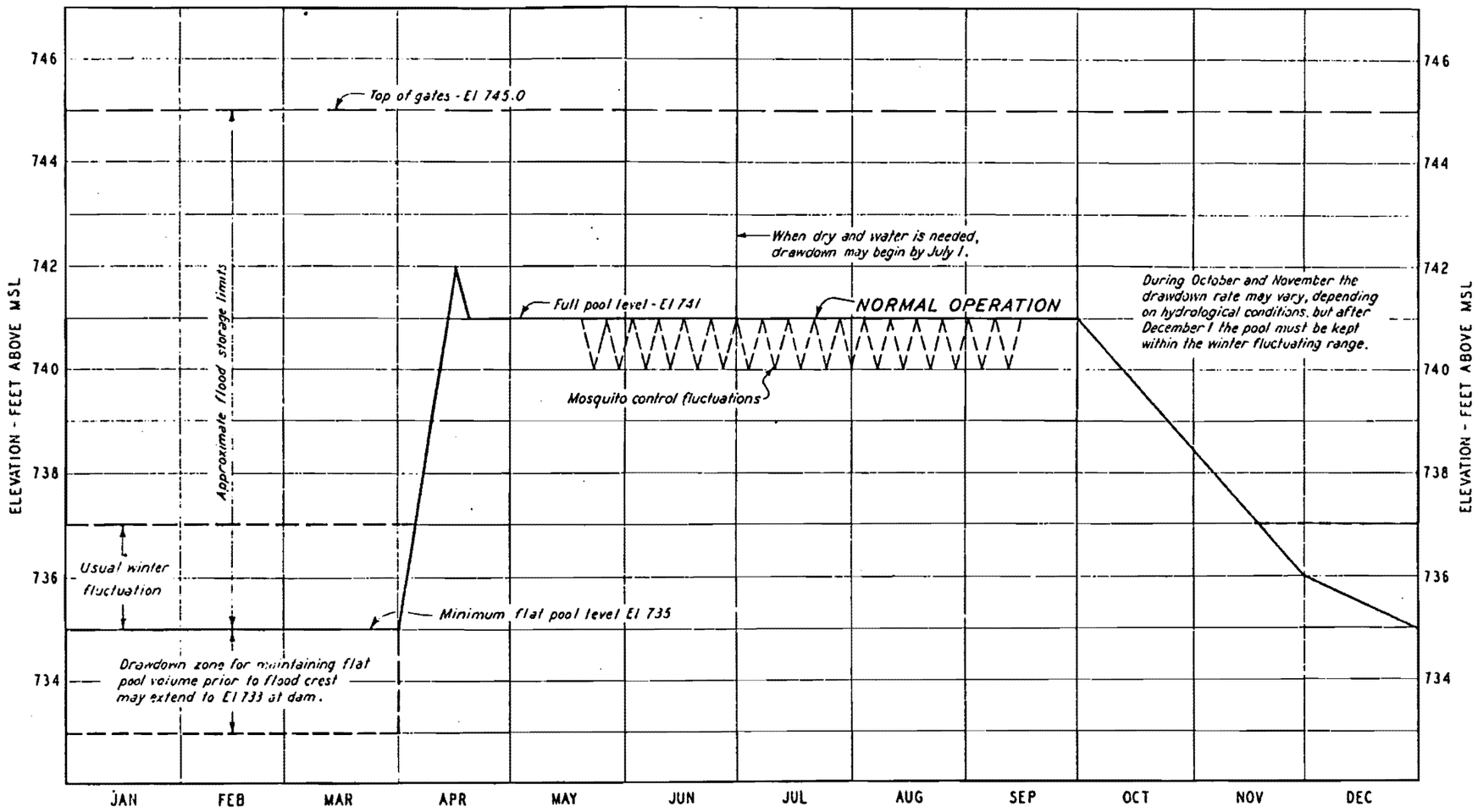
Figure 2.4-10 General Plan Elevations & Sections - Boone Project

Figure 2.4-11 General Plan Elevations &. Sections - Watauga Project

Figure 2.4-12 General Plan Elevation & Sections - South Holston Project

Figure 2.4-13 General Plan Elevation & Sections - Tellico Project

Figure 2.4-14 General Plan Elevation &. Sections - Chickamauga Project



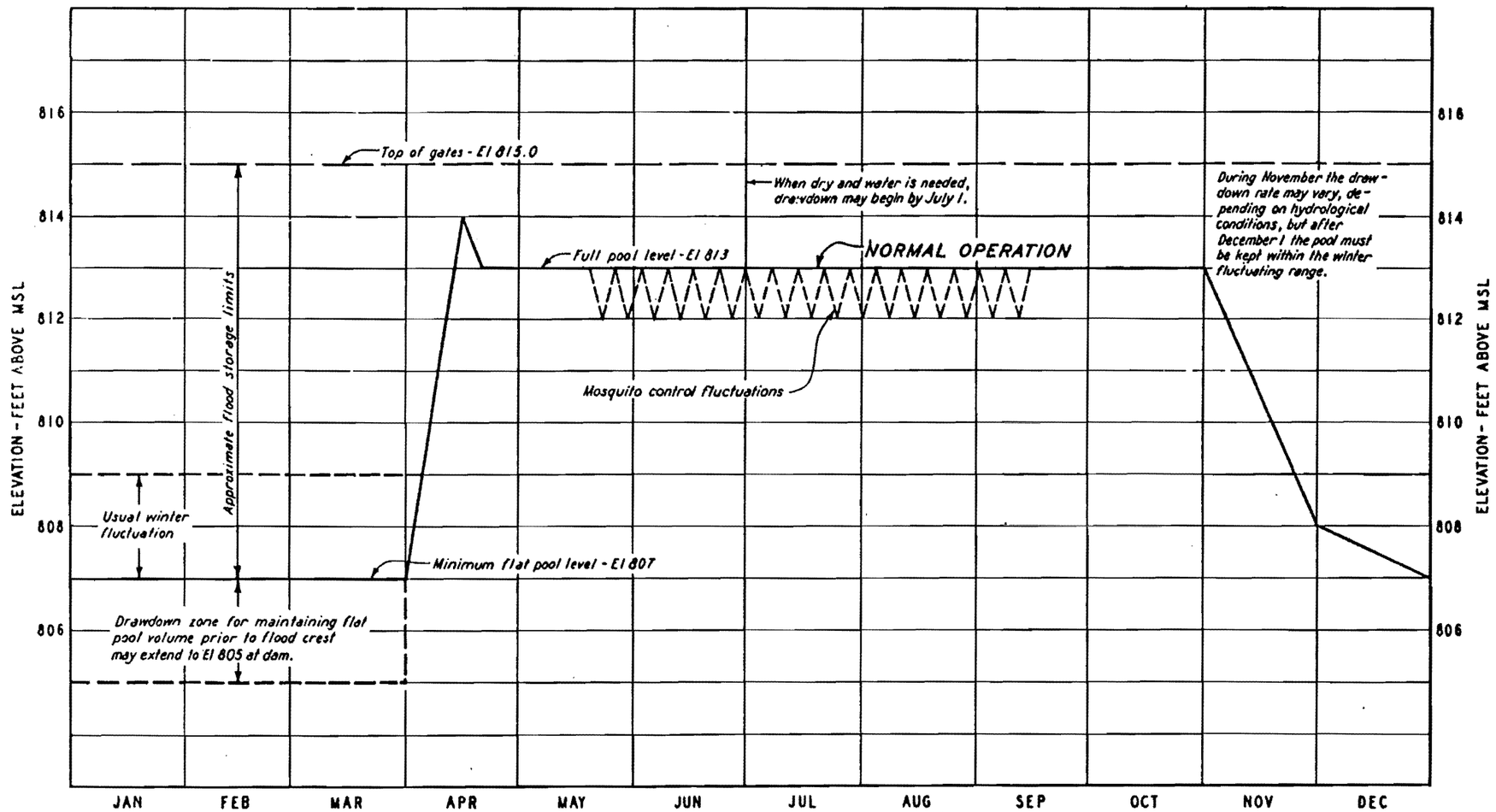
NOTES:
(1) Elevations apply only at dam.
(2) Maximum level assumed for design of dam - El 745.0.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATIONS
Watts Bar Project
Figure 2.4-15

Figure 2.4-15 Multiple-Purpose - Reservoir Operations Watts Bar Project



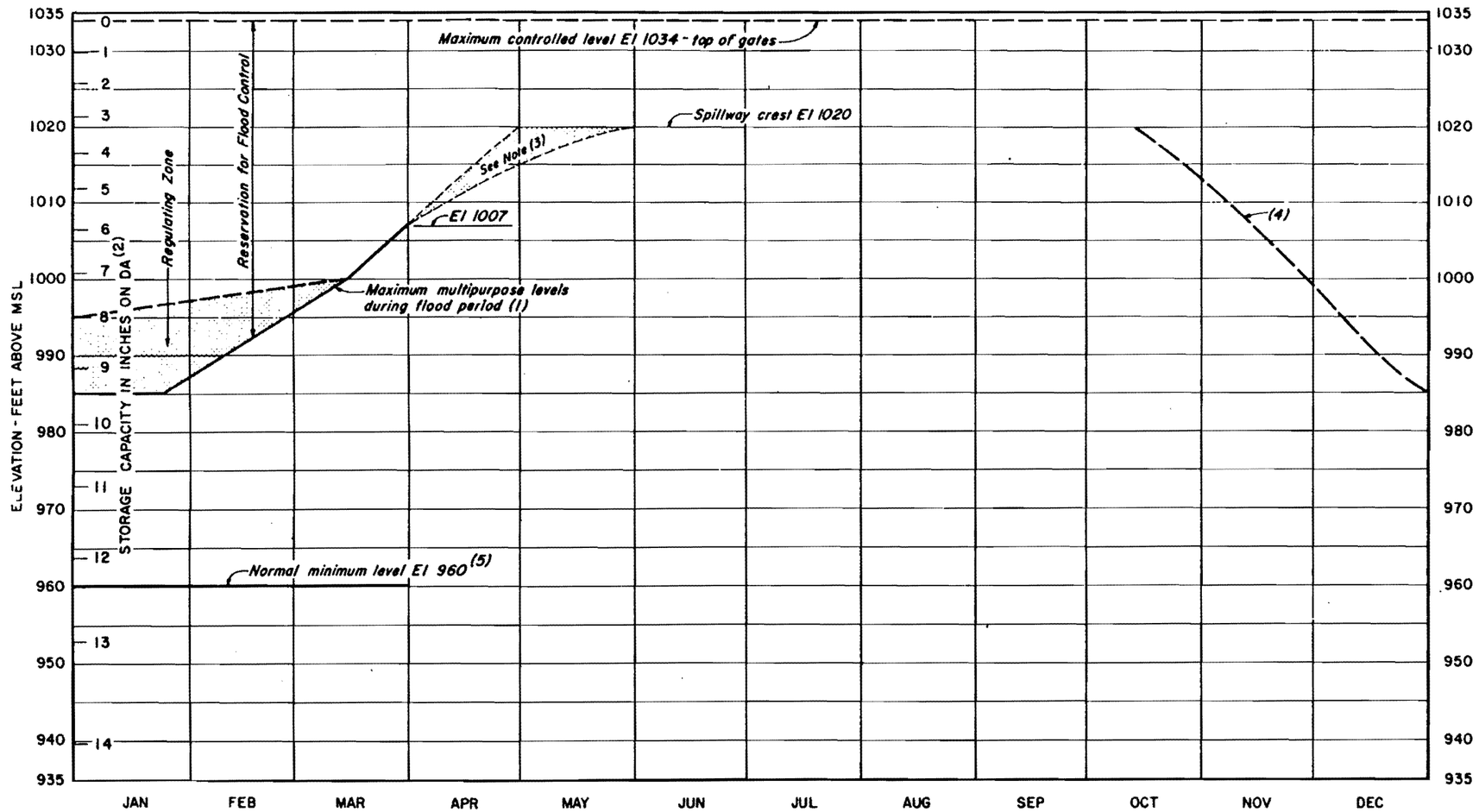
NOTES:
(1) Elevations apply only at dam.
(2) Maximum level assumed for design of dam - El 815.0.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATIONS
Fort Loudoun Project
Figure 2.4-16

Figure 2.4-16 Multiple-Purpose - Reservoir Operations - Fort Loudoun Project



NOTES:

(1) To be exceeded only during flood control operations or for temporary regulation dependent upon hydrological conditions.

(2) Based upon drainage area, 2,912 square miles.

(3) Limitation on filling after April 1 or on drawdown following floods will depend on currently existing hydrological conditions and levels in other reservoirs.

(4) Drawdown at full machine capacity as limited by generator or by full-gate turbine discharge with median inflow.

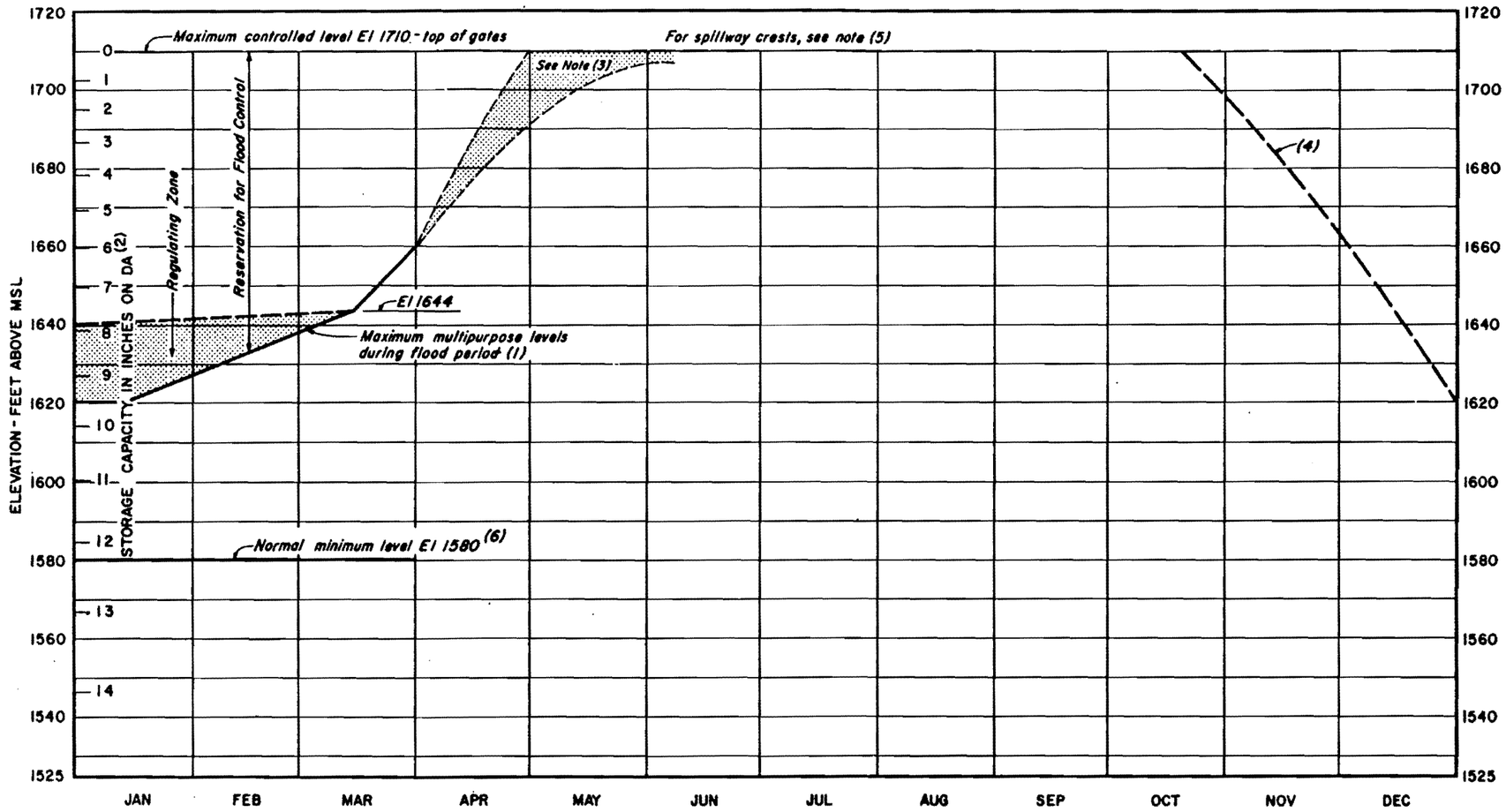
(5) Reservoir may be drawn infrequently to lower levels in the event of drought conditions. Generation can be maintained to approximately elevation 900.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATIONS
Norris Project
Figure 2.4-17

Figure 2.4-17 Multiple-Purpose - Reservoir Operations Norris Project



NOTES:

(1) To be exceeded only during flood control operations or for temporary regulation dependent upon hydrological conditions.

(2) Based upon drainage area at Fontana Dam less drainage areas at Thorpe and Nantahala Dams (1571-(36.7+91.0)= 1443.3 square miles).

(3) Limitation on filling after April 1 or on drawdown following floods will depend on currently existing hydrological conditions and levels in other reservoirs.

(4) Drawdown at full machine capacity as limited by generator or by full-gate turbine discharge with median inflow.

(5) Main spillway crest - El 1675, Emergency spillway crest - El 1715.

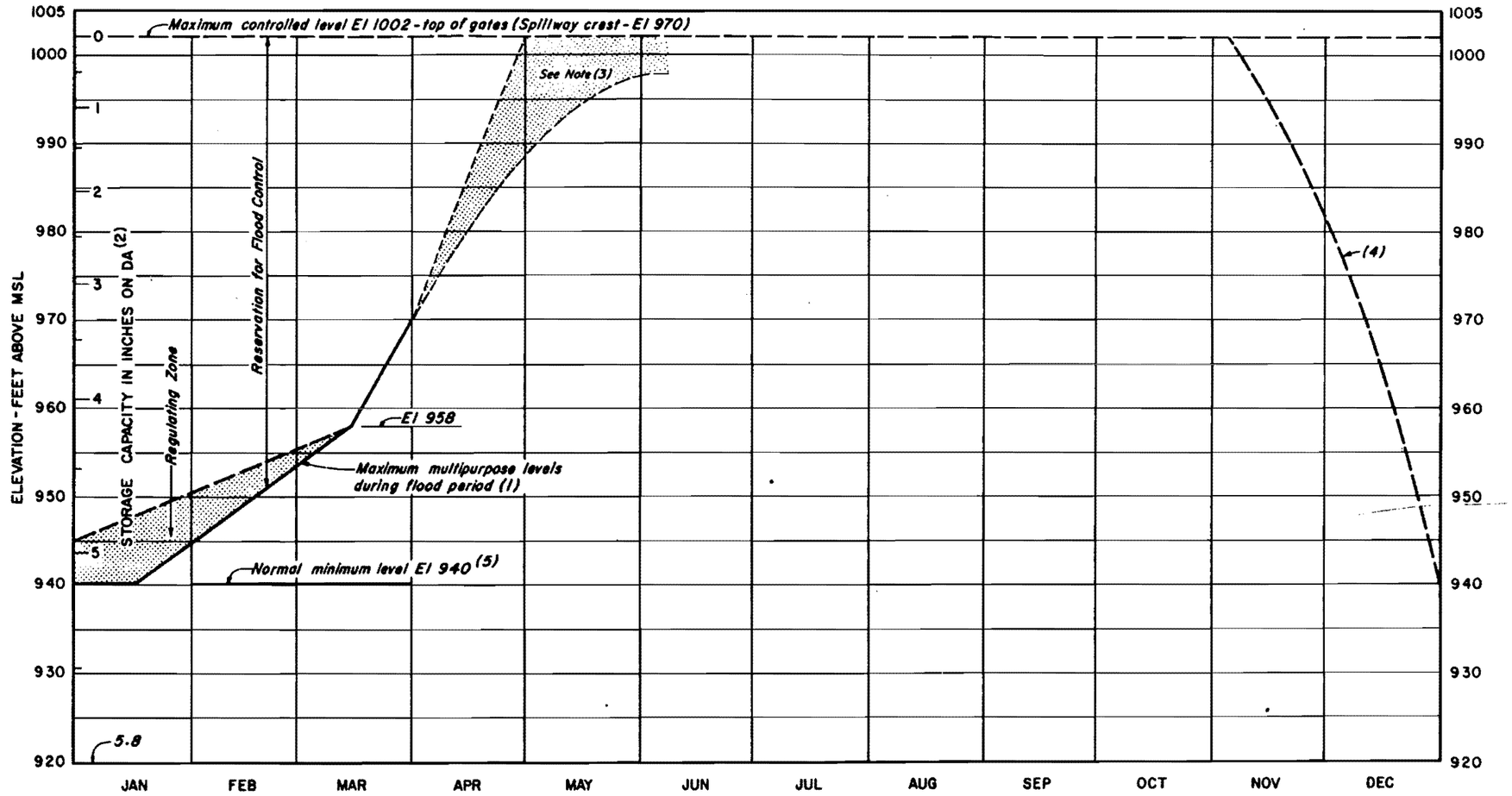
(6) Reservoir may be drawn infrequently to lower levels in the event of drought conditions. Generation can be maintained to approximately elevation 1470.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATION
Fontana Project
Figure 2.4-18

Figure 2.4-18 Multiple-Purpose - Reservoir Operations - Fontana Project



NOTES:

(1) To be exceeded only during flood control operations or for temporary regulation dependent upon hydrological conditions.

(2) Based upon drainage area, 4,541 square miles.

(3) Limitation on filling after April 1 or on drawdown following floods will depend on currently existing hydrological conditions and levels in other reservoirs.

(4) Drawdown at full machine capacity as limited by generator or by full-gate turbine discharge with median inflow.

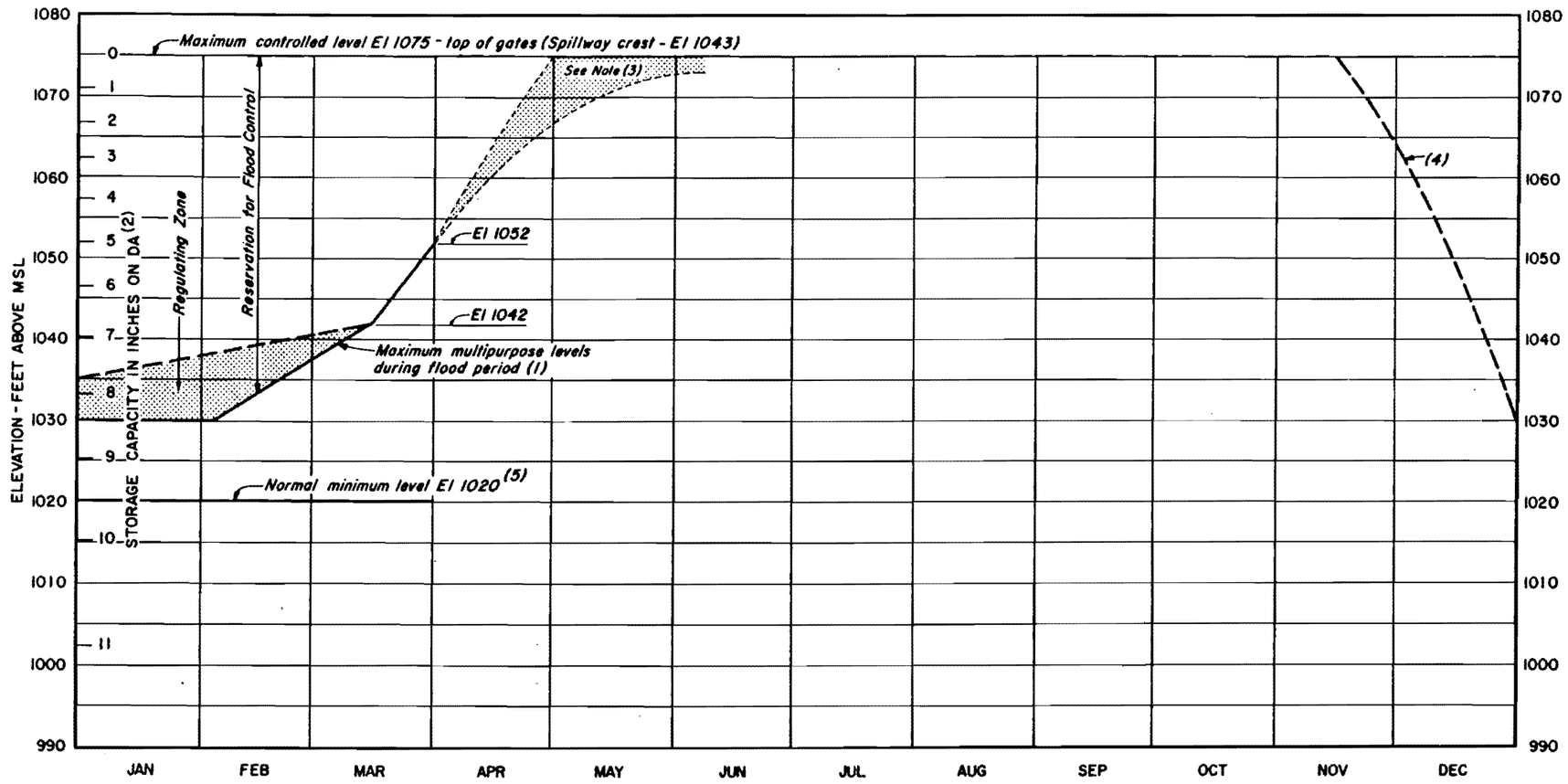
(5) Reservoir may be drawn infrequently to lower levels in the event of drought conditions. Generation can be maintained to approximately elevation 910.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATION
Douglas Project
Figure 2.4-19

Figure 2.4-19 Multiple-Purpose - Reservoir Operations - Douglas Project



NOTES:

(1) To be exceeded only during flood control operations or for temporary regulation dependent upon hydrological conditions.

(2) Based upon drainage area at Cherokee Dam less drainage areas at South Holston Dam and Watauga Dam (3428 - (703 + 468) = 2257 square miles). Does not include storage in Boone Reservoir.

(3) Limitation on filling after April 1 or on drawdown following floods will depend on currently existing hydrological conditions and levels in other reservoirs.

(4) Drawdown at full machine capacity as limited by generator or by full-gate turbine discharge with median inflow.

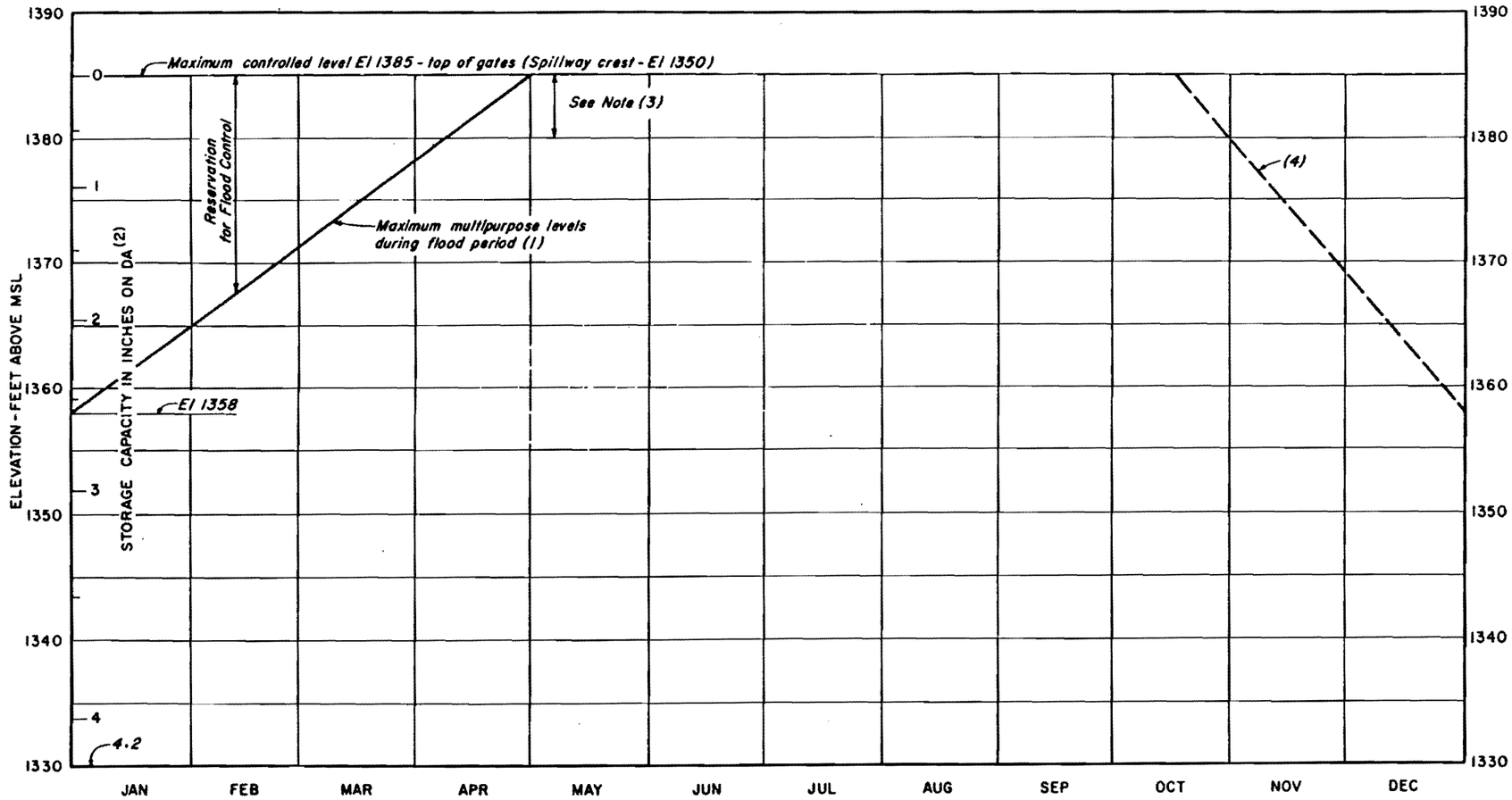
(5) Reservoir may be drawn infrequently to lower levels in the event of drought conditions. Generation can be maintained to approximately elevation 980.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATION
Cherokee Project
Figure 2.4-20

Figure 2.4-20 Multiple-Purpose - Reservoir Operations - Cherokee Project



NOTES:

(1) To be exceeded only during flood control operations.

(2) Based upon drainage area at Boone Dam less drainage areas at South Holston and Watauga Dams (1840 - (703 + 468) = 669 square miles).

(3) During the summer and fall, levels within the range 1380 - 1385 will be controlled to regulate flash floods and to conserve water.

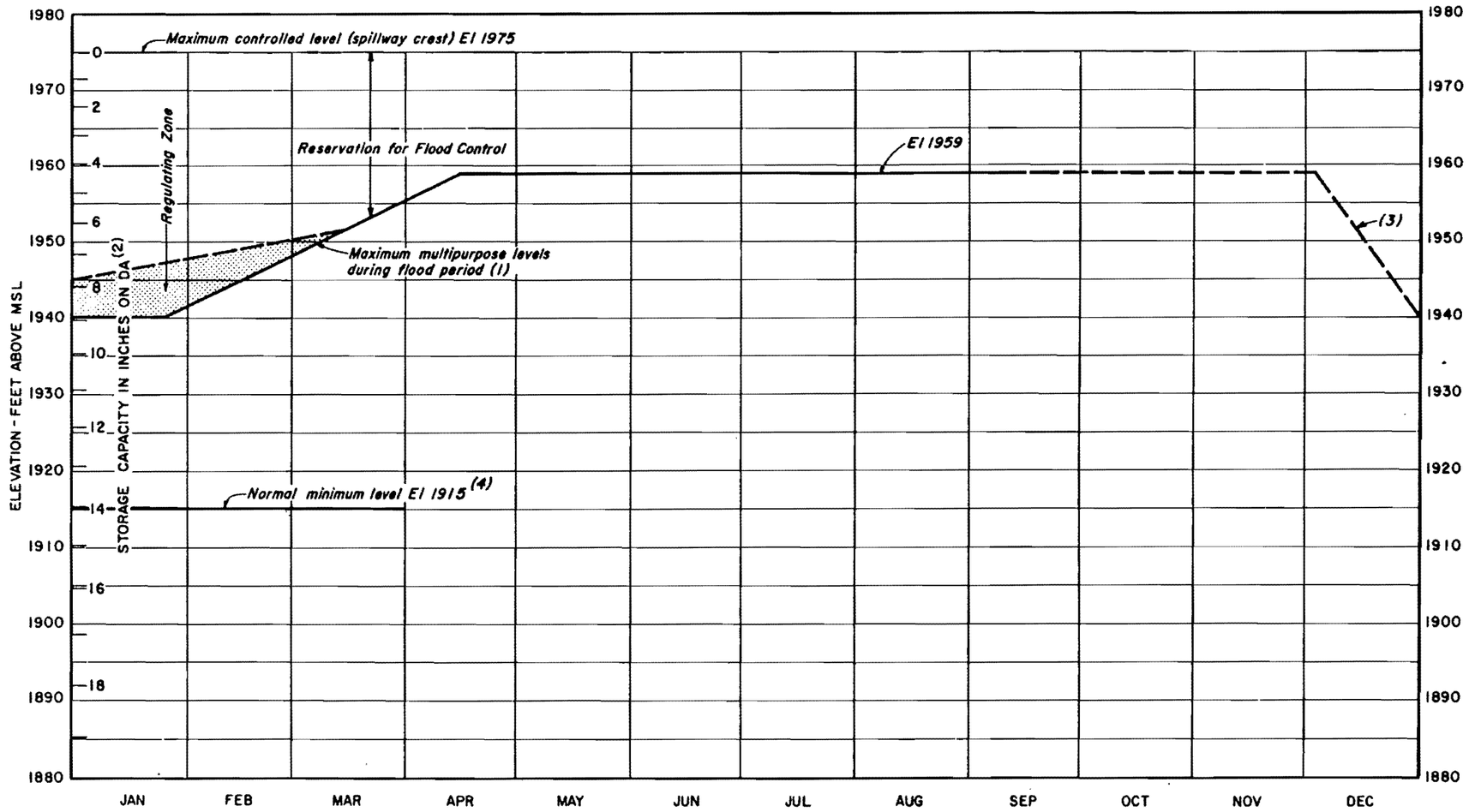
(4) Probable maximum levels.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATION
Boone Project
Figure 2.4-21

Figure 2.4-21 Multiple-Purpose - Reservoir Operations - Boone Project



NOTES:

(1) To be exceeded only during flood control operations or for temporary regulation dependent upon hydrological conditions.

(2) Based upon drainage area, 468 square miles.

(3) Drawdown at full machine capacity as limited by generator or by full-gate turbine discharge with median inflow.

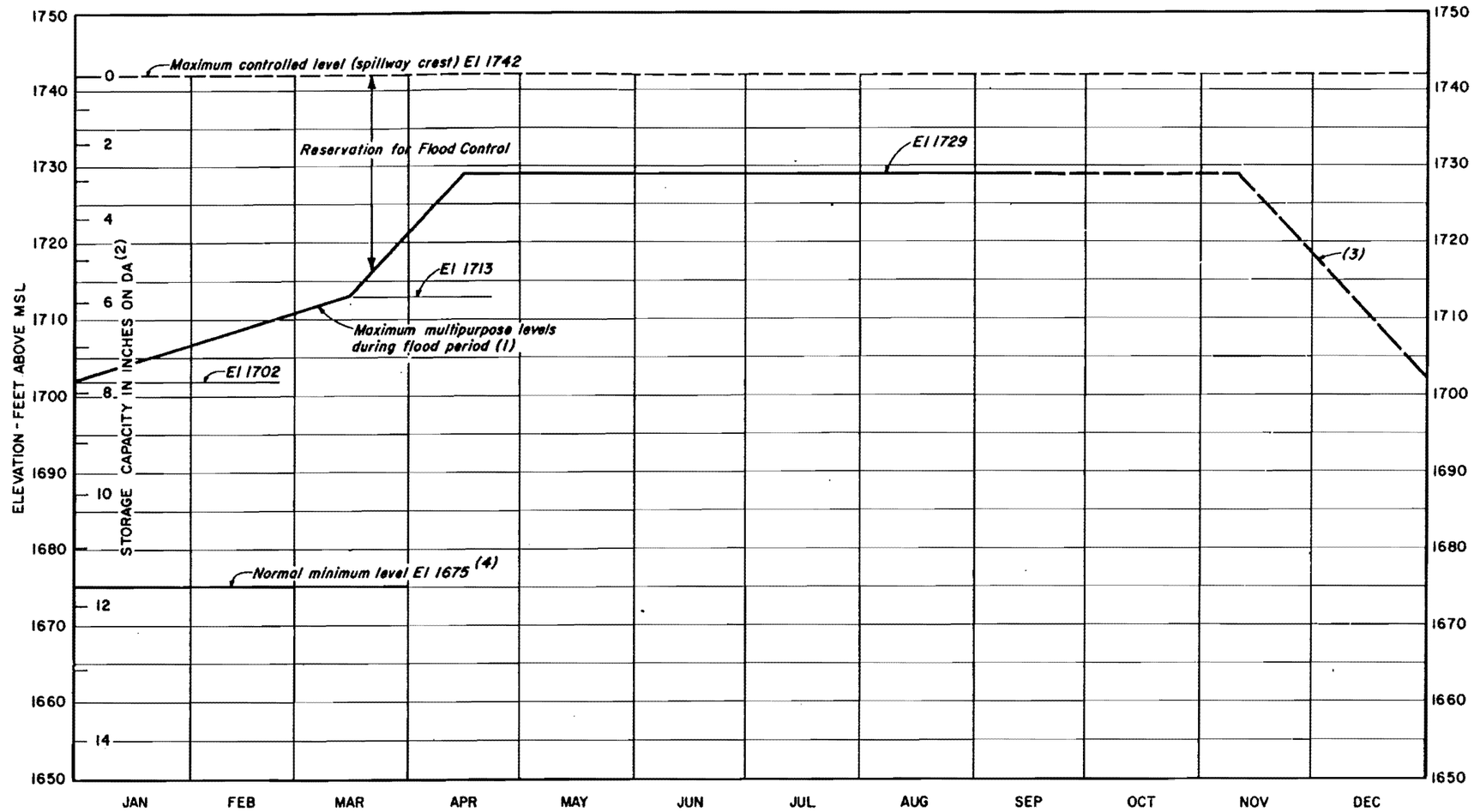
(4) Reservoir may be drawn infrequently to lower levels in the event of drought conditions. Generation can be maintained to approximately elevation 1815.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATION
Watauga Project
Figure 2.4-22

Figure 2.4-22 Multiple-Purpose - Reservoir Operations - Watauga Project



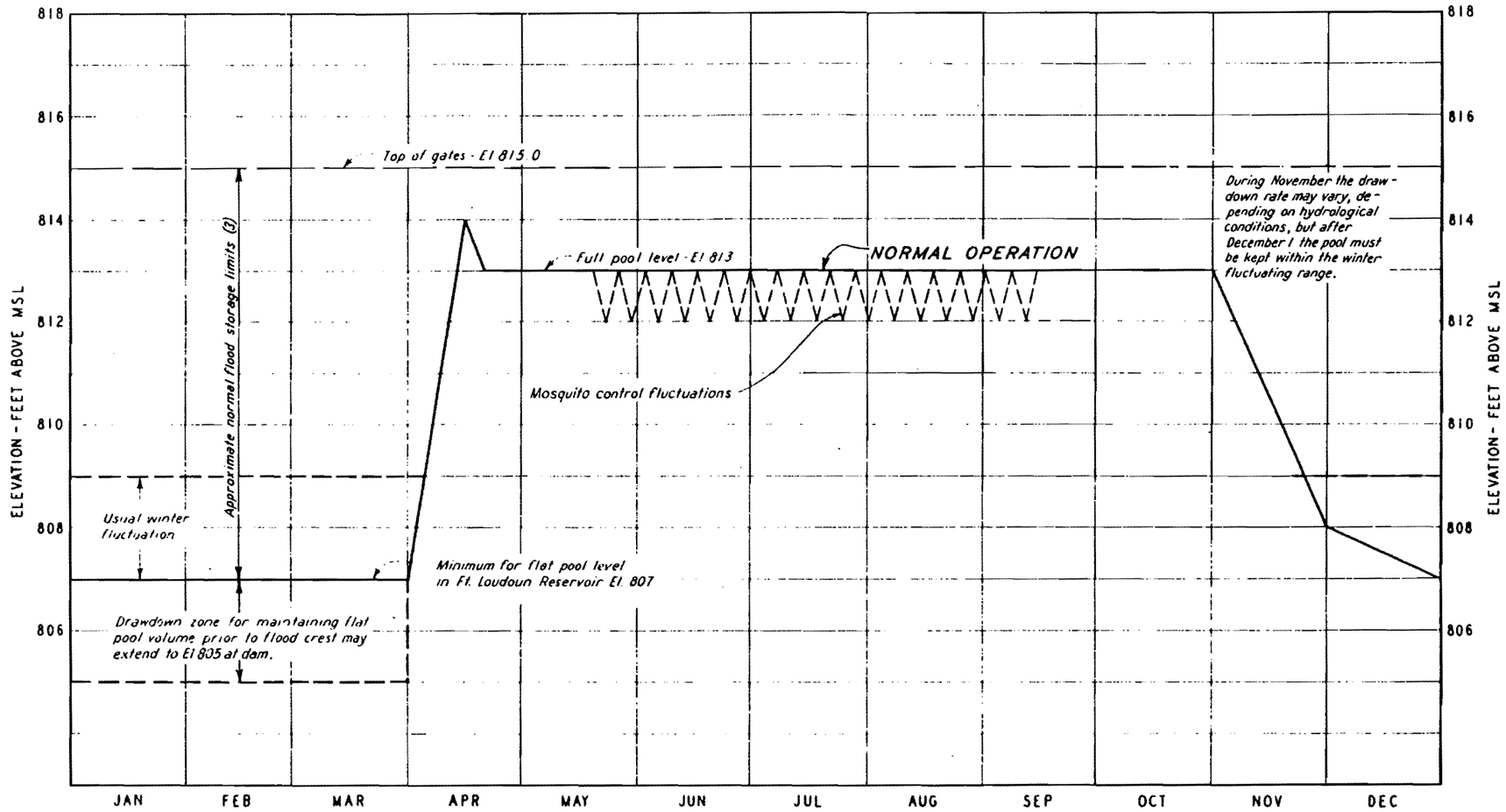
NOTES:
(1) To be exceeded only during flood control operations.
(2) Based upon drainage area, 703 square miles.
(3) Drawdown at full machine capacity as limited by generator or by full-gate turbine discharge with median inflow.
(5) Reservoir may be drawn infrequently to lower levels in the event of drought conditions. Generation can be maintained to approximately elevation 1616.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATION
South Holston Project
Figure 2.4-23

Figure 2.4-23 Multiple-Purpose - Reservoir Operations - South Holston Project



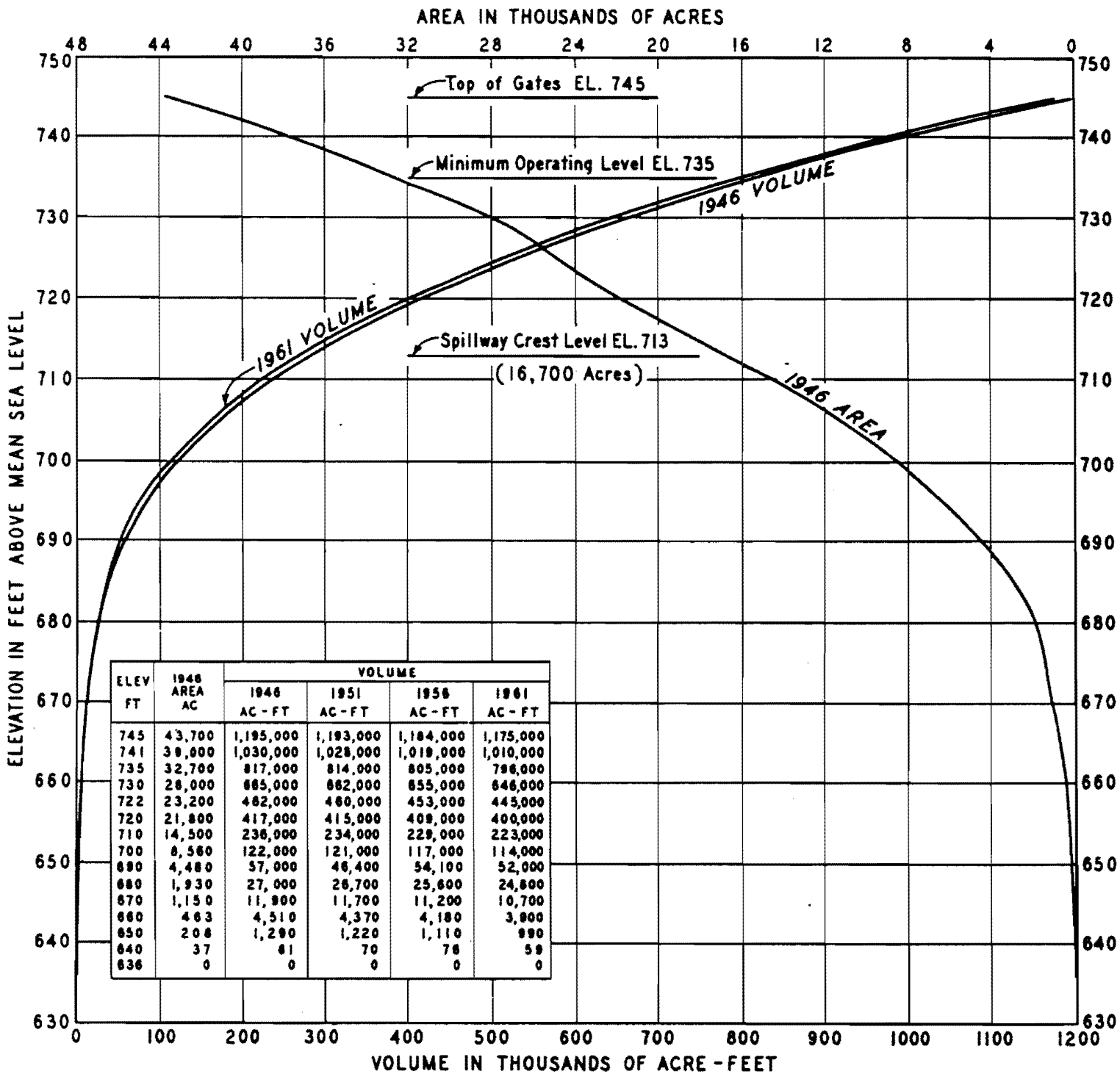
NOTES:
(1) Elevations apply only at dam.
(2) Maximum level assumed for design of dam - El 817.5.
(3) Under extreme flood conditions the reservoir may be surcharged as high as El 817.5.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATIONS
Tellico Project
Figure 2.4-24

Figure 2.4-24 Multiple-Purpose - Reservoir Operations - Tellico Project



ELEV FT	1946 AREA AC	VOLUME 1946 AC-FT	1951 AC-FT	1956 AC-FT	1961 AC-FT
745	43,700	1,195,000	1,193,000	1,184,000	1,175,000
741	39,000	1,030,000	1,028,000	1,019,000	1,010,000
735	32,700	817,000	814,000	805,000	796,000
730	28,000	665,000	662,000	655,000	646,000
722	23,200	462,000	460,000	453,000	445,000
720	21,800	417,000	415,000	409,000	400,000
710	14,500	236,000	234,000	229,000	223,000
700	8,560	122,000	121,000	117,000	114,000
690	4,480	57,000	46,400	54,100	52,000
680	1,930	27,000	26,700	25,600	24,800
670	1,150	11,900	11,700	11,200	10,700
660	463	4,510	4,370	4,180	3,800
650	208	1,290	1,220	1,110	990
640	37	81	70	76	59
636	0	0	0	0	0

NOTES:

Reservoir areas at elevation 720 and below were measured on a composite map prepared by the Hydraulic Data Branch with contours drawn at 10' intervals. The map was prepared from Tennessee River Survey Maps made by the U.S. Army Engineers, with contours at 690, 700, 710 and 720. Contours were made to conform to elevations on TVA sediment range cross sections located at one to five mile intervals. Areas above elevation 720 were measured on TVA navigation maps with contours at elevations 722, 730, 735, 741 and 745, scale 1" = ½ mile. The areas at these elevations check with areas at the same elevations previously measured on TVA land maps. The 1946 volume was computed by the contour method. Volumes of sediment on succeeding dates were computed by the constant factor method.

Elevations are referred to the USC & GS 1936 Supplementary Adjustment.

Area of original river within reservoir = 10,343 acres.

Drainage area at dam = 17,310 square miles.

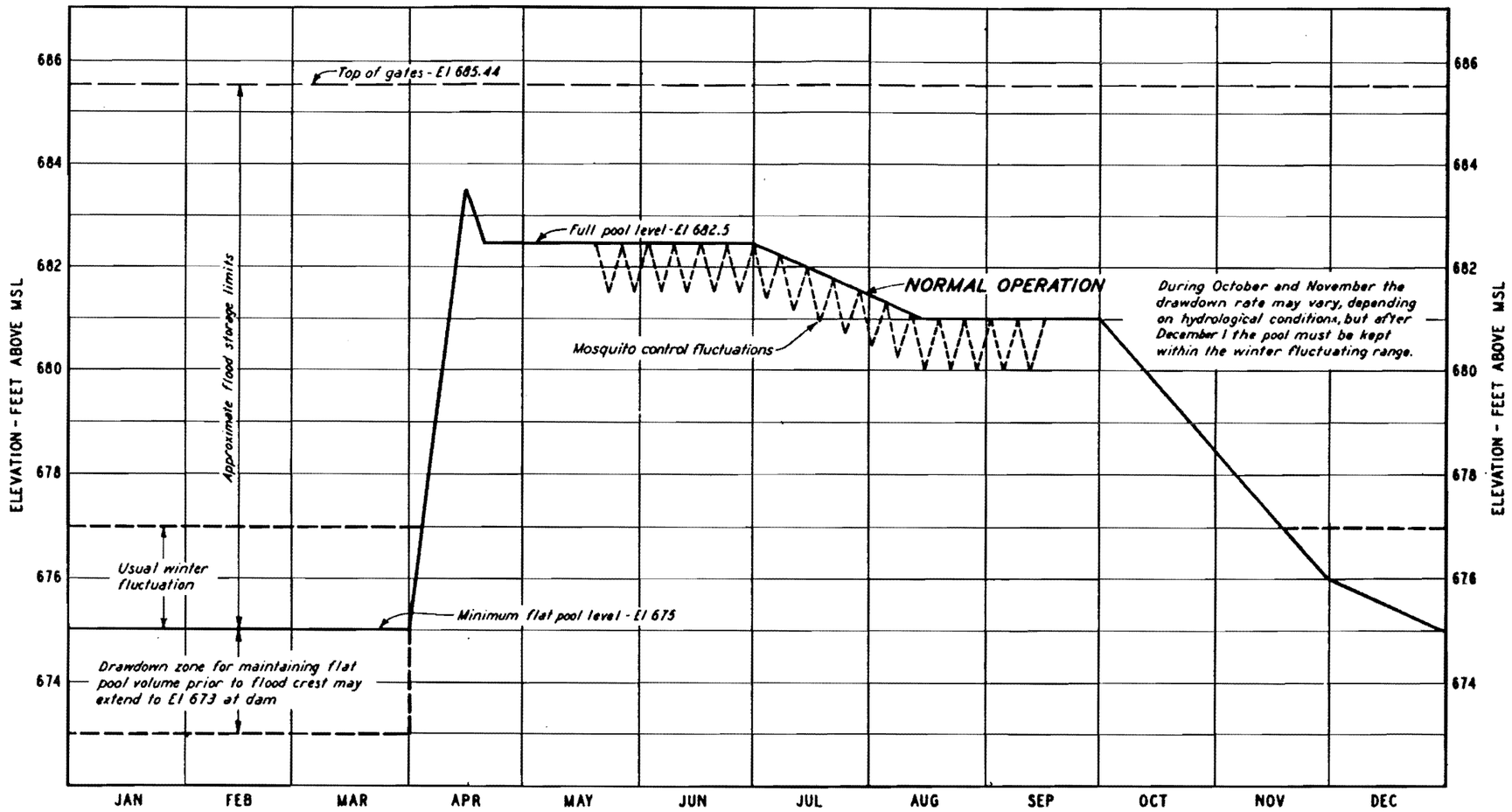
Dam closure January 1, 1942.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

TENNESSEE RIVER-MILE 529.9
RESERVOIR AREAS
AND VOLUMES
Watts Bar Project

Figure 2.4-25

Figure 2.4-25 Tennessee River-Mile 529.9 - Reservoir Areas And Volumes - Watts Bar Project



NOTES:
(1) Elevations apply only at dam.
(2) Maximum level assumed for design of dam - El 701.0.

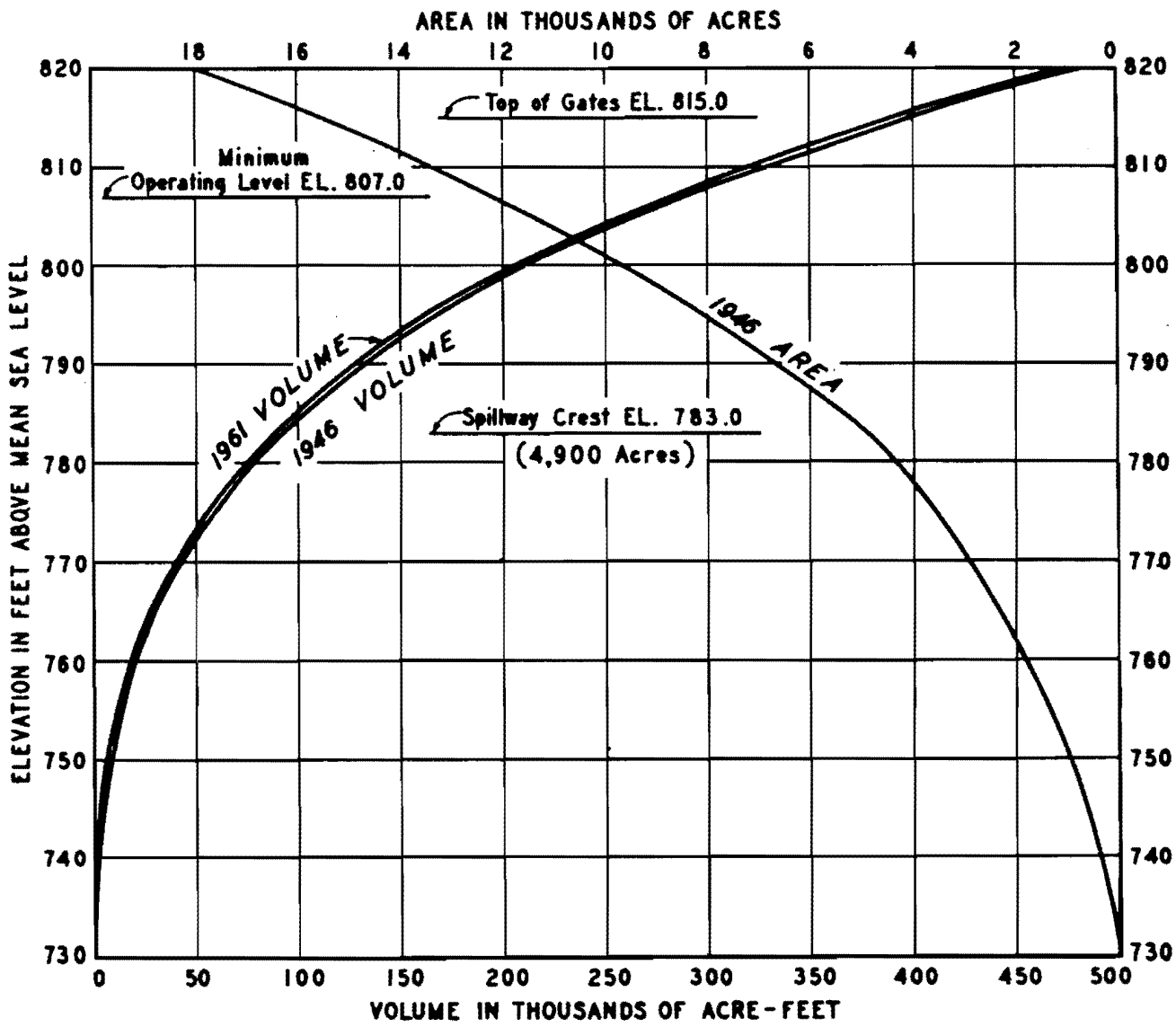
WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MULTIPLE-PURPOSE

RESERVOIR OPERATIONS
Chickamauga Project

Figure 2.4-26

Figure 2.4-26 Multiple-Purpose -Reservoir Operations - Chickamauga Project



ELEV FT	1946 AREA AC	VOLUME			
		1946 AC-FT	1951 AC-FT	1956 AC-FT	1961 AC-FT
820	18,000	485,000	483,000	482,000	477,000
815	15,500	401,000	399,000	398,000	393,000
813	14,800	371,000	369,000	368,000	363,000
807	12,200	290,000	289,000	287,000	282,000
802	10,200	234,000	233,000	232,000	227,000
796	8,340	179,000	178,000	177,000	172,000
790	6,680	133,000	133,000	132,000	128,000
780	4,230	79,200	78,800	78,500	75,700
770	2,880	43,800	43,700	43,600	41,600
760	1,790	20,700	20,400	20,200	19,600
750	966	7,930	7,550	6,950	7,450
740	324	1,620	1,350	810	1,890
730	0	0	0	0	0

NOTES:

Reservoir areas were measured on a composite map prepared by Hydraulic Data Branch with contours drawn at 10' intervals and scale 1"=500'. The map was prepared from TVA land maps with contours at elevations 790, 796, 802, 807, 813, 815 and 820. Contours were made to conform to elevations on TVA sediment range cross sections located at one-half to five-mile intervals. The 1946 volume was computed by the contour method. Volumes of sediment on succeeding dates were computed by the constant factor method.

Elevations are referred to the USC & GS 1936 Supplementary Adjustment.

Area of original river within reservoir = 4,420 acres.

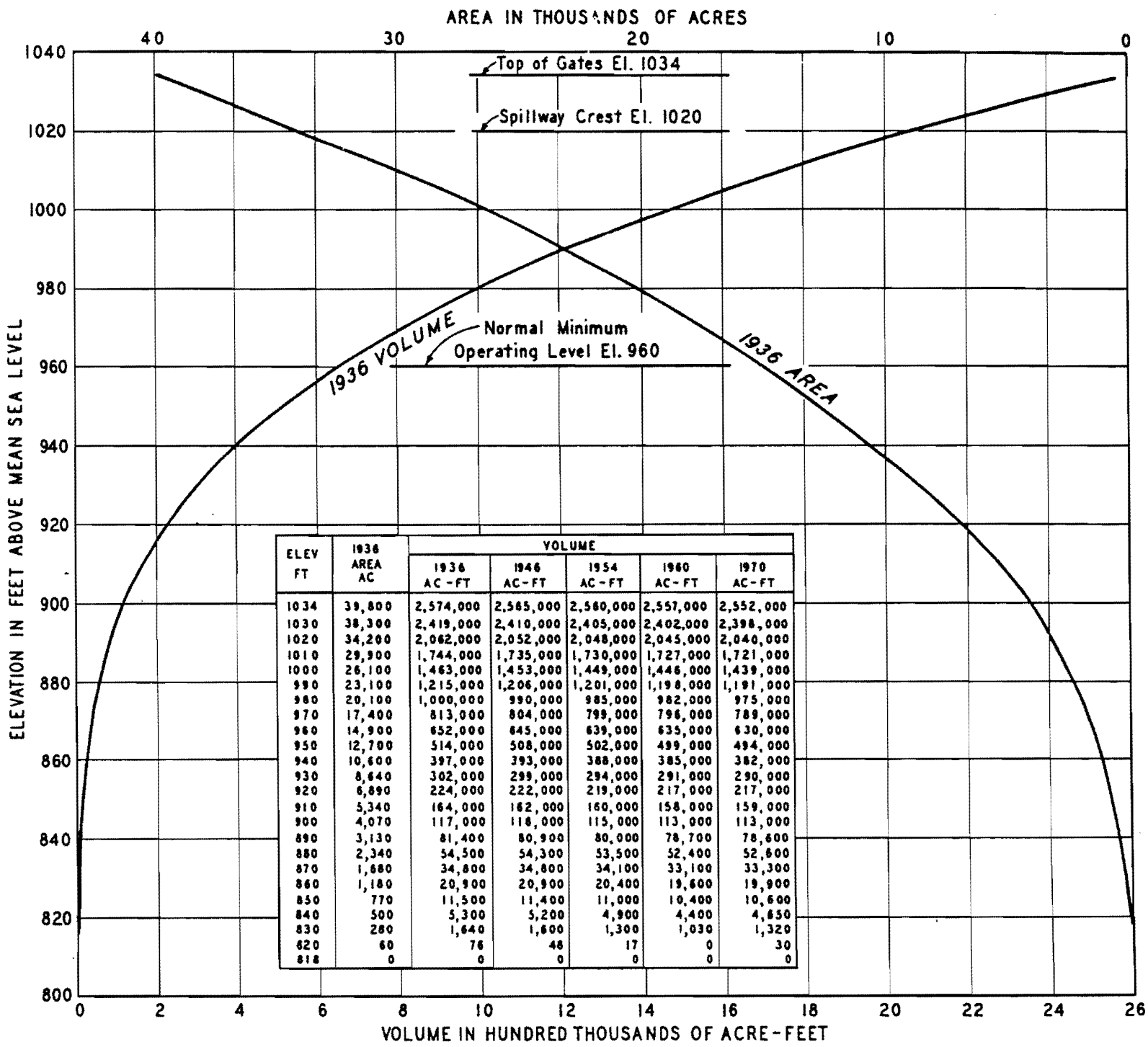
Drainage area at dam = 9,550 square miles.

Dam closure August 2, 1943.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

TENNESSEE RIVER MILE 602.3
RESERVOIR AREAS
AND VOLUMES
Fort Loudoun Project
Figure 2.4-27

Figure 2.4-27 Tennessee River Mile 602.3 -Reservoir Areas And Volumes - Fort Loudoun Project



ELEV FT	1936 AREA AC	VOLUME 1936 AC-FT	1946 AC-FT	1954 AC-FT	1960 AC-FT	1970 AC-FT
1034	39,800	2,574,000	2,565,000	2,560,000	2,557,000	2,552,000
1030	38,300	2,419,000	2,410,000	2,405,000	2,402,000	2,398,000
1020	34,200	2,062,000	2,052,000	2,048,000	2,045,000	2,040,000
1010	29,900	1,744,000	1,735,000	1,730,000	1,727,000	1,721,000
1000	26,100	1,463,000	1,453,000	1,449,000	1,446,000	1,439,000
990	23,100	1,215,000	1,206,000	1,201,000	1,198,000	1,191,000
980	20,100	1,000,000	990,000	985,000	982,000	975,000
970	17,400	813,000	804,000	799,000	796,000	789,000
960	14,900	652,000	645,000	639,000	635,000	630,000
950	12,700	514,000	508,000	502,000	499,000	494,000
940	10,600	397,000	393,000	388,000	385,000	382,000
930	8,640	302,000	299,000	294,000	291,000	290,000
920	6,890	224,000	222,000	219,000	217,000	217,000
910	5,340	164,000	162,000	160,000	158,000	159,000
900	4,070	117,000	116,000	115,000	113,000	113,000
890	3,130	81,400	80,900	80,000	78,700	78,600
880	2,340	54,500	54,300	53,500	52,400	52,600
870	1,680	34,800	34,800	34,100	33,100	33,300
860	1,180	20,900	20,900	20,400	19,600	19,900
850	770	11,500	11,400	11,000	10,400	10,600
840	500	5,300	5,200	4,900	4,400	4,650
830	280	1,640	1,600	1,300	1,030	1,320
820	60	76	48	17	0	30
818	0	0	0	0	0	0

NOTES:

Reservoir areas were measured on Tennessee River Survey Maps with contours drawn at 10' intervals and scale 1" = 1,250'. The contours were made to conform to elevations on the TVA sediment range cross-sections located at approximately one-half mile intervals within the reservoir. The areas on these maps were adjusted proportionately to conform to the area of the 1020 contour on the TVA Reservation Map, scale 1" = 1000'.

Elevations are referred to the 1912 Fourth General Adjustment of the USC&GS. To correct to the 1936 Supplementary Adjustment, add 0.11 Foot.

Area of original river within the reservoir = 2,930 acres.

Drainage area at the dam = 2,912 square miles.

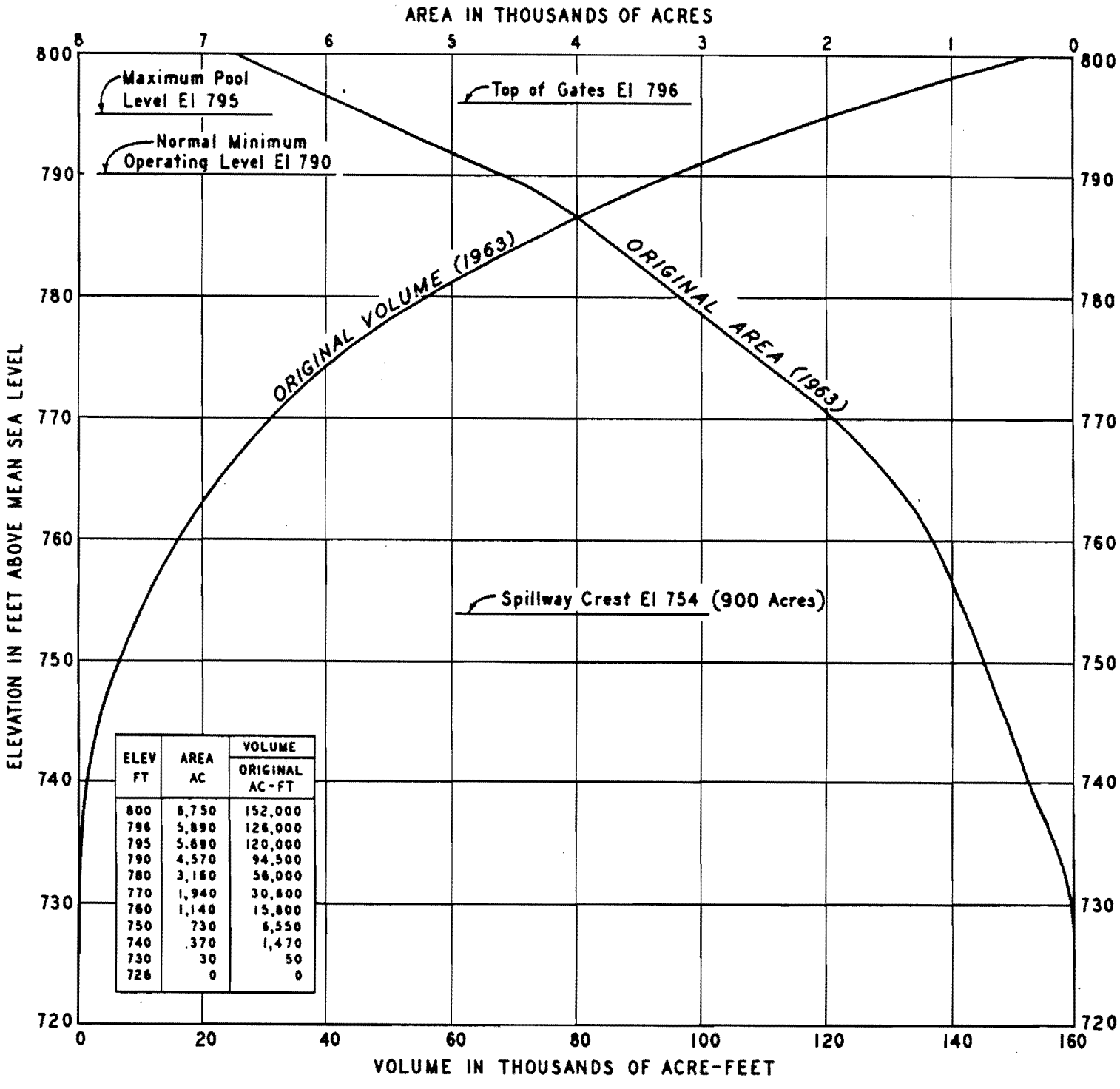
The 1946 and subsequent volumes were determined by the constant factor method for computing sediment.

Dam closure March 4, 1936.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

CLINCH RIVER MILE 79.8
RESERVOIR AREAS
AND VOLUMES
Norris Project
Figure 2.4-28

Figure 2.4-28 Clinch River Mile 79.8 -Reservoir Areas And Volumes - Norris Project



ELEV FT	AREA AC	VOLUME ORIGINAL AC-FT
800	6,750	152,000
796	5,890	126,000
795	5,690	120,000
790	4,570	94,500
780	3,160	56,000
770	1,940	30,600
760	1,140	15,800
750	730	6,550
740	370	1,470
730	30	50
726	0	0

NOTES:

Reservoir areas were measured on a composite map prepared by the Hydraulic Data Branch with contours drawn at 10' intervals and scale 1"=1,000'. The map was prepared from TVA topographic maps with contours at 5' intervals below elevation 820.

Contours were made to conform to elevations on TVA sediment range cross sections located at one to two mile intervals.

The 1963 volume was computed by the contour method.

Elevations are referred to the USC & GS 1936 Supplementary Adjustment.

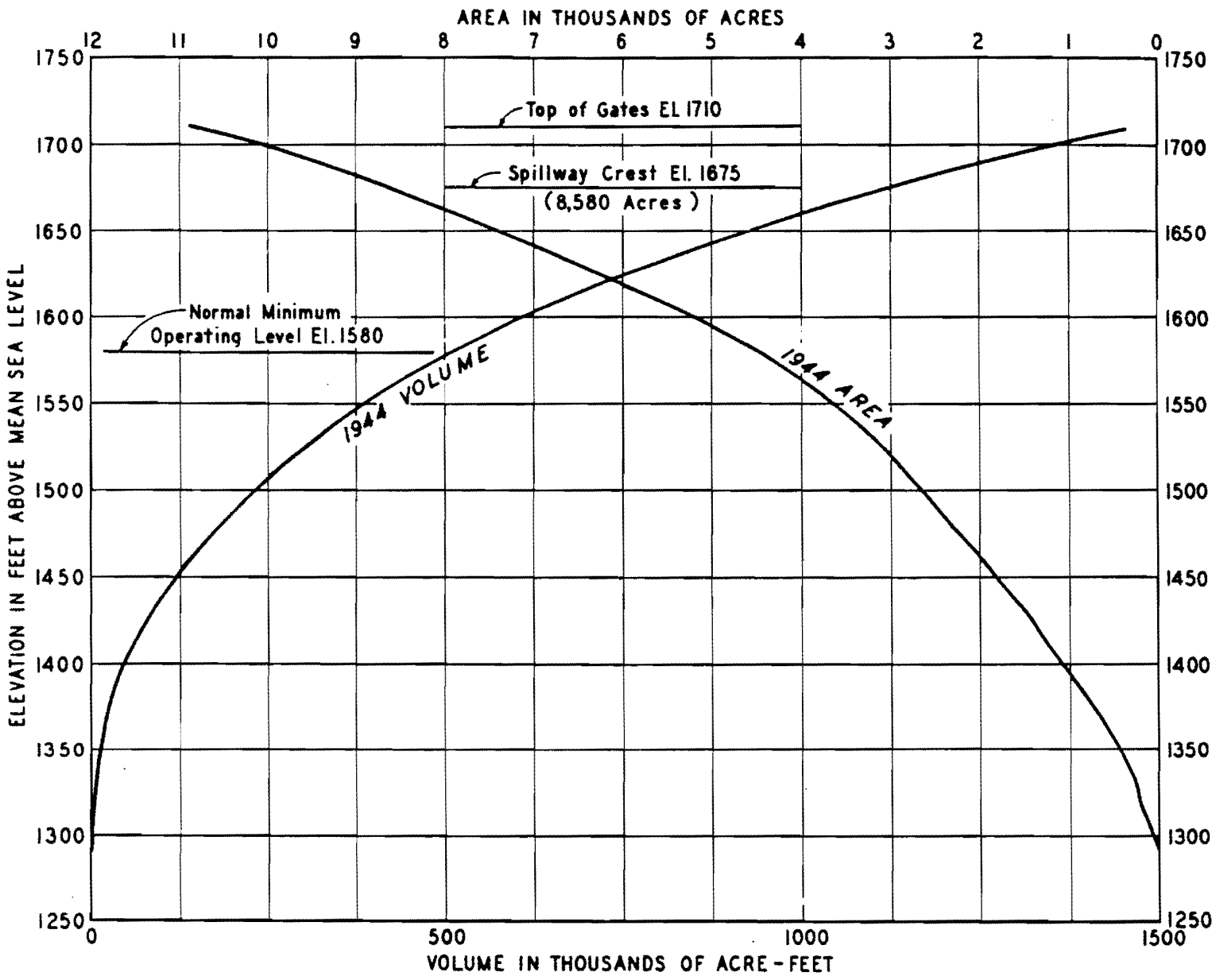
Area of original river within reservoir=1,461 acres.

Drainage area at dam=3,343 square miles.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

CLINCH RIVER MILE 23.1
RESERVOIR AREAS
AND VOLUMES
Melton Hill Project
Figure 2.4-29

Figure 2.4-29 Clinch River Mile 23.1 - Reservoir Areas and Volumes - Melton Hill Project



ELEV FT	1944 AREA AC	VOLUME				
		1944 AC-FT	1950 AC-FT	1954 AC-FT	1959 AC-FT.	1967 AC-FT
1710	10,870	1,455,000	1,452,000	1,451,000	1,448,000	1,443,000
1700	10,040	1,351,000	1,349,000	1,347,000	1,344,000	1,339,000
1680	8,900	1,162,000	1,160,000	1,158,000	1,155,000	1,150,000
1660	7,870	995,000	992,000	991,000	988,000	983,000
1640	6,940	847,000	844,000	843,000	840,000	835,000
1620	6,050	717,000	714,000	713,000	710,000	706,000
1600	5,240	604,000	602,000	600,000	598,000	593,000
1580	4,480	507,000	505,000	503,000	501,000	497,000
1560	3,890	424,000	422,000	420,000	418,000	415,000
1540	3,380	351,000	350,000	348,000	346,000	343,000
1520	2,990	288,000	287,000	286,000	284,000	282,000
1500	2,650	231,000	230,000	229,000	228,000	226,000
1480	2,310	182,000	181,000	180,000	179,000	178,000
1460	1,970	139,000	138,000	138,000	137,000	136,000
1440	1,670	102,000	101,000	101,000	101,000	100,000
1420	1,350	72,600	72,000	71,800	71,000	70,800
1400	1,070	48,400	47,900	47,700	47,500	46,900
1380	790	29,900	29,500	29,400	29,400	28,700
1360	530	16,800	16,600	16,400	16,400	16,100
1340	330	8,570	8,360	8,160	8,160	8,030
1320	210	3,010	3,010	2,850	2,850	2,770
1300	80	275	275	209	209	210
1291	0	0	0	0	0	0

NOTES:

Reservoir areas were measured on a composite map prepared by the Hydraulic Data Branch with contours drawn at 10' intervals and scale 1"=500'. The map was prepared from TVA land maps with contours at 1540 & 1710, and from TVA topographic maps with contours at elevations 1320, 1400, 1480, 1560, 1600 & 1680. Contours were made to conform to elevations on TVA sediment range cross sections which are located at two mile intervals. The 1944 volume was computed by the contour method. Volumes of sediment on succeeding dates were computed by the constant factor method.

Elevations are referred to the USC & GS 1936 supplementary adjustment.

Drainage area at dam = 1,571 square miles.

Dam closure November 7, 1944.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

LITTLE TENNESSEE RIVER MILE 61.0
RESERVOIR AREAS
AND VOLUMES
Fontana Project
Figure 2.4-30

Figure 2.4-30 Little Tennessee River Mile 61.0 - Reservoir Areas and Volumes - Fontana Project

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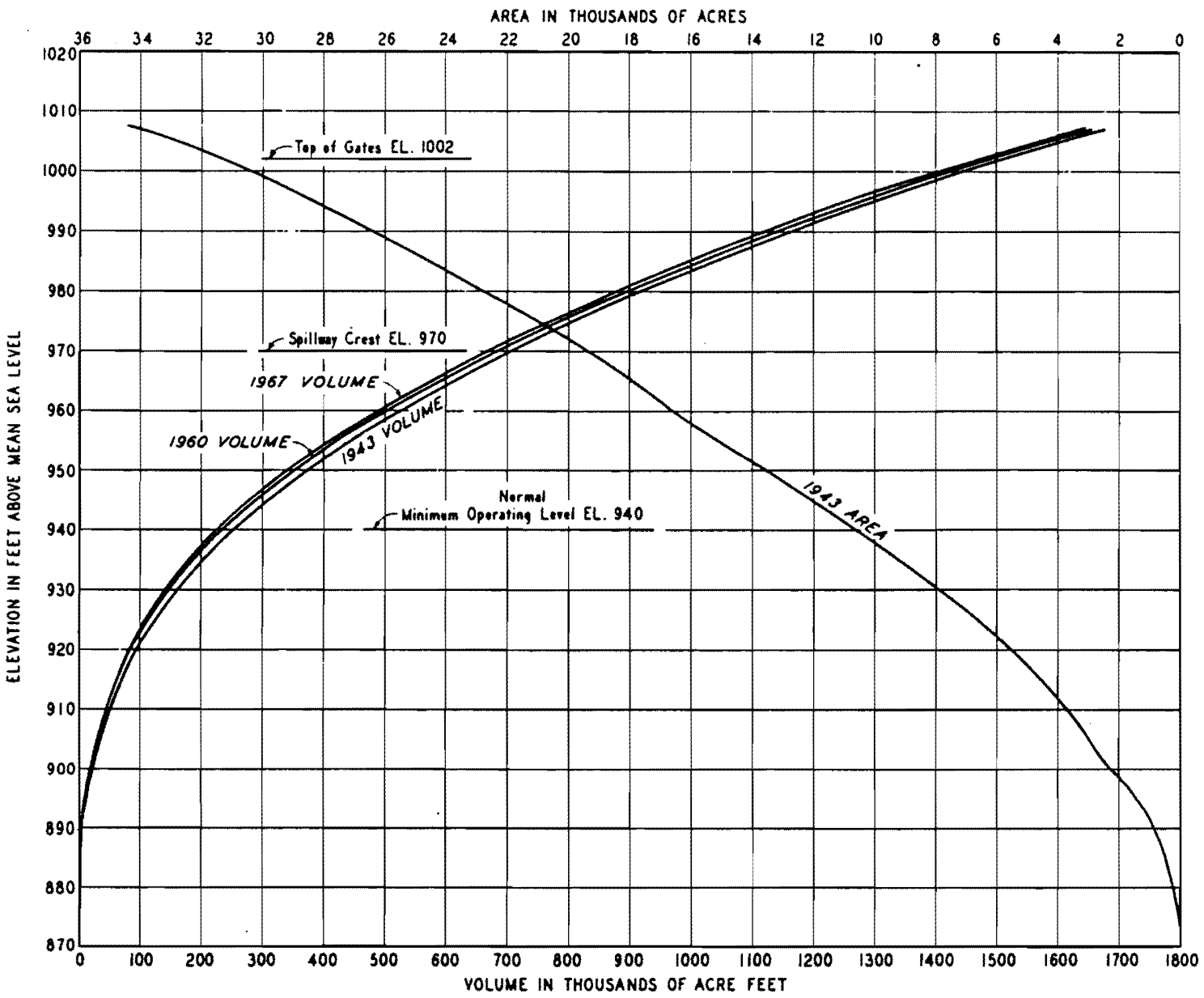


ELEV FT	1943 AREA AC	VOLUME 1943 AC-FT	1949 AC-FT	1955 AC-FT	1960 AC-FT	1967 AC-FT
1,007	34,000	1,677,000	1,662,000	1,666,000	1,655,000	1,641,000
1,002	31,300	1,512,000	1,497,000	1,501,000	1,490,000	1,475,000
990	26,450	1,164,000	1,148,000	1,153,000	1,142,000	1,127,000
980	22,860	918,000	902,000	906,000	896,000	881,000
970	19,410	706,000	691,000	694,000	683,000	669,000
960	16,650	526,000	512,000	514,000	503,000	489,000
950	13,540	376,000	363,000	363,000	353,000	340,000
940	10,600	255,000	245,000	243,000	235,000	223,000
930	7,890	163,000	156,000	153,000	146,000	138,000
920	5,540	96,300	92,600	89,100	84,500	80,300
910	3,700	50,400	48,500	47,200	43,900	42,500
900	2,320	20,800	19,900	19,500	17,500	16,700
890	860	5,300	4,800	4,600	3,800	3,600
880	220	600	400	400	300	200
873	0	0	0	0	0	0

NOTES:

Reservoir areas were measured on a composite map prepared by the Hydraulic Data Branch with contours drawn at 10' intervals. The map was prepared from TVA land maps with contours at elevations 910, 930, 1002 & 1007. Contours were made to conform to elevations on TVA sediment range cross sections which are located at one to three mile intervals within the reservoir.

Elevations are referred to the 1936 Supplementary Adjustment of the U.S.C. & G.S.

Area of original river within reservoir at elevation 1002 = 3,170 acres.

Drainage area at dam = 4,541 square miles.

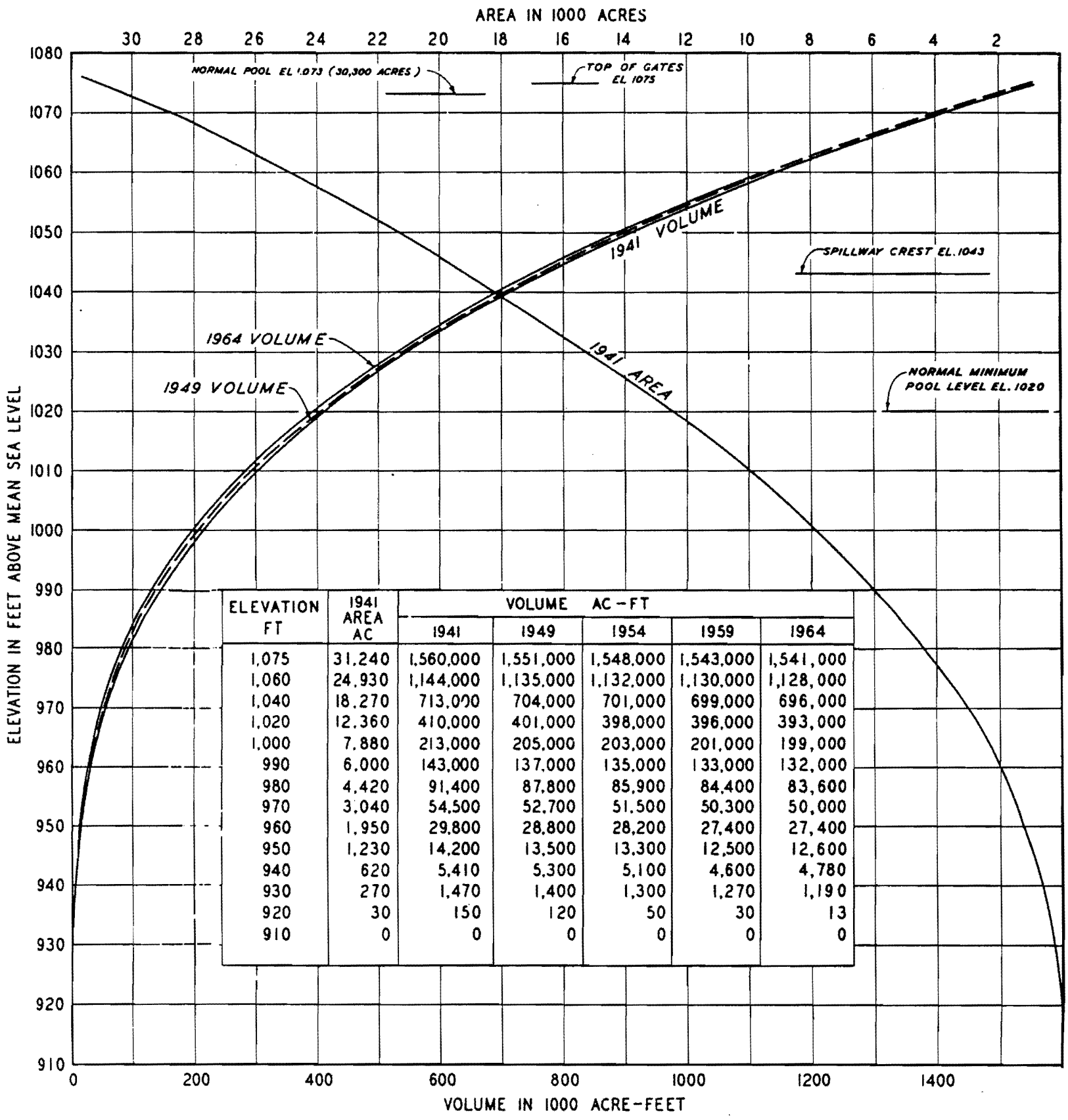
The 1949 and subsequent volumes were determined by the constant factor method for computing sediment.

Dam closure February 19, 1943.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

FRENCH BROAD RIVER MILE 32.3
RESERVOIR AREAS
AND VOLUMES
Douglas Project
Figure 2.4-31

Figure 2.4-31 French Broad River Mile 32.3 - Reservoir Areas And Volumes - Douglas Project



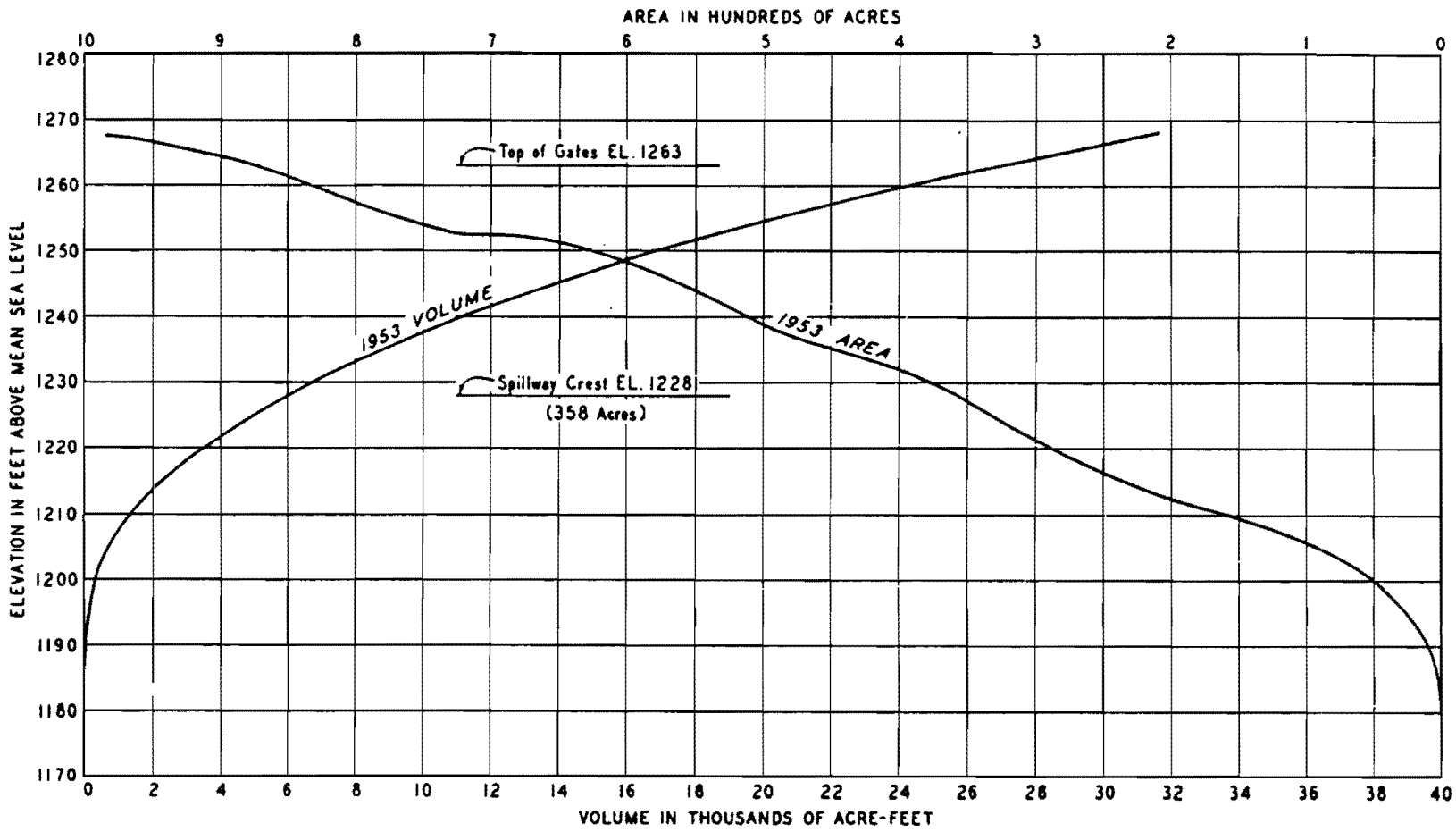
ELEVATION FT	1941 AREA AC	VOLUME AC-FT				
		1941	1949	1954	1959	1964
1,075	31,240	1,560,000	1,551,000	1,548,000	1,543,000	1,541,000
1,060	24,930	1,144,000	1,135,000	1,132,000	1,130,000	1,128,000
1,040	18,270	713,000	704,000	701,000	699,000	696,000
1,020	12,360	410,000	401,000	398,000	396,000	393,000
1,000	7,880	213,000	205,000	203,000	201,000	199,000
990	6,000	143,000	137,000	135,000	133,000	132,000
980	4,420	91,400	87,800	85,900	84,400	83,600
970	3,040	54,500	52,700	51,500	50,300	50,000
960	1,950	29,800	28,800	28,200	27,400	27,400
950	1,230	14,200	13,500	13,300	12,500	12,600
940	620	5,410	5,300	5,100	4,600	4,780
930	270	1,470	1,400	1,300	1,270	1,190
920	30	150	120	50	30	13
910	0	0	0	0	0	0

All areas were determined from Corps of Engineers topographic map of the Holston River Valley Dwg Nos. 169-174 at a contour interval of 10 feet, with the exception of a few areas at the higher elevations which were taken from TVA topographic maps at a contour interval of 20 feet, and the area at the 1075 contour which was determined from TVA navigation maps of Cherokee Reservoir.
All elevations are referred to the 1936 Supplementary Adjustment of the USC & GS.
Area of original river within reservoir = 2,426 acres.
Drainage area at dam = 3,428 square miles.
This drawing was prepared by the Hydraulic Data Branch.
The 1949 and subsequent volumes were determined by the constant factor method for computing sediment.
1959 and 1964 volumes and area at normal pool elevation 1073 exclude John Sevier Detention Pool.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

HOLSTON RIVER MILE 52.3
RESERVOIR AREAS
AND VOLUMES
Cherokee Project
Figure 2.4-32

Figure 2.4-32 Holston River Mile 52.3 - Reservoir Areas And Volumes - Cherokee Project



ELEV FT	1953 AREA AC	VOLUME		
		1953 AC-FT	1959 AC-FT	1964 AC-FT
1268	990	31,700	31,500	31,500
1263	872	27,100	26,900	26,900
1250	628	16,900	16,700	16,700
1240	511	11,200	11,100	11,100
1230	377	6,820	6,710	6,670
1220	288	3,540	3,450	3,410
1210	160	1,320	1,250	1,250
1200	49	300	290	290
1190	9	30	30	30
1180	1	2	2	2
1173	0	0	0	0

NOTES:

Reservoir areas were measured on a composite map prepared by the Hydraulic Data Branch with contours drawn at 10' intervals and scale 1"=500'. The map was prepared from TVA land maps with contours at elevations 1253, 1263, and 1268 and TVA topographic maps with contours at 20' intervals. Contours were made to conform to elevations on TVA sediment range cross sections which are located at one half to one mile intervals in the reservoir.

The original areas and volumes were computed by the contour method and volumes on succeeding dates were determinded by correcting the original volume by the volumes of sediment found on these dates. The volumes of sediment were determined by the constant factor method.

Elevations are referred to the 1936 Supplementary Adjustment of the U.S.C. & G.S.

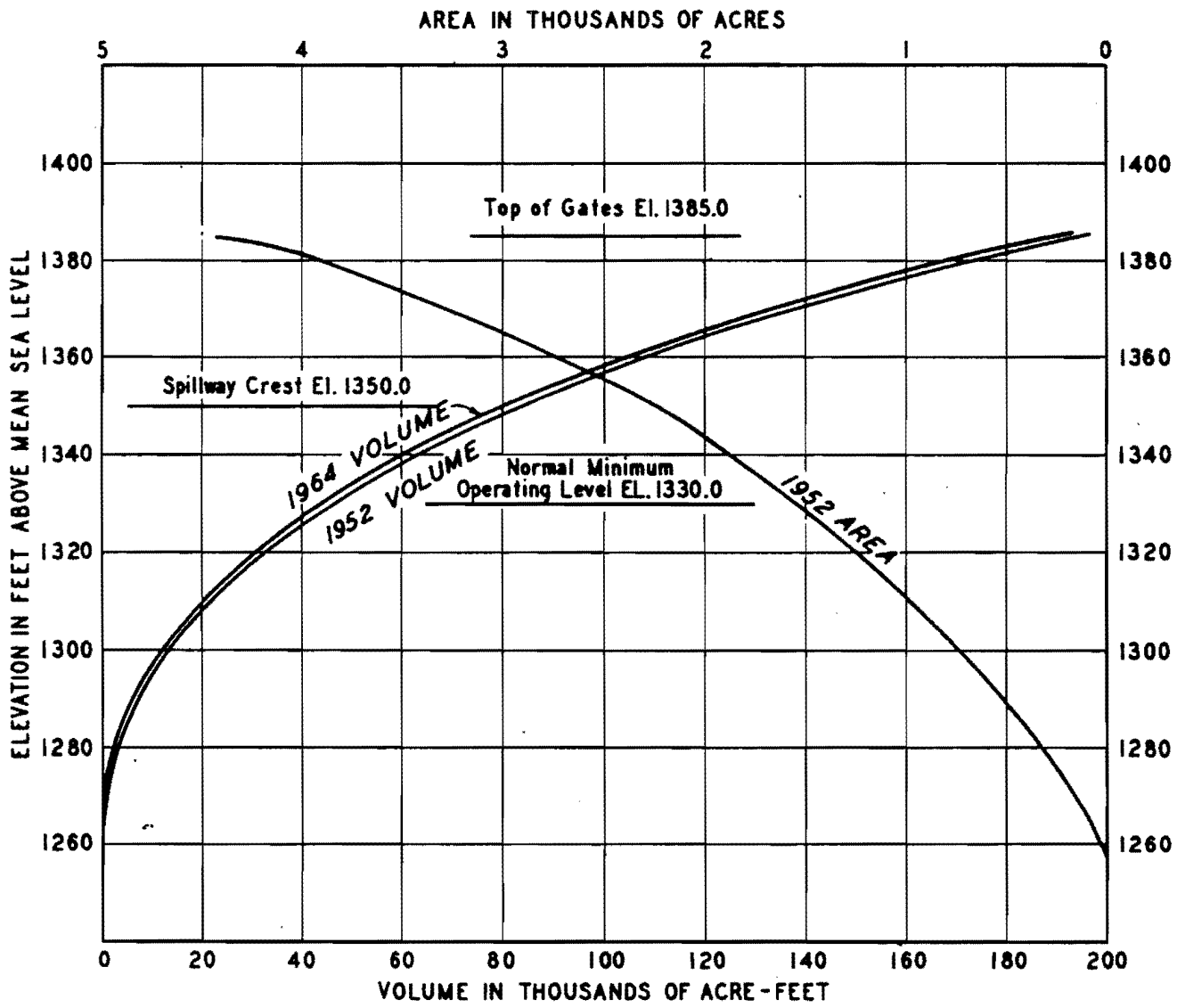
Area of original river within reservoir = 339 acres.
Drainage area at dam = 1,903 square miles.

Dam closure October 27, 1953.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

S.F.HOLSTON RIVER MILE 8.2
RESERVOIR AREAS
AND VOLUMES
Fort Patrick Henry Project
Figure 2.4-33

Figure 2.4-33 S.F. Holston River Mile 8.2 - Reservoir Areas and Volumes - Fort Patrick Henry Project



ELEV FT	1952 AREA AC	VOLUME		
		1952 AC-FT	1958 AC-FT	1964 AC-FT
1,385	4,400	196,668	195,633	193,446
1,380	3,946	175,501	174,487	172,318
1,370	3,317	139,215	138,219	136,260
1,360	2,742	108,998	108,063	106,346
1,350	2,229	84,309	83,484	81,987
1,340	1,885	63,779	63,053	61,778
1,330	1,547	46,661	46,083	44,898
1,320	1,257	32,690	32,275	31,352
1,310	974	21,660	21,354	20,568
1,300	738	13,142	12,932	12,305
1,290	513	6,957	6,867	6,382
1,280	309	2,938	2,832	2,561
1,270	156	705	675	581
1,260	19	49	38	24
1,254.9	0	0	0	0

NOTES:

Reservoir areas were measured on a composite map prepared by the Hydraulic Data Branch with contours drawn at 10' intervals and scale 1" = 500'. The map was prepared from TVA land maps with contours at elevations 1,350, 1,385 & 1,390 and TVA topographic maps with contours at 20' intervals. Contours were made to conform to elevations on TVA sediment range cross sections located at one-half to one-mile intervals. The 1952 volume was computed by the contour method. Volumes of sediment on succeeding dates were computed by the constant factor method.

Elevations are referred to the USC & GS 1936 Supplementary Adjustment.

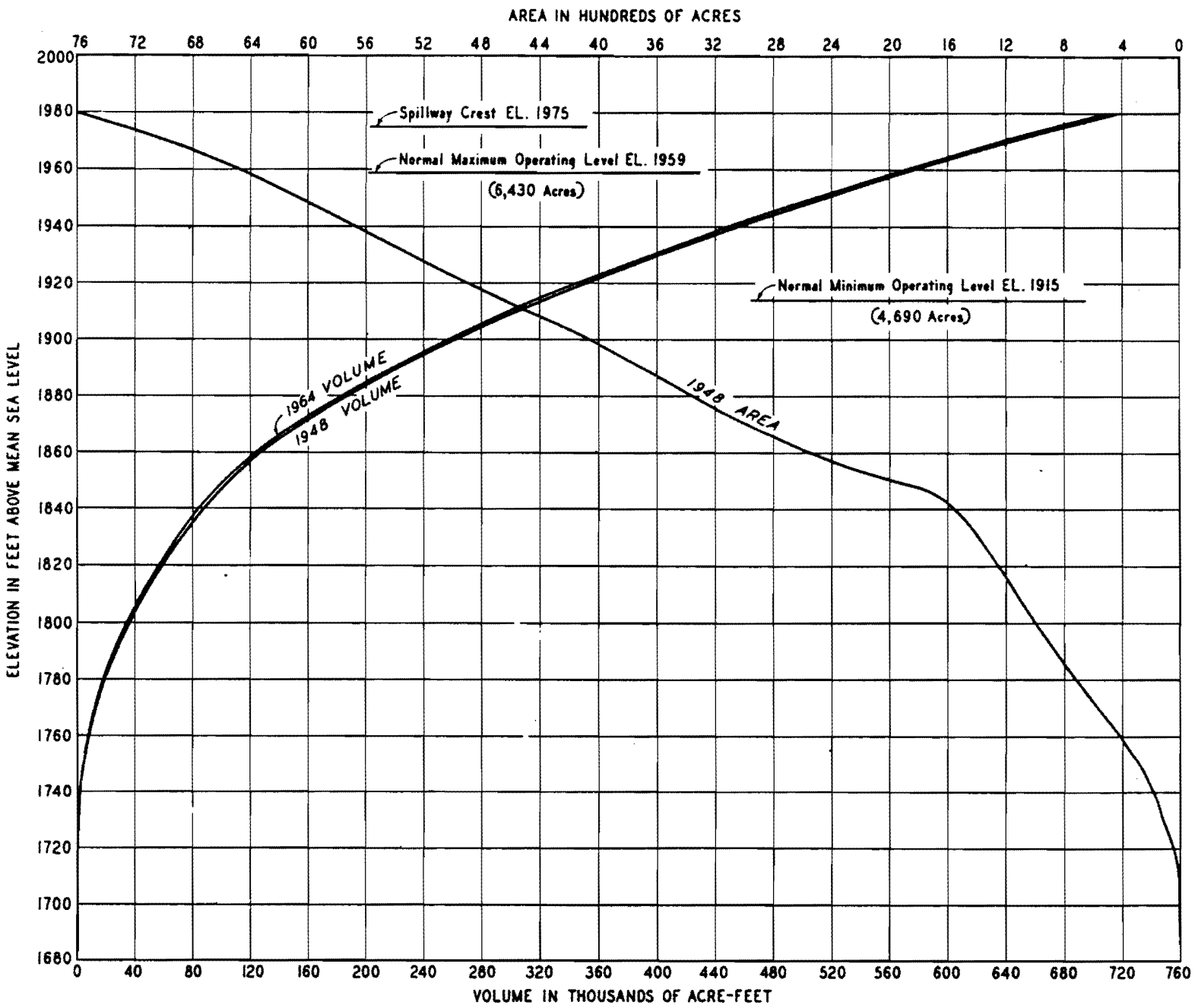
Area of original river within reservoir = 719 acres.

Drainage area at dam = 1,840 square miles.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

SOUTH FORK HOLSTON RIVER MILE 18.6
RESERVOIR AREAS
AND VOLUMES
Boone Project
Figure 2.4-34

Figure 2.4-34 South Fork Holston River Mile 18.6 - Reservoir Areas and Volumes - Boone Project



ELEV FT	1948 AREA. AC	VOLUME 1948 AC-FT	1953 AC-FT	1958 AC-FT	1964 AC-FT
1,980	7,600	718,000	717,000	716,000	714,000
1,975	7,300	681,000	680,000	679,000	677,000
1,970	6,930	646,000	645,000	643,000	642,000
1,960	6,480	579,000	578,000	576,000	575,000
1,950	6,070	516,000	515,000	514,000	512,000
1,940	5,680	457,000	456,000	455,000	454,000
1,930	5,310	402,000	401,000	400,000	399,000
1,920	4,900	351,000	350,000	349,000	348,000
1,910	4,500	304,000	303,000	302,000	301,000
1,900	4,070	262,000	261,000	260,000	258,000
1,890	3,710	223,000	222,000	221,000	220,000
1,875	3,200	171,000	170,000	169,000	168,000
1,860	2,560	128,000	127,000	127,000	126,000
1,850	2,000	105,000	104,000	104,000	103,000
1,840	1,550	87,200	86,800	86,600	85,900
1,830	1,390	72,500	72,100	72,000	71,300
1,820	1,250	59,300	59,000	58,900	58,200
1,810	1,130	47,400	47,100	47,000	46,300
1,795	929	32,100	31,800	31,700	31,000
1,780	715	19,800	19,500	19,500	18,800
1,770	572	13,300	13,200	13,200	12,700
1,760	411	8,440	8,370	8,380	7,980
1,750	285	4,950	4,940	4,920	4,580
1,740	183	2,610	2,610	2,600	2,390
1,730	113	1,140	1,120	1,120	990
1,720	38	379	374	378	300
1,710	11	134	129	134	76
1,685	0	0	0	0	0

NOTES:

Reservoir areas were measured on TVA land maps, scale 1:6,000, at elevations 1795, 1875, 1960, 1975 & 1980, and TVA topographic maps, scale 1:24,000, at elevations 1720, 1760, 1840 & 1920. Contours were made to conform to elevations on TVA sediment range cross sections which are located at one to three mile intervals within the reservoir.

The 1953 and subsequent volumes were determined by the constant factor method for computing sediment.

Elevations are referred to the 1936 Supplementary Adjustment of the U.S.C. & G.S.

Area of original river within the reservoir = 313 acres.

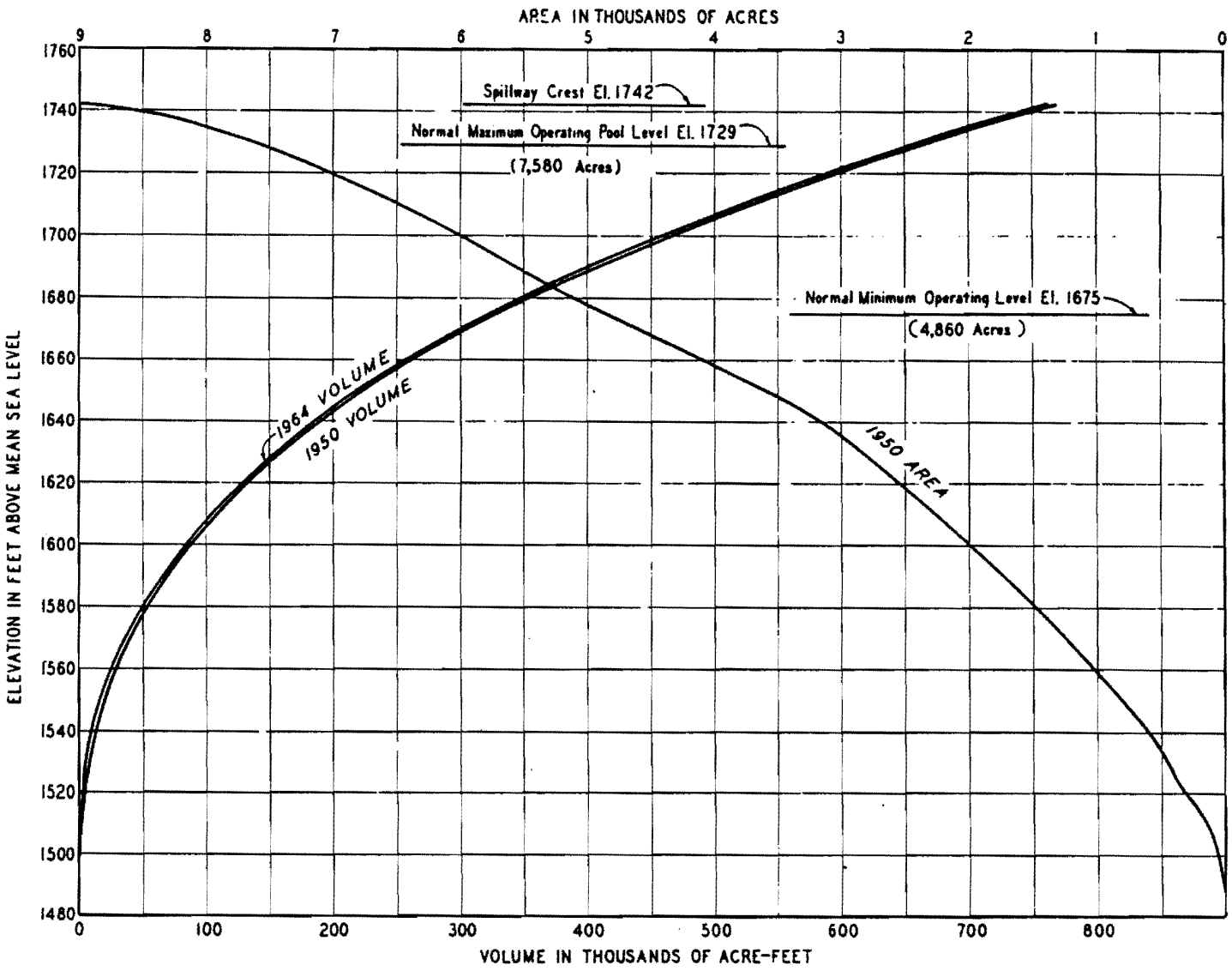
Drainage area above dam = 468 square miles.

Dam closure December 1, 1948.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

WATAUGA RIVER MILE 36.7
RESERVOIR AREAS
AND VOLUMES
Watauga Project
Figure 2.4-35

Figure 2.4-35 Watauga River Mile 36.7 - Reservoir Areas And Volumes - Watauga Project



ELEVATION FT	1950 AREA AC	VOLUME AC - FT		
		1950	1958	1964
1742	9,010	767,970	766,640	764,040
1730	7,660	669,520	668,190	665,590
1720	7,060	595,920	594,600	591,980
1710	6,500	528,140	526,850	524,180
1700	6,000	465,670	464,450	461,690
1690	5,550	407,900	406,720	403,910
1680	5,120	354,510	353,380	350,560
1670	4,630	305,770	304,750	302,020
1660	4,100	262,150	261,240	258,720
1650	3,590	223,720	222,880	220,540
1640	3,160	190,010	189,350	187,130
1630	2,820	160,150	159,660	157,500
1620	2,540	133,380	132,990	130,930
1610	2,260	109,410	109,130	107,190
1600	1,990	88,170	87,970	86,120
1590	1,730	69,580	69,430	67,740
1580	1,470	53,570	53,460	51,900
1570	1,250	39,990	39,860	38,430
1560	1,020	28,660	28,520	27,140
1550	800	19,580	19,470	18,290
1540	600	12,610	12,500	11,450
1530	450	7,450	7,390	6,430
1520	310	3,670	3,600	2,710
1510	140	1,390	1,340	974
1500	60	390	390	180
1490	10	30	30	0
1485	0	0	0	0

NOTES:

Reservoir areas were measured on TVA land maps, scale 1" = 500', at elevations 1640, 1730, & 1742, and on TVA topographic maps, scale 1"=2,000', at elevations 1500, 1520, 1540, & 1560. The TVA maps were prepared by the Maps & Surveys Branch. Contours were made to conform to elevations on TVA sediment range cross sections which are at one-half to two mile intervals within the reservoir.

Elevations are referred to the 1936 Supplementary Adjustment of the USC & GS.

Area of the original river within reservoir = 710 acres.

Drainage area above dam =703 square miles.

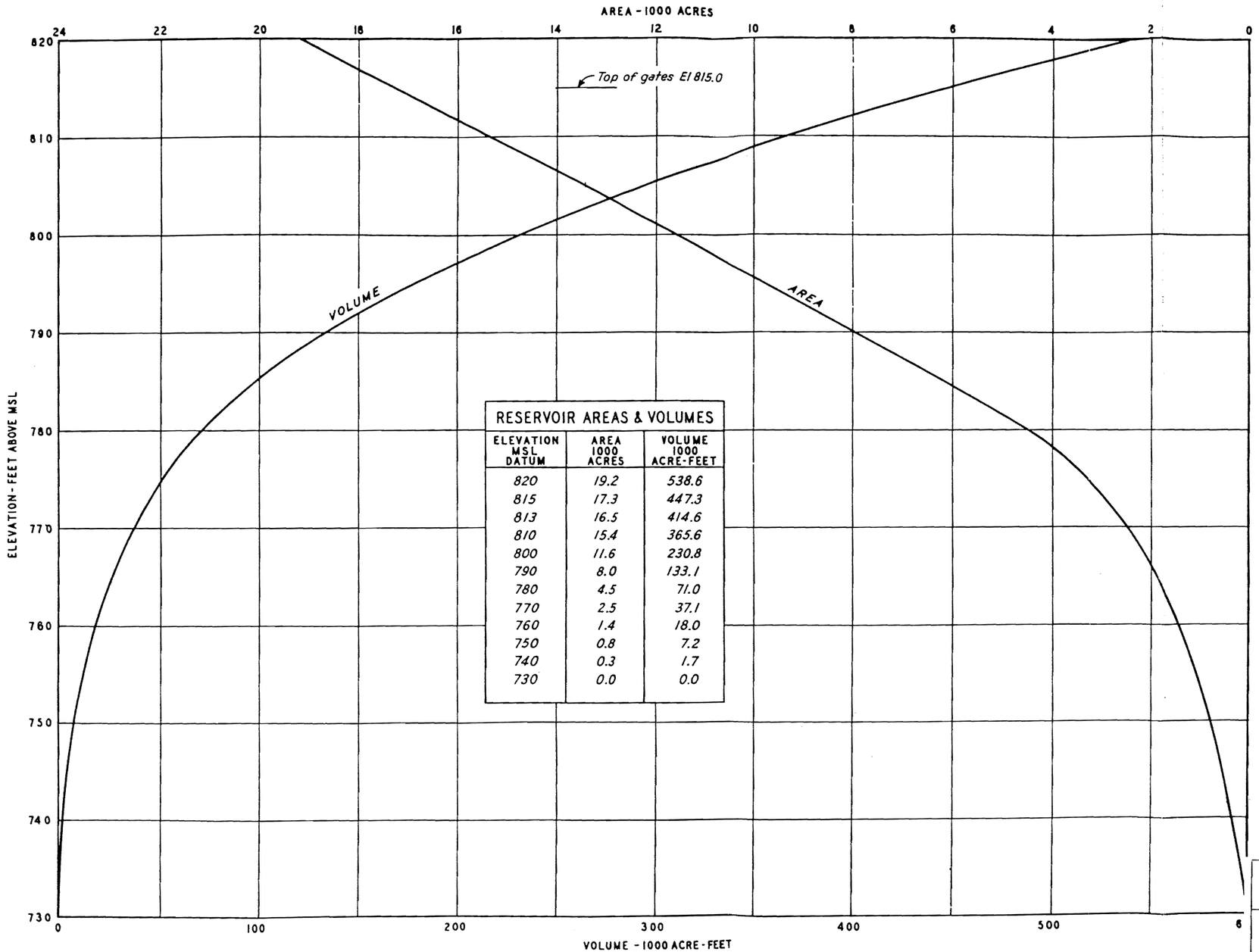
The 1958 and subsequent volumes were determined by the constant factor method for computing sediment.

Dam closure November 20, 1950.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

S.F. HOLSTON RIVER MILE 49.8
RESERVOIR AREAS
AND VOLUMES
South Holston Project
Figure 2.4-36

Figure 2.4-36 S.F. Holston River Mile 49.8 - Reservoir Areas And Volumes - South Holston Project



RESERVOIR AREAS & VOLUMES

ELEVATION MSL DATUM	AREA 1000 ACRES	VOLUME 1000 ACRE-FEET
820	19.2	538.6
815	17.3	447.3
813	16.5	414.6
810	15.4	365.6
800	11.6	230.8
790	8.0	133.1
780	4.5	71.0
770	2.5	37.1
760	1.4	18.0
750	0.8	7.2
740	0.3	1.7
730	0.0	0.0

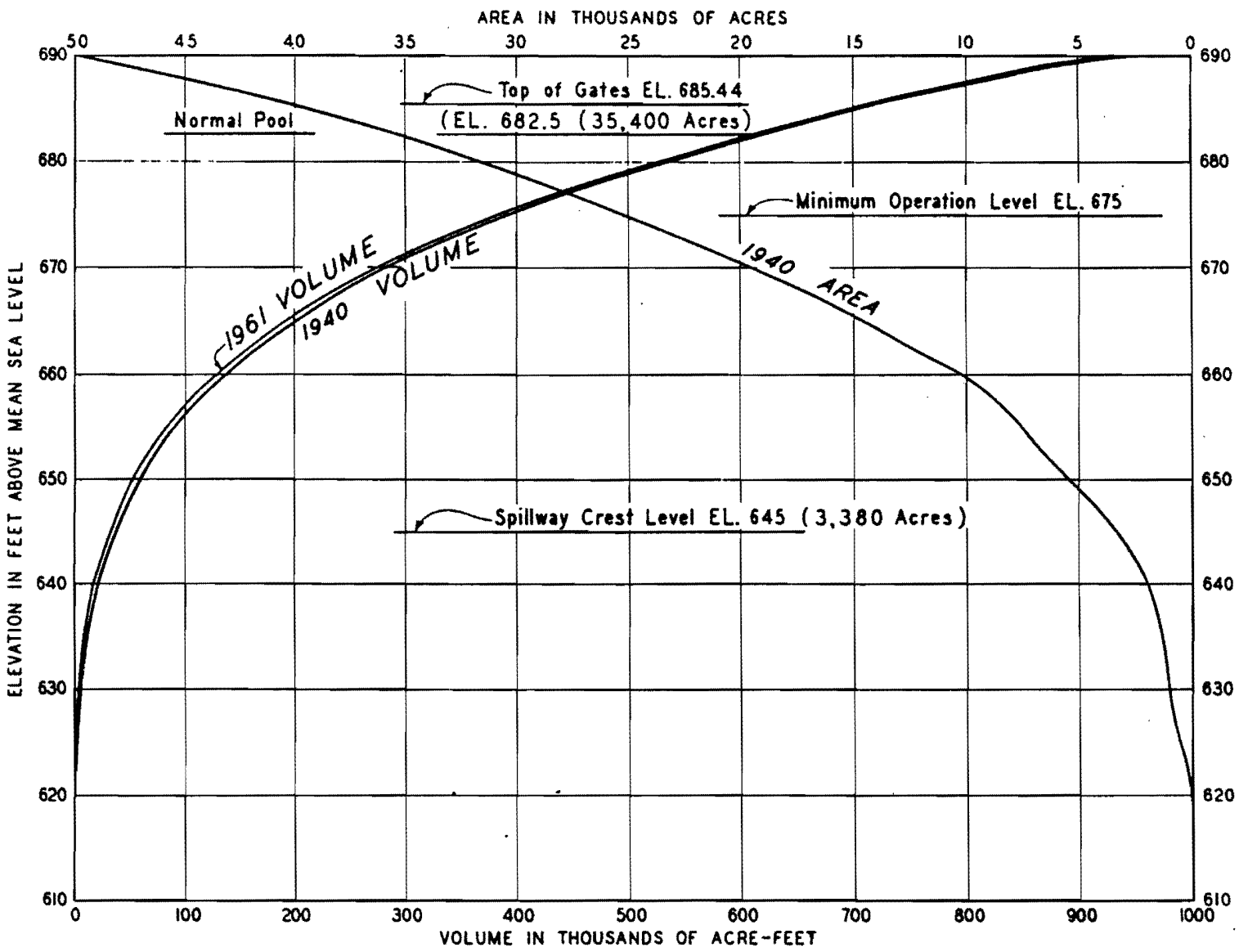
NOTES:
Areas obtained by planimeter from USGS-TVA topographic maps, scale 1"=2000'; contour interval 20'.
Drainage area at site = 2,627 square miles.
Area of original river within reservoir to Chilhowee Dam = 2,133 acres.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

LITTLE TENNESSEE RIVER MILE 0.3
RESERVOIR AREAS
AND VOLUMES
Tellico Project

Figure 2.4-37

Figure 2.4-37 Little Tennessee River Mile 0.3 - Reservoir Areas And Volumes - Tellico Project



ELEV FT	1940 AREA AC	VOLUME 1940 AC-FT	1947 AC-FT	1954 AC-FT	1956 AC-FT	1961 AC-FT
690	49,800	951,000	939,000	950,000	945,000	944,000
685.44	40,800	747,000	735,000	745,000	740,000	739,000
685	39,900	727,000	715,000	725,000	721,000	719,000
680	32,200	547,000	535,000	544,000	539,000	537,000
675	25,600	402,000	390,000	398,000	394,000	392,000
669	18,800	269,000	260,000	265,000	263,000	261,000
662	12,400	160,000	153,000	156,000	155,000	153,000
660	10,500	137,000	131,000	133,000	132,000	130,000
650	5,720	57,700	53,700	54,700	53,600	52,200
640	2,020	20,000	18,600	18,600	18,600	18,000
630	1,140	4,740	4,020	3,970	3,950	3,660
620	104	135	94	66	47	88
617	0	0	0	0	0	0

NOTES:

Reservoir areas at elevation 660 and below were measured on a composite map prepared by the Hydraulic Data Branch with contours drawn at 10' intervals. The map was prepared from Tennessee River Survey Maps made by the U. S. Army Engineers, with contours at 620, 630, 640, 650 and 660. Contours were made to conform to elevations on TVA sediment range cross sections located at one to five mile intervals. Areas above elevation 660 were measured on TVA navigation maps at elevations 662, 669, 675, 680, 685 and 690 from TVA navigation maps adjusted to agree with the area at elevation 683 from TVA land maps.

The 1940 volume was computed by the contour method. Volumes of sediment on succeeding dates were computed by the constant factor method.

Elevations are referred to the USC & GS 1936 Supplementary Adjustment.

Area of original river within reservoir = 9,500 acres.

Drainage area at dam = 20,790 square miles.

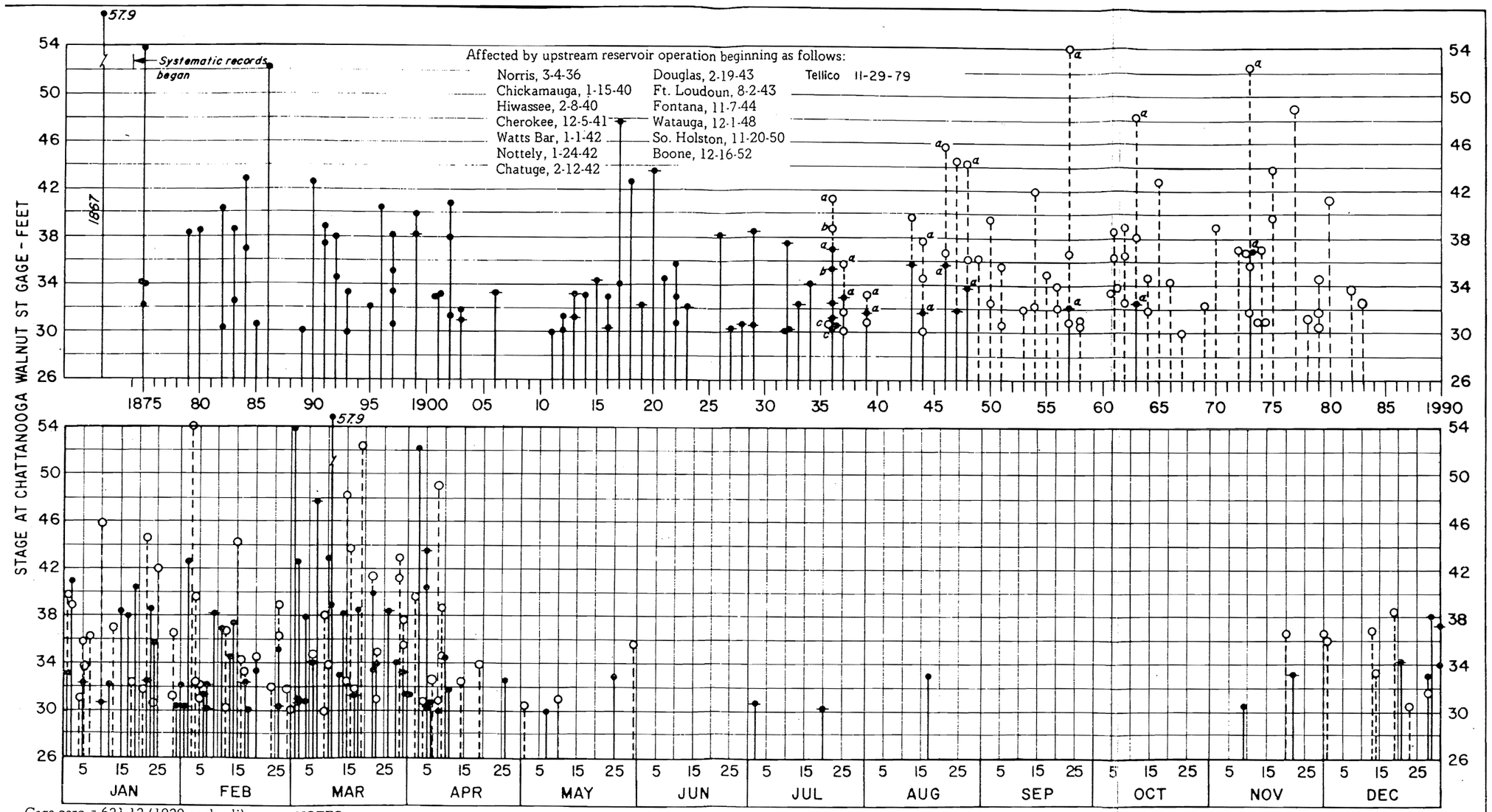
WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

TENNESSEE RIVER MILE 471.0
RESERVOIR AREAS
AND VOLUMES
Chickamauga Project

Figure 2.4-38

Figure 2.4-38 Tennessee River Mile 471.0 - Reservoir Areas And Volumes - Chickamauga Project

Figure 2.4-39 Deleted by Amendment 63



Gage zero = 621.12 (1929 genl. adj)
USWB flood stage = 30 feet
Drainage area = 21,400 sq mi

NOTES:

The stages shown for the 1867 flood and the highest flood in 1875 are from well authenticated flood marks. All other stages, except in 1874 and 1875, are reported (◆) or estimated (●) crests from observations at the same datum and location, Walnut Street. The stage readings in 1874 and 1875 were based on a different datum, but the estimated crests were corrected to be comparable to later records. Stages since October 22, 1913 are not comparable with earlier ones because of the backwater effect of Hales Bar Dam, 35 miles downstream. A change in Hales Bar Spillway in 1948, the closure of Nickajack Dam in December 1967, and subsequent removal of Hales Bar Dam further affected Chattanooga stages, making later stages incomparable to earlier periods.

Since March 4, 1936, when upstream regulation began, both computed natural (O) and reported crests are shown on the yearly chart. These natural crests are based on conditions when TVA was established, and hence are comparable to stages from 1913 to 1948. Only natural crests since March 1936 are shown on the seasonal diagram.

Revised by Amendment 50

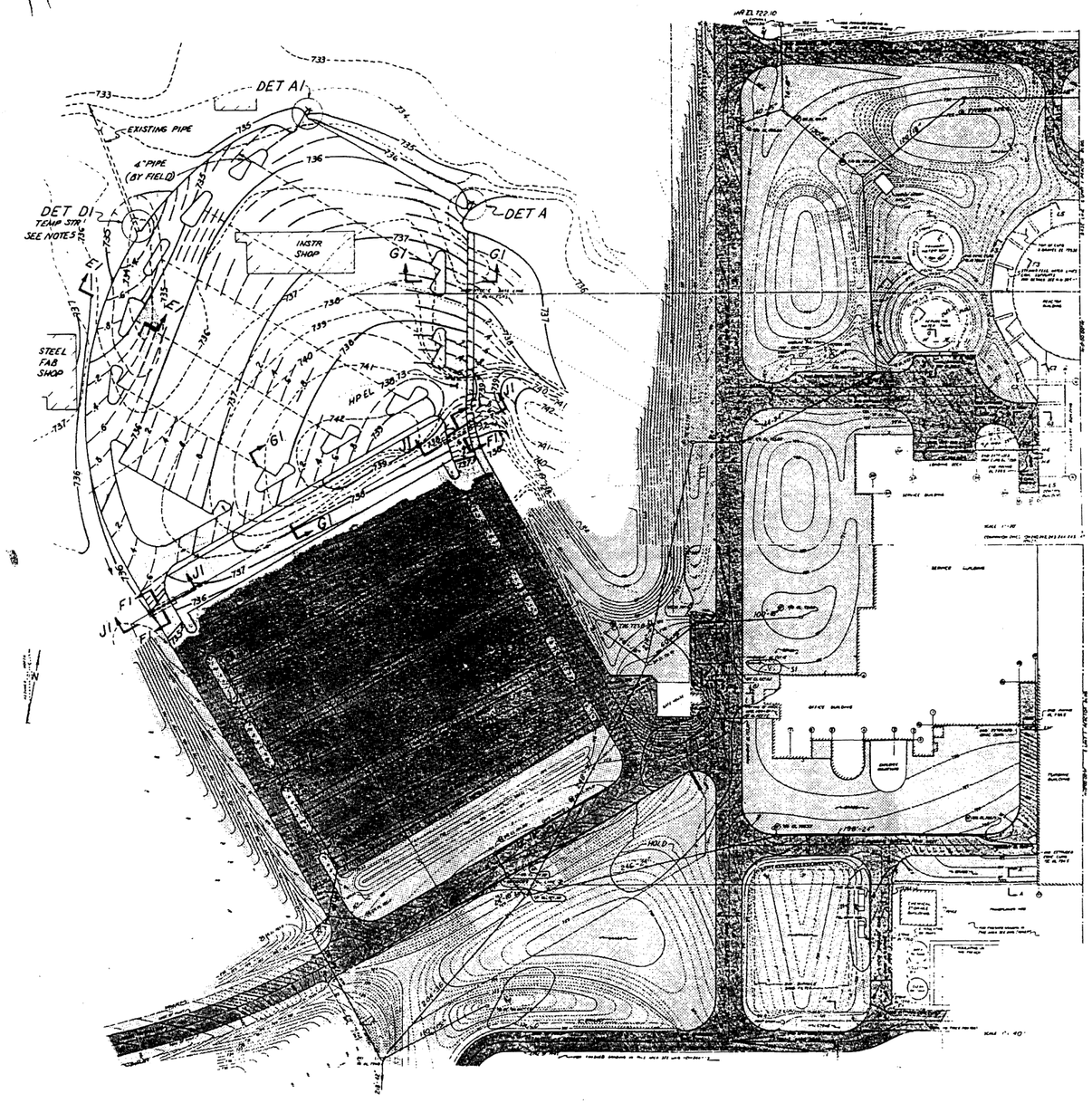
WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

TENNESSEE RIVER MILE 464.2
DISTRIBUTION OF FLOODS
AT CHATTANOOGA, TENN.
Figure 2.4 - 40

Figure 2.4-40 Tennessee River Mile 464.2 - Distribution Of Floods At Chattanooga, Tennessee

Figure 2.4-40a Main Plant Site Grading And Drainage System For Flood Studies Sheet 1

Figure 2.4-40a Main Plant Site Grading and Drainage System For Flood Studies Sheet 2



Added by Amendment 50

LEGEND
PAVED AREAS
GRASSED AREAS

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MAIN PLANT
SITE GRADING AND DRAINAGE
SYSTEM FOR FLOOD STUDIES
SHEET 3
FIGURE 2.4-40a

Figure 2.4-40a Main Plant Site Grading and Drainage System For Flood Studies Sheet 3

Figure 2.4-40b Main Plant General Plan

Figure 2.4-40c Yard Site Grading and Drainage System For Flood Studies

Figure 2.4-40d-1 Main Plant Plant Perimeter Roads Plan and Profile Sheet 1

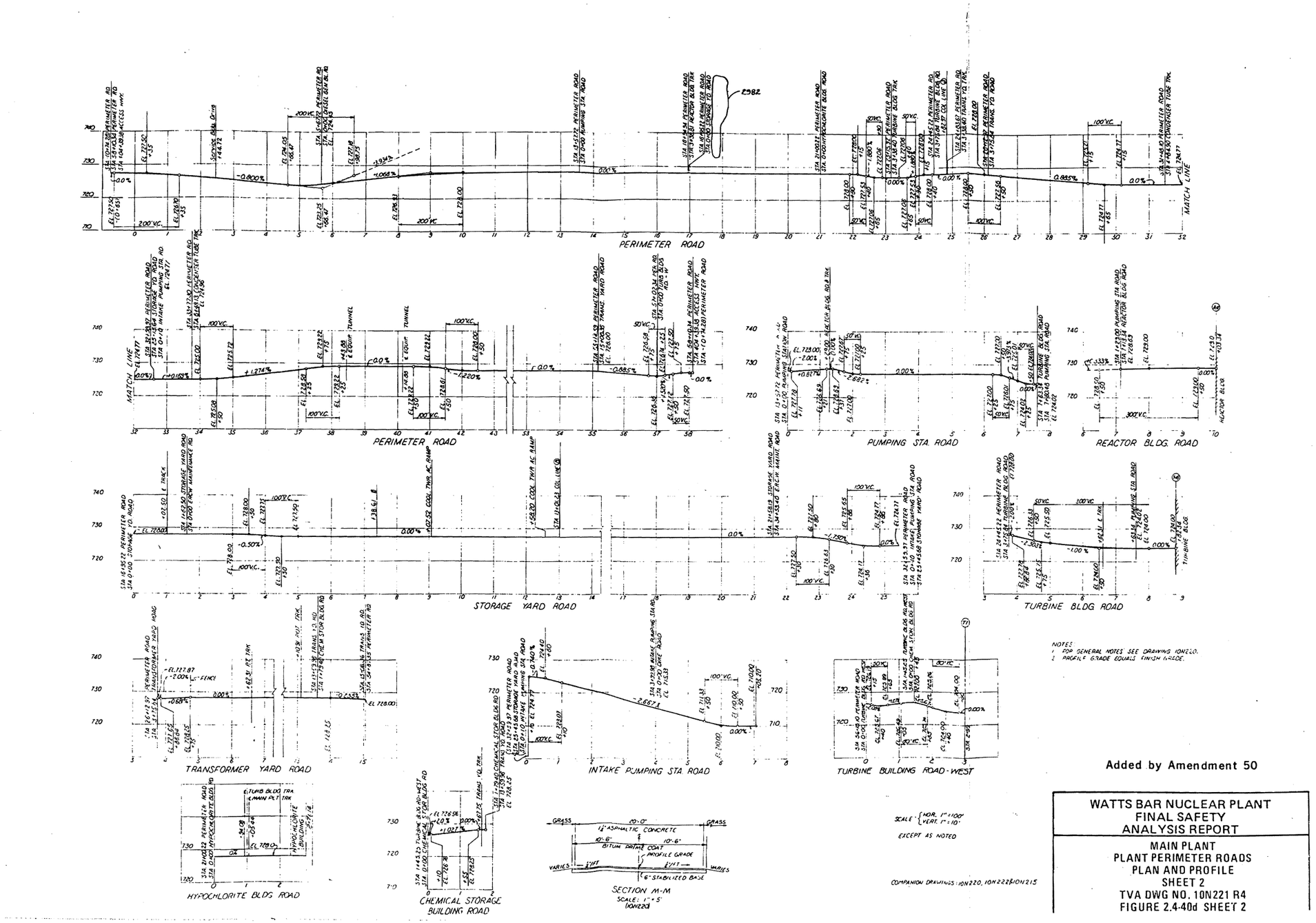
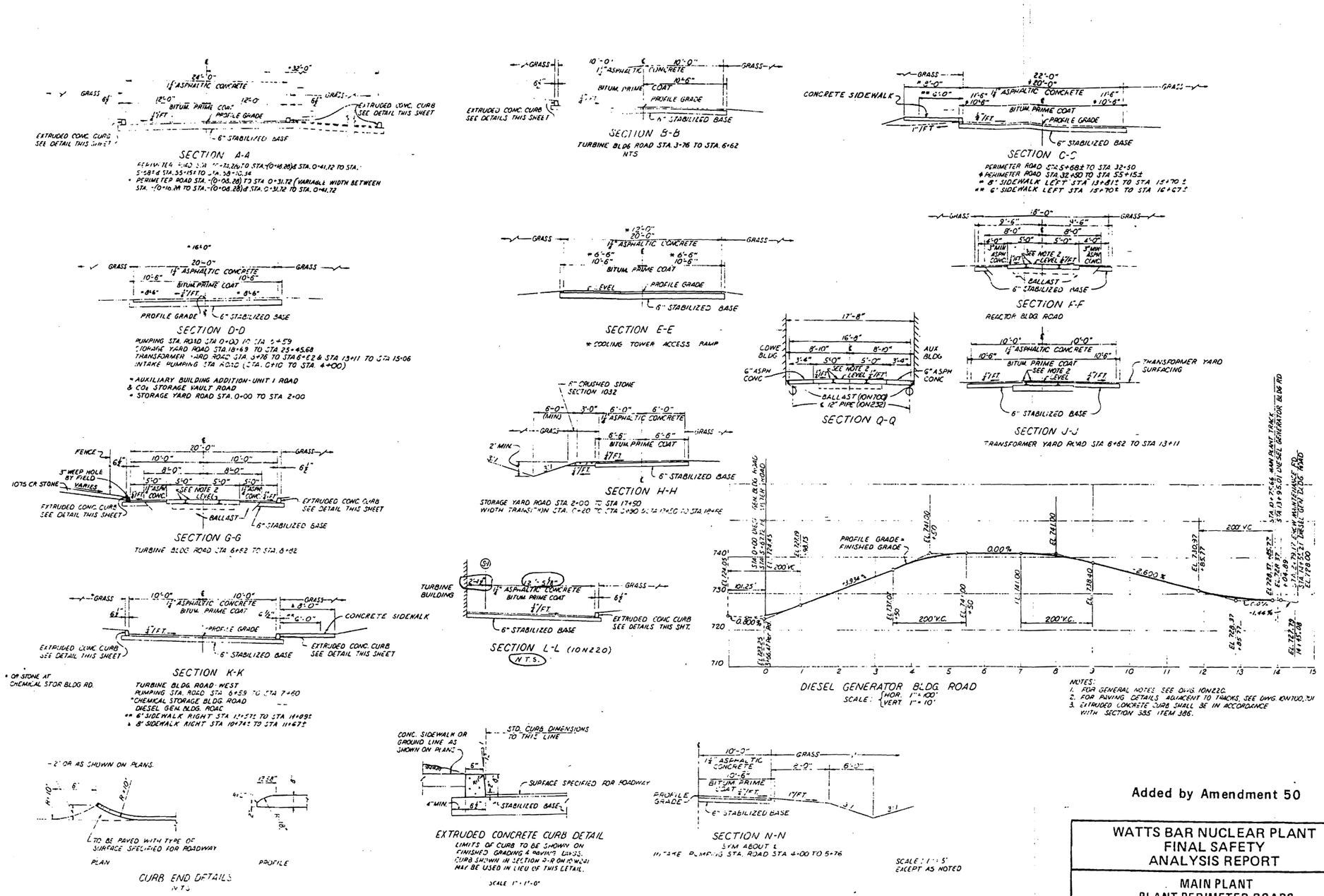


Figure 2.4-40d Main Plant Plant Perimeter Roads Plan and Profile - Sheet 2



Added by Amendment 50

WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
MAIN PLANT PLANT PERIMETER ROADS PLAN AND PROFILE SHEET 3 TVA DWG NO. 10N222 R10 FIGURE 2.4-40d SHEET 3

Figure 2.4-40d Main Plant Plant Perimeter Roads Plan and Profile - Sheet 3

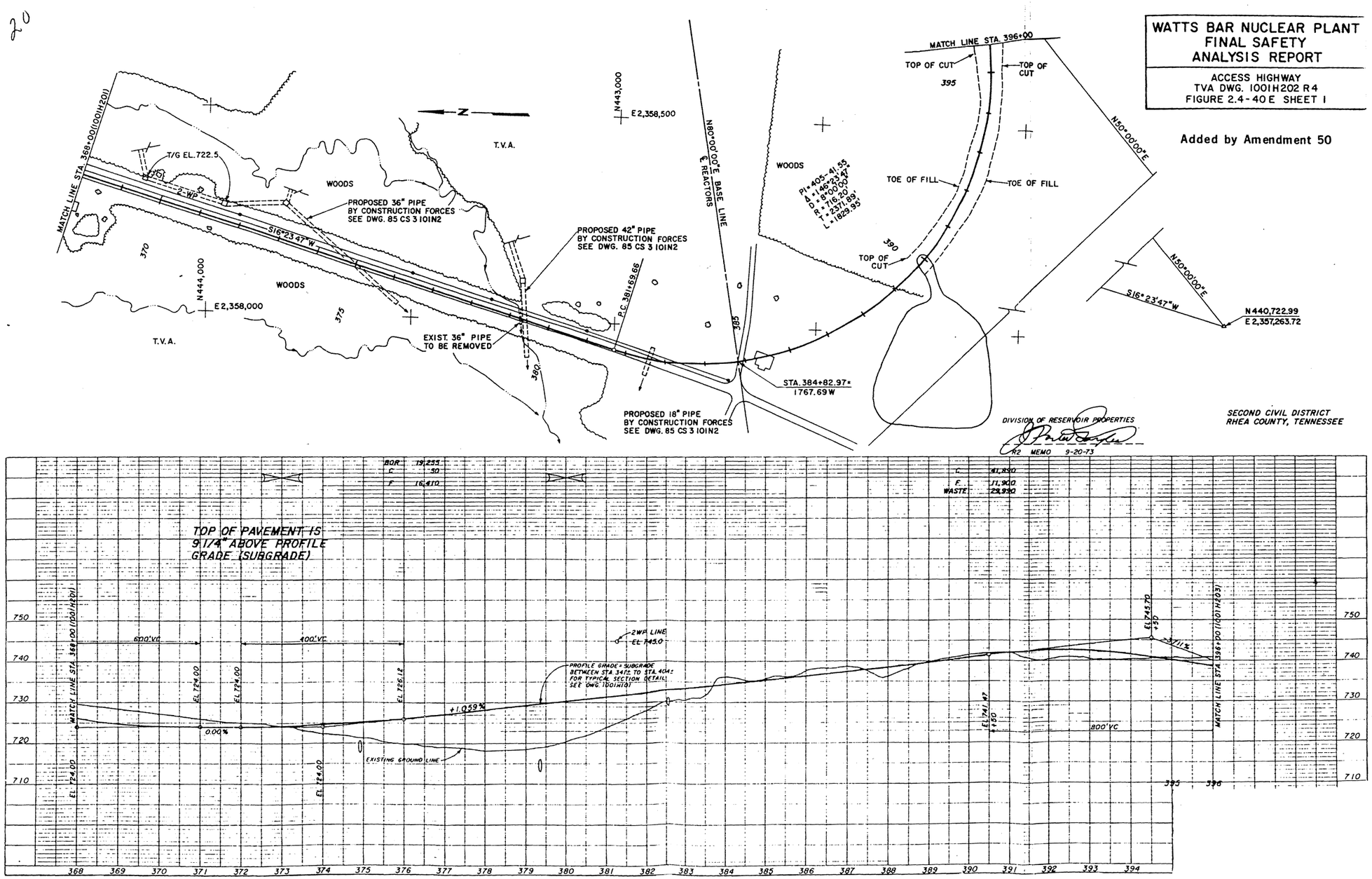


Figure 2.4-40e Access Highway TVA DWG 1001H202 R4 - Sheet 1

Added by Amendment 50

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

ACCESS HIGHWAY
TVA DWG. 1001H201 R4
FIGURE 2.4-40E SHEET 2

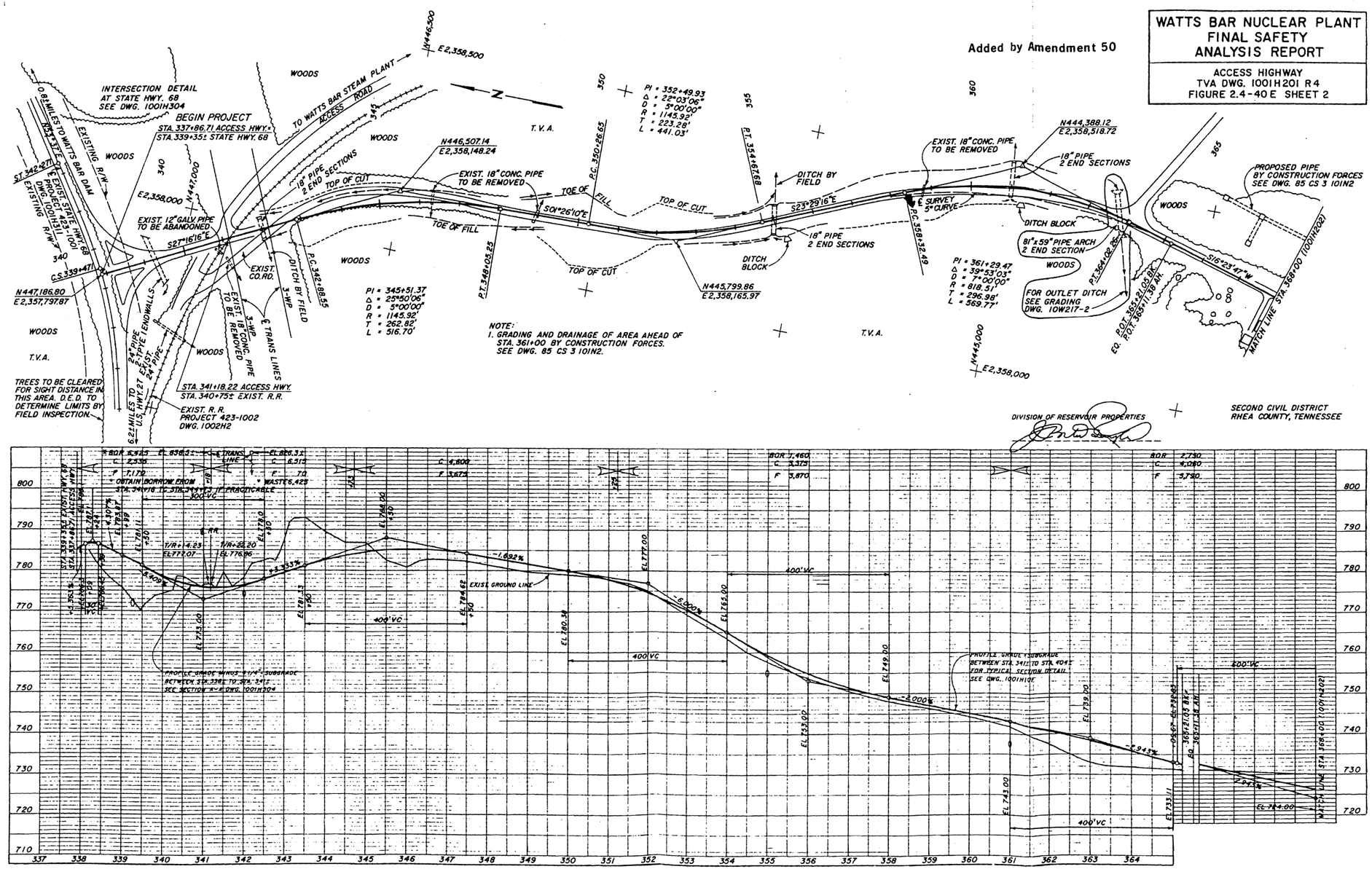


Figure 2.4-40e Access Highway TVA DWG. 1001H201 R4 - Sheet 2

Figure 2.4-40f Main Plant Main Plant Tracks Plan - Sheet 1

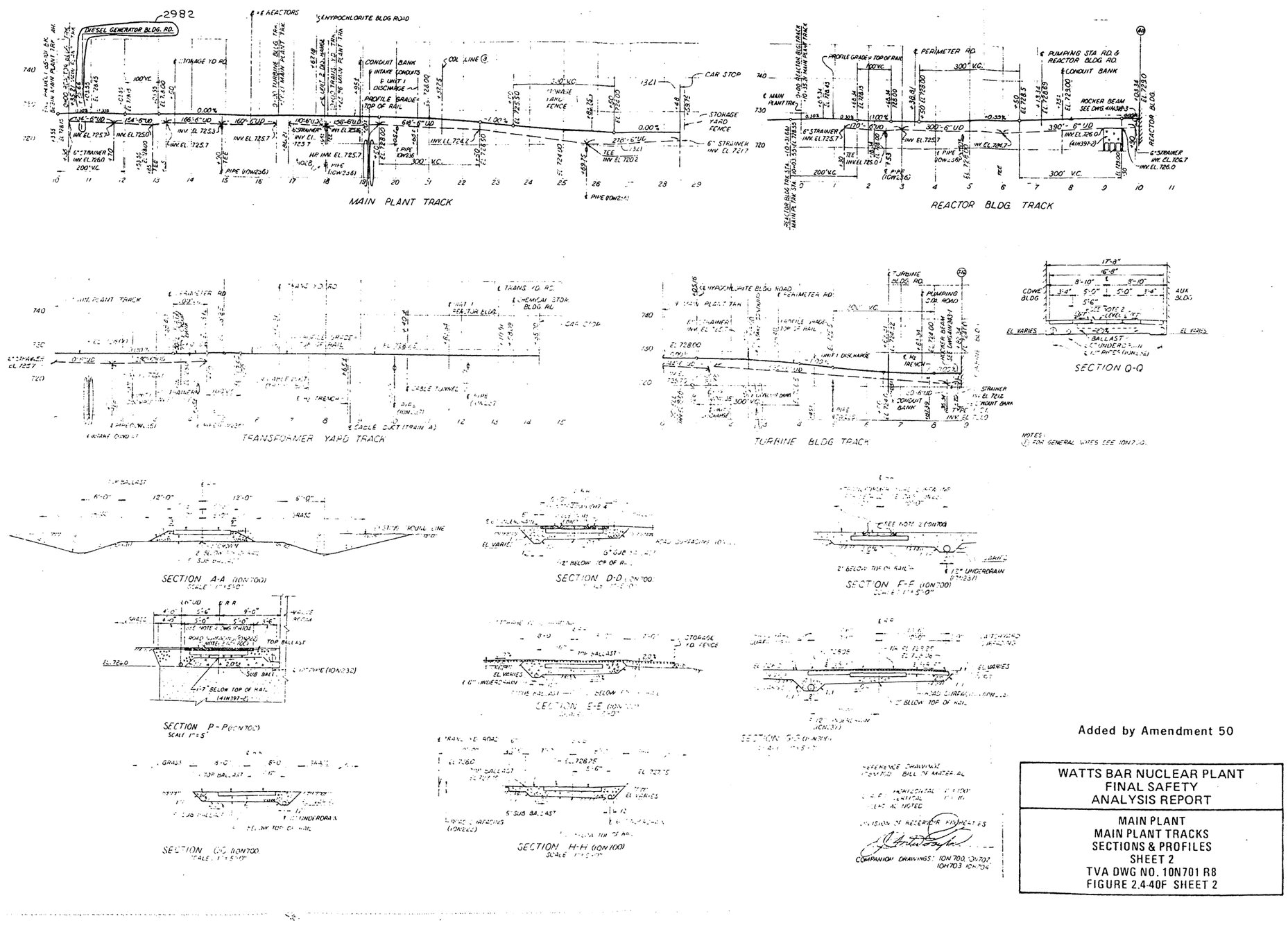
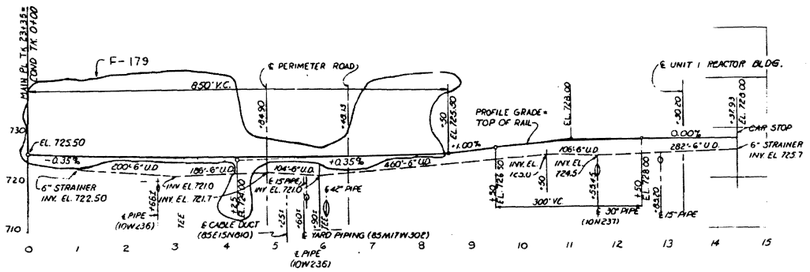
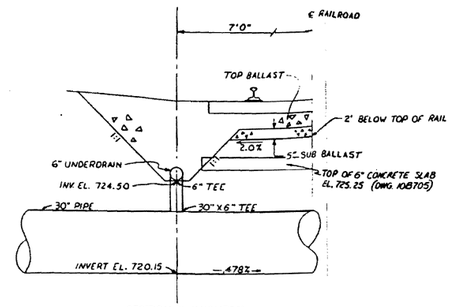


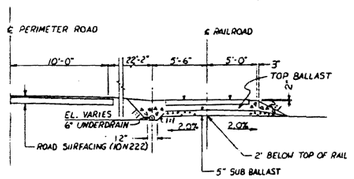
Figure 2.4-40f Main Plant Main Plant Tracks Sections & Profiles - Sheet 2



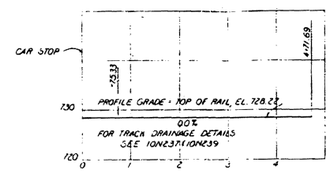
CONDENSER TUBE TRACK
SCALE: 1" = 100' HORIZONTAL
1" = 10' VERTICAL



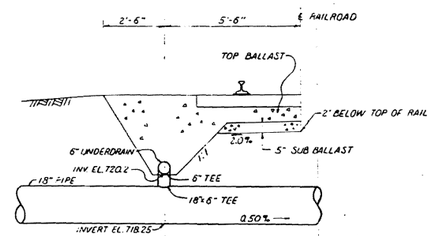
SECTION K-K (10N700)
SCALE: 1" = 2'-0"



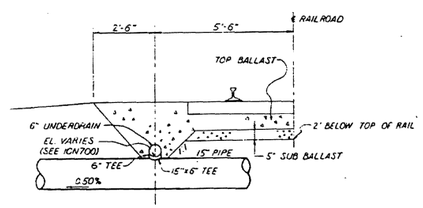
SECTION J-J (10N700)
SCALE: 1" = 5'-0"



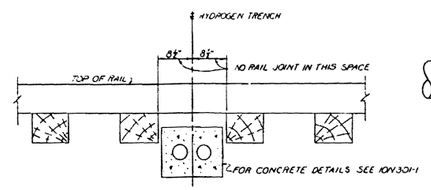
CONDENSER TUBE ACCESS TRACK
SCALE: 1" = 100' HORIZONTAL
1" = 10' VERTICAL



SECTION L-L (10N700)
SCALE: 1" = 2'-0"

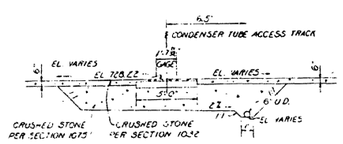


SECTION M-M (10N700)
SCALE: 1" = 2'-0"



DETAIL A
SCALE: 1" = 1'-0"

NOTES:
1. FOR GENERAL NOTES SEE 10N700.



SECTION N-N (10N700)
SCALE: 1" = 5'-0"

Added by Amendment 50

SCALE AS SHOWN

COMPANION DRAWINGS: 10N700, 10N701, 10N703, 10N704, 10N705, 10N237, 10N239

WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
MAIN PLANT MAIN PLANT TRACKS SECTIONS & PROFILES SHEET 3 TVA DWG NO. 10N702 R3 FIGURE 2.4-40F SHEET 3

Figure 2.4-40f Main Plant Main Plant Tracks Sections & Profiles - Sheet 3

Figure 2.4-40g Yard, Grading Drainage and Surfacing Transformer & Switchyard - Sheet 1

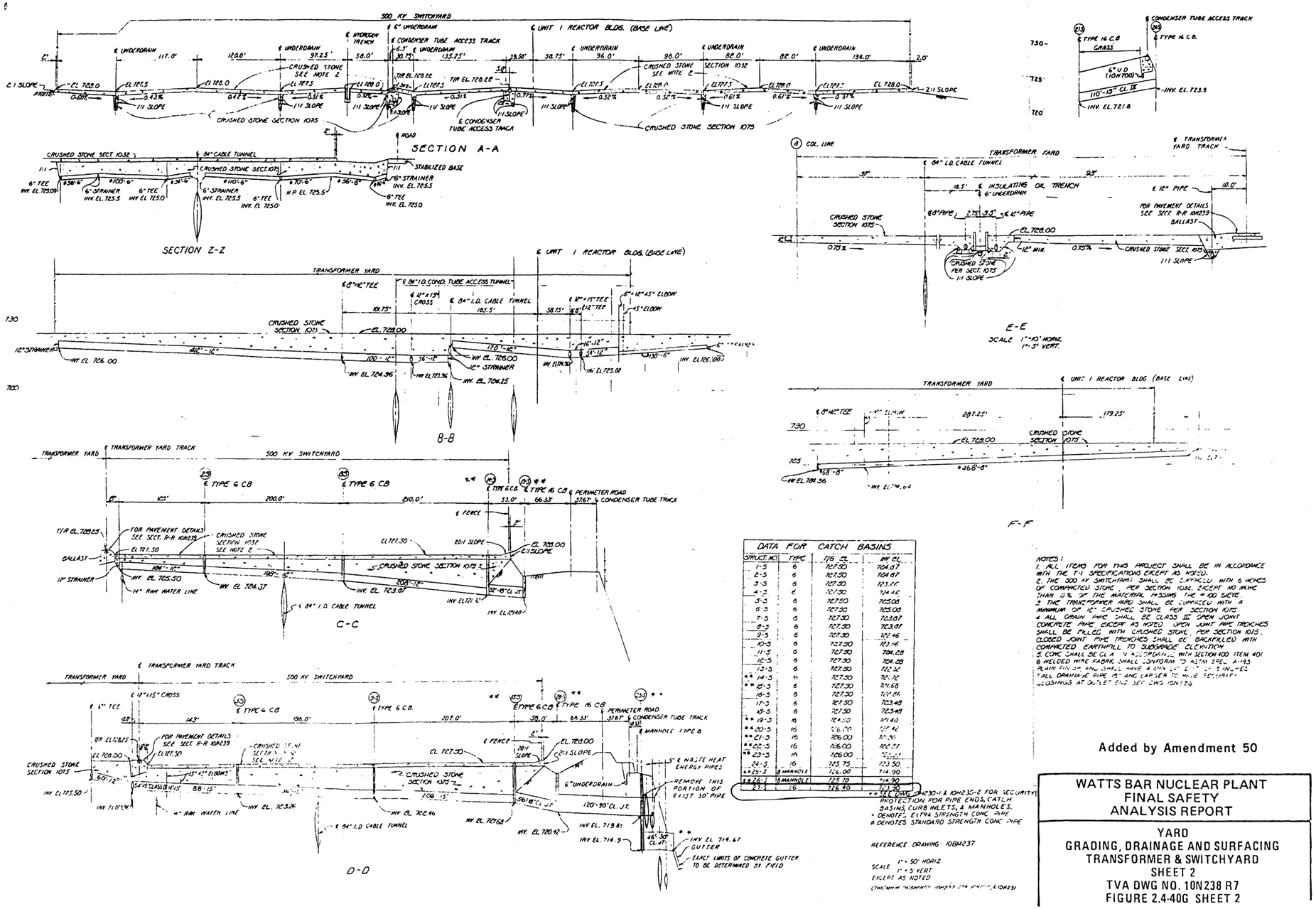


Figure 2.4-40g Yard, Grading Drainage and Surfacing Transformer & Switchyard - Sheet 2

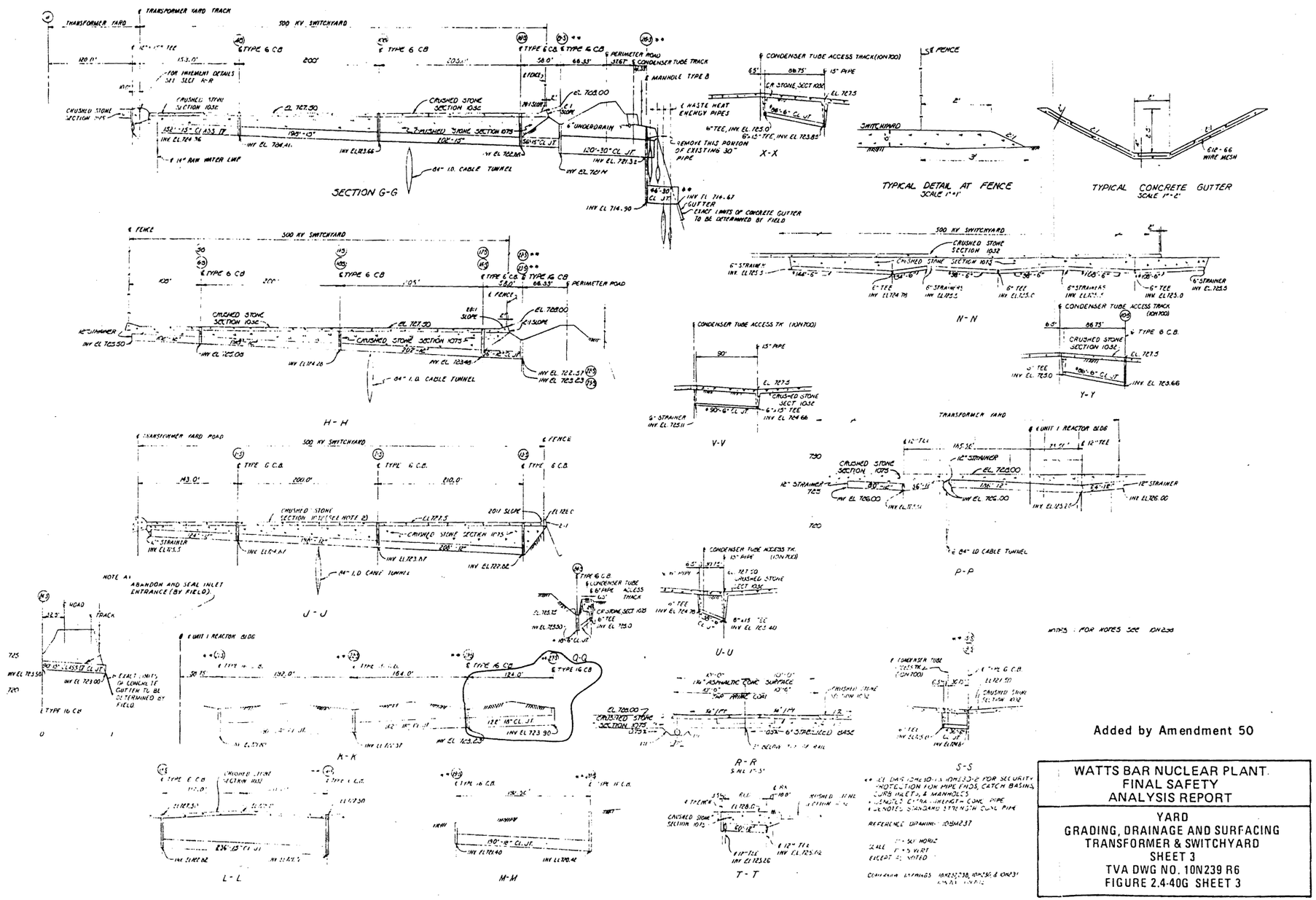


Figure 2.4-40g Yard, Grading Drainage and Surfacing Transformer & Switchyard - Sheet 3

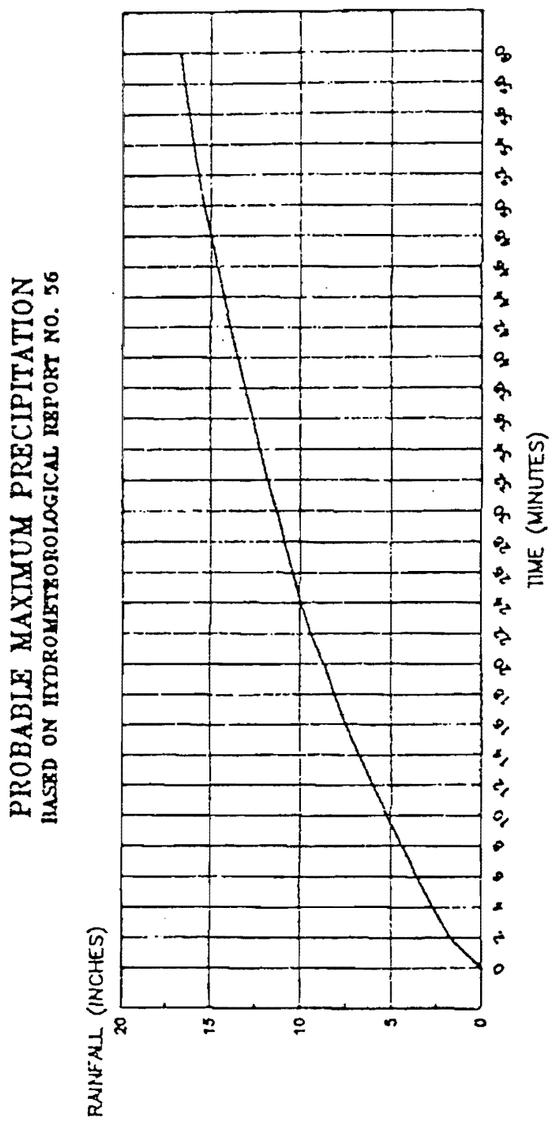


Figure 2.4-40h Probable Maximum Precipation Point Rainfall

AMENDMENT 83

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

PROBABLE MAXIMUM PRECIPATION
POINT RAINFALL

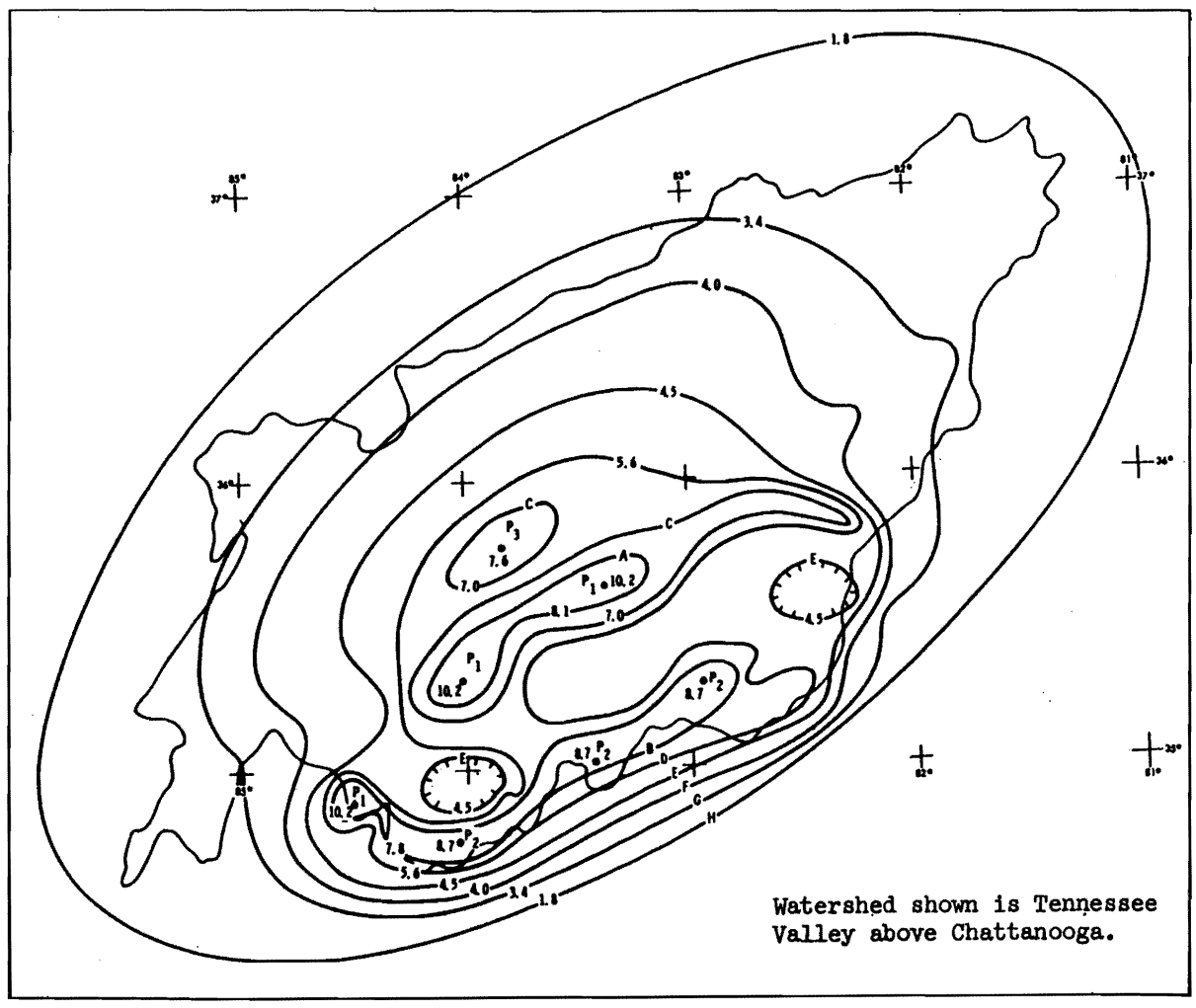
FIGURE 2.4-40h

Figure 2.4-40i Deleted by Amendment 83

Figure 2.4-40j Deleted by Amendment 83

Figure 2.4-40k Deleted by Amendment 83

Figure 2.4-40L Deleted by Amendment 83



WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

PROBABLE MAXIMUM
MARCH ISOHYETS
(21,400-sq. mi. downstream)
1st 6 HOURS (IN.)

Figure 2.4-41

Figure 2.4-41 Probable Maximum March Isohyets (21,400-sq. mi. downstream) 1st 6 Hours (IN.)

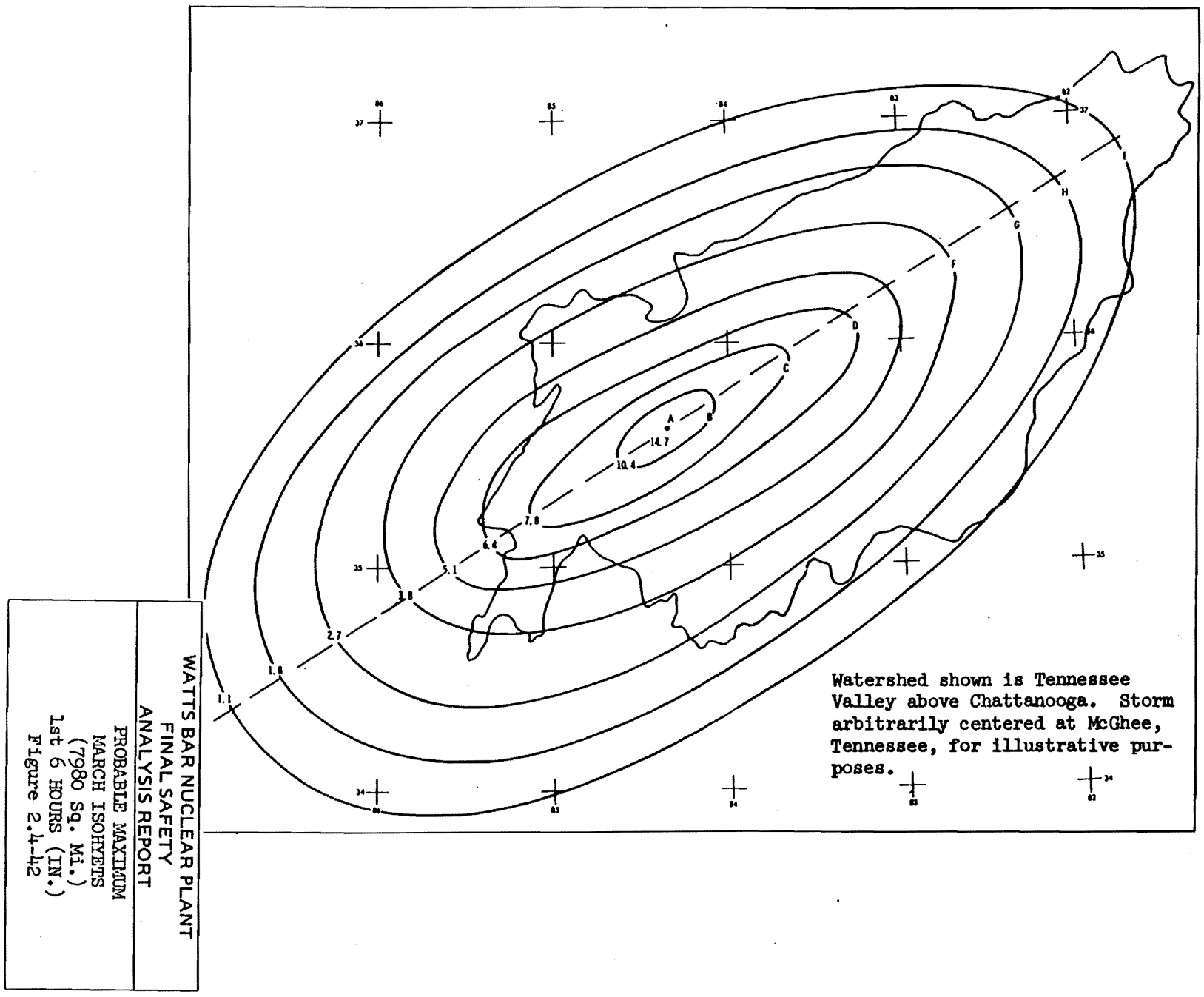
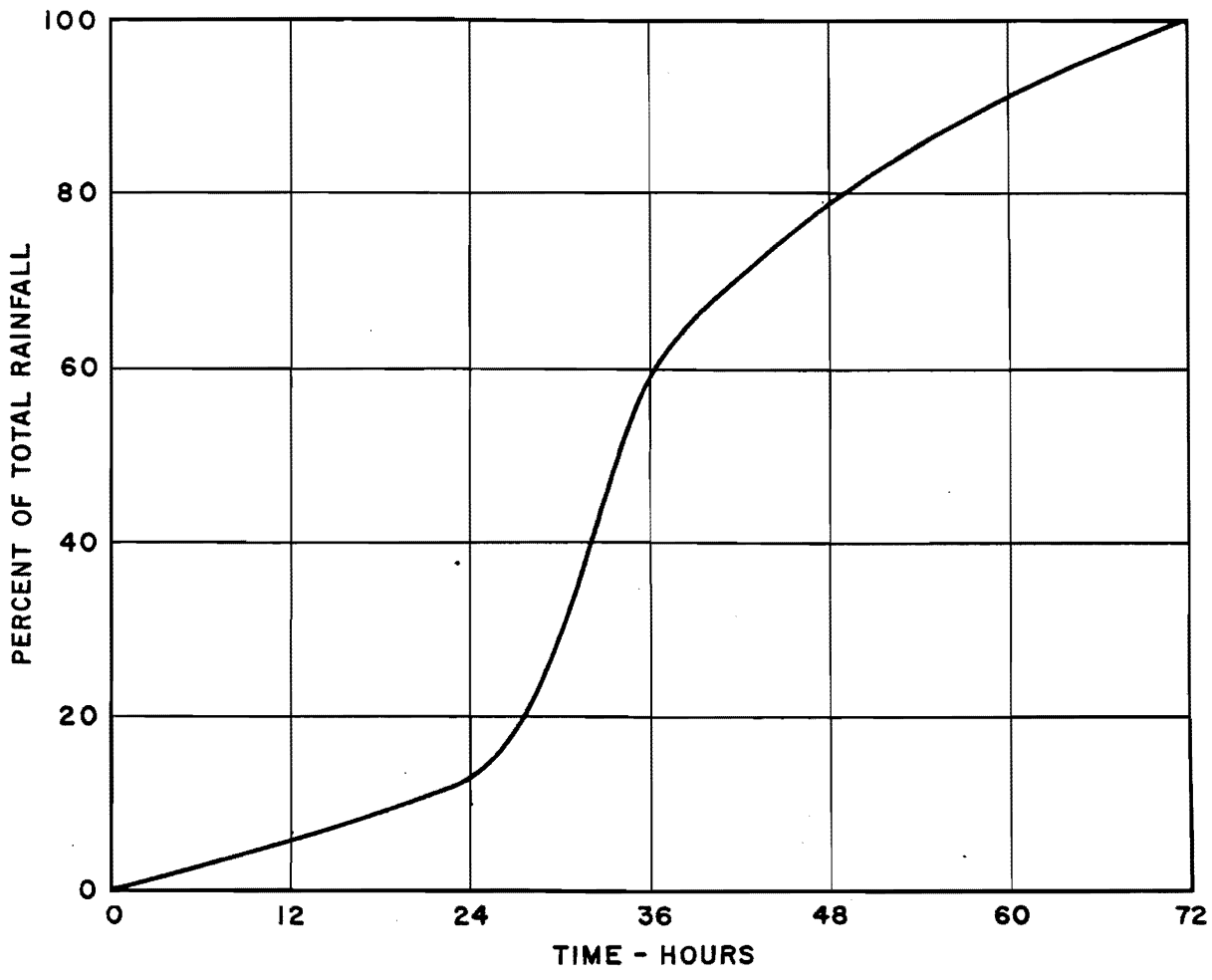


Figure 2.4-42 Probable Maximum March Isohyets (7980 Sq. Mi.) 1st 6 Hours (IN.)



RAINFALL TIME DISTRIBUTION
ADOPTED STANDARD MASS CURVE

Figure 2.4-43

Figure 2.4-43 Rainfall Time Distribution Adopted Standard Mass Curve

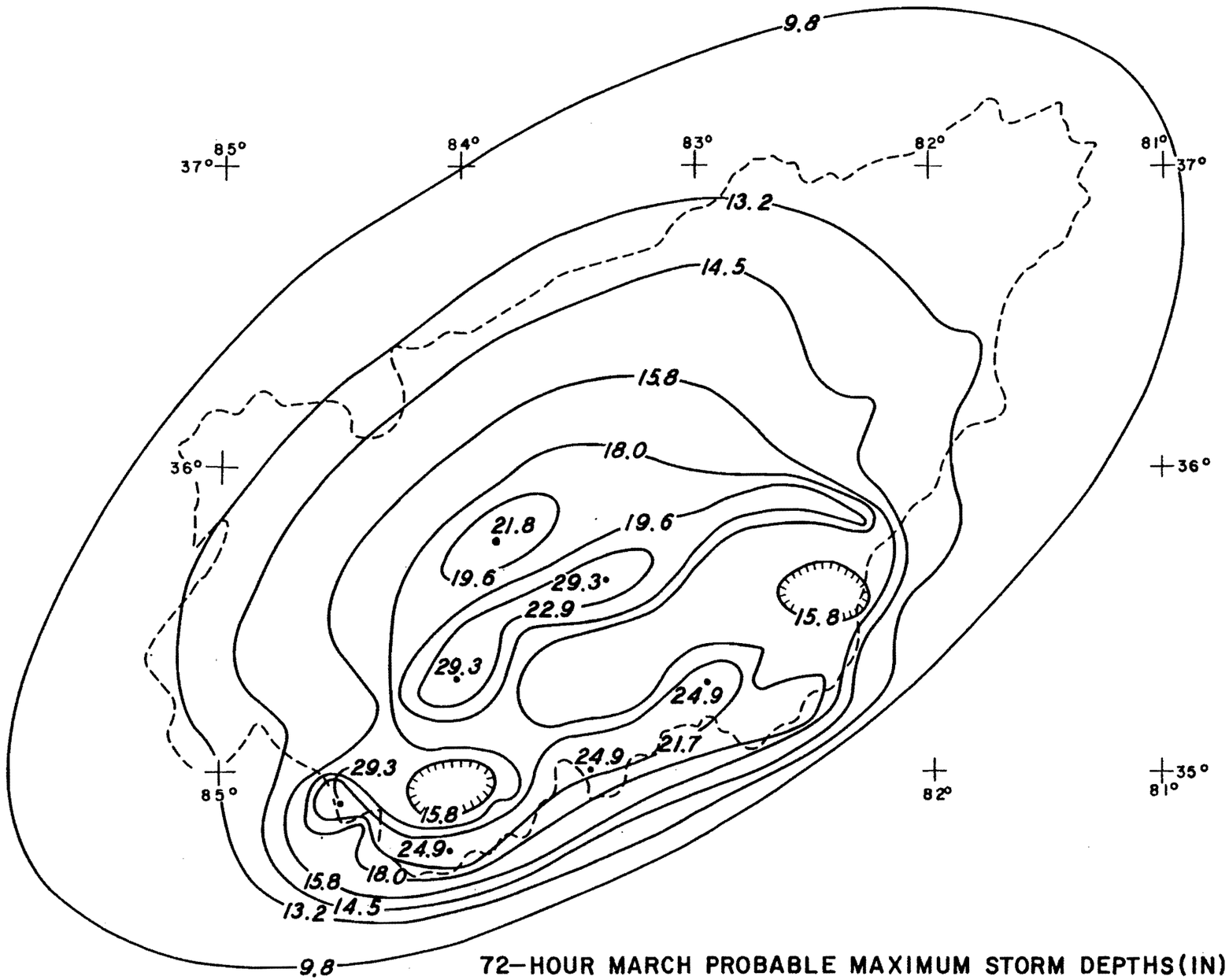


Figure 2.4-44

Figure 2.4-44 72-Hour March Probable Maximum Storm Depths (IN) Tennessee River Watershed Above Chickamauga Dam

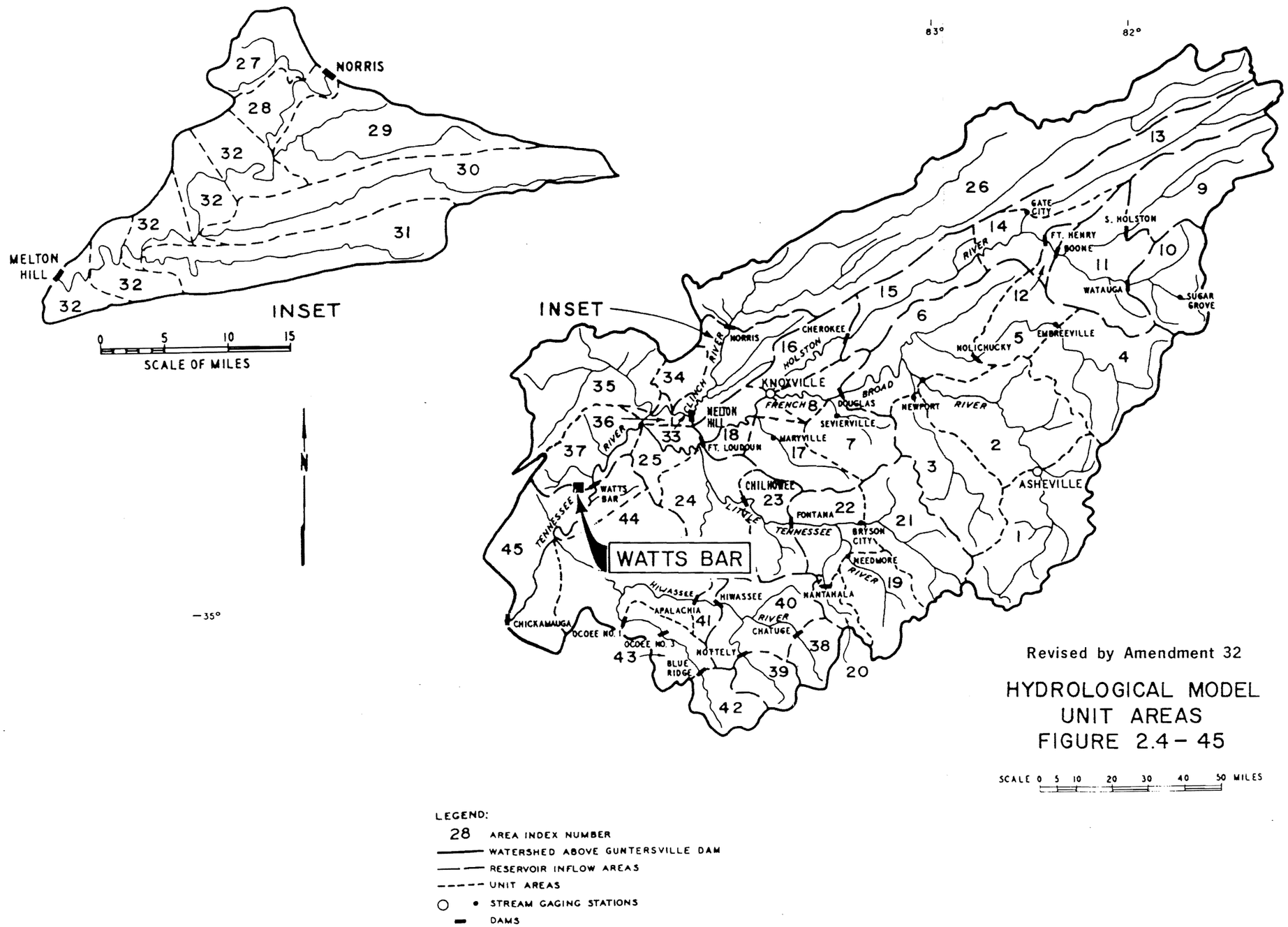
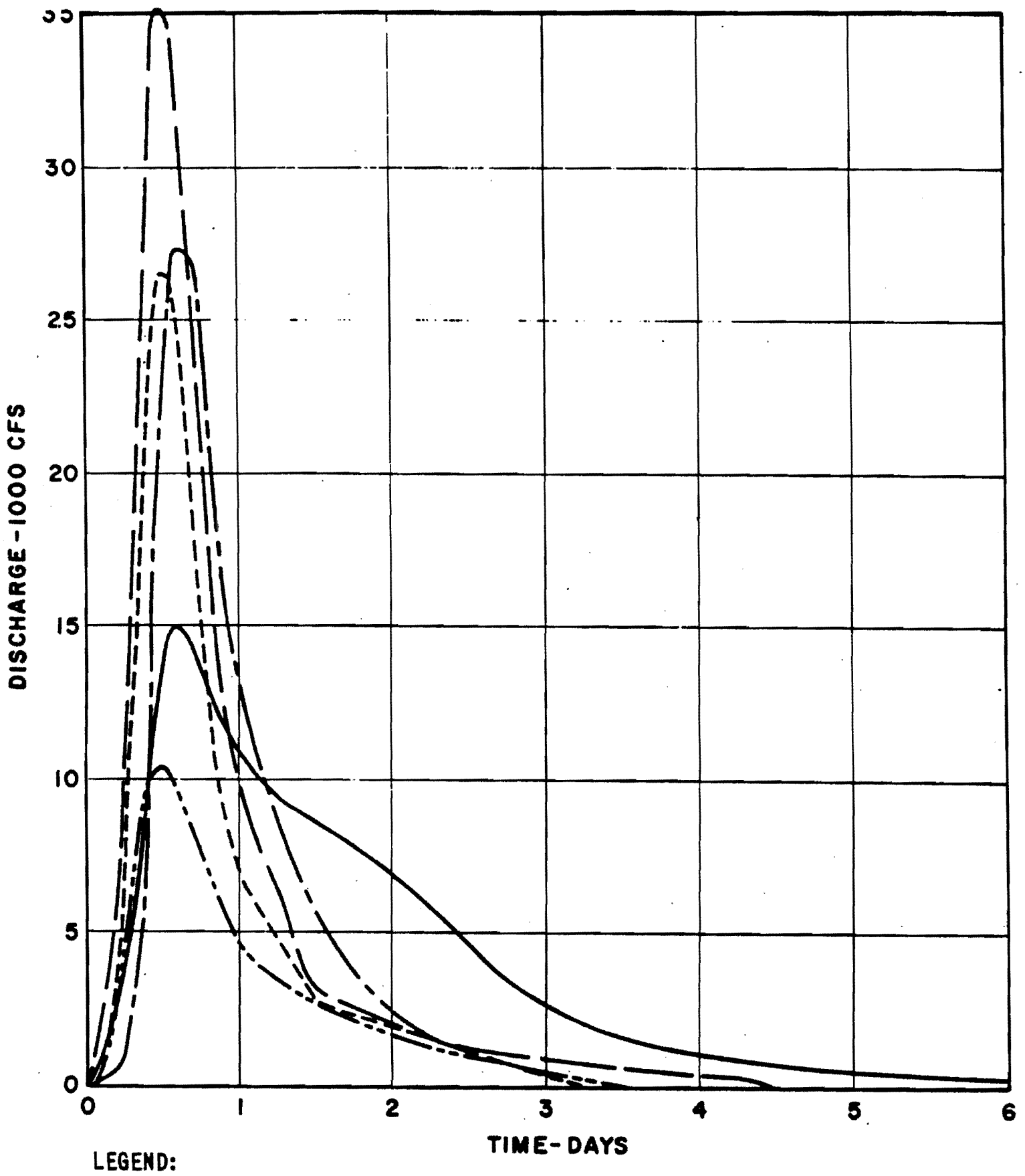


Figure 2.4-45 Hydrological Model Unit Areas



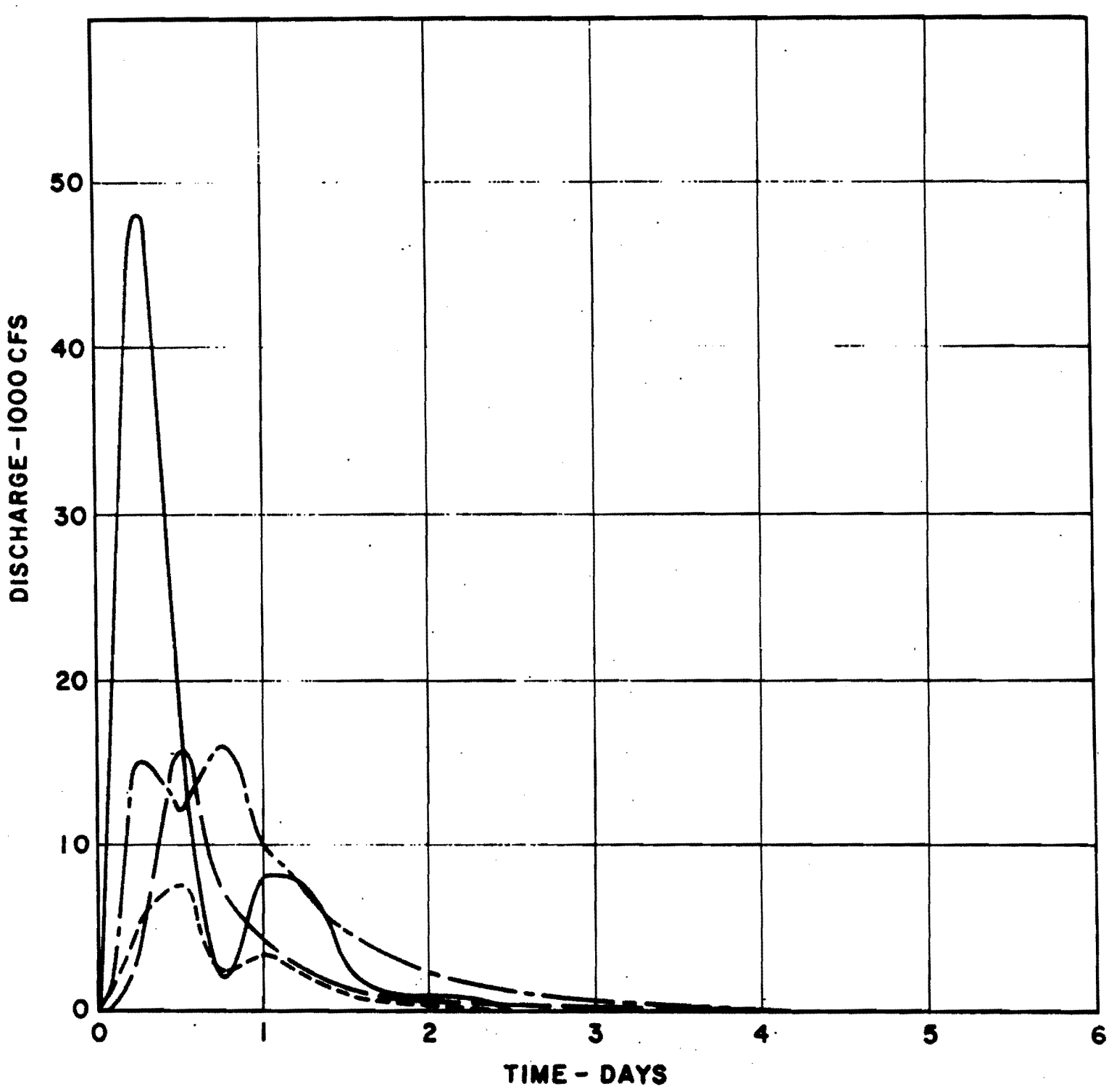
LEGEND:

AREA 1, FRENCH BROAD RIVER AT ASHEVILLE, 945 SQ. MI.
AREA 2, FRENCH BROAD RIVER, NEWPORT TO ASHEVILLE, 913 SQ. MI.
AREA 3, PIGEON RIVER AT NEWPORT, 666 SQ. MI.
AREA 4, NOLICHUCKY RIVER AT EMBREEVILLE, 805 SQ. MI.
AREA 5, NOLICHUCKY LOCAL, 378 SQ. MI.

Revised by Amendment 32

6-HOUR UNIT HYDROGRAPHS
SHEET 1 OF 11
FIGURE 2.4-46

Figure 2.4-46 6-Hour Unit Hydrographs Sheet 1 of 11



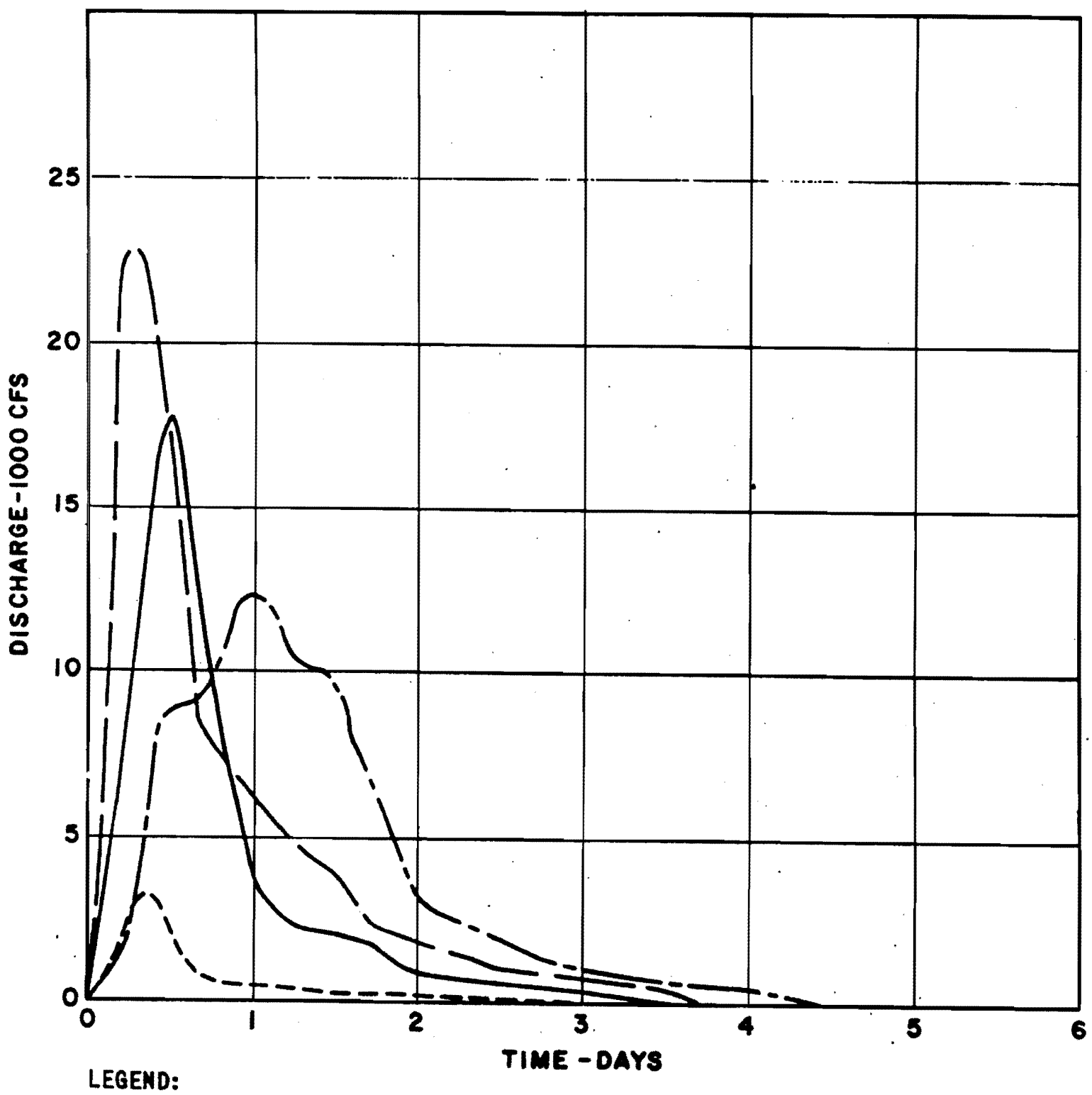
LEGEND:

AREA 6, DOUGLAS LOCAL, 832 SQ. MI.
AREA 7, LITTLE PIGEON RIVER, 353 SQ. MI.
AREA 8, FRENCH BROAD RIVER LOCAL, 207 SQ. MI.
AREA 9, SOUTH HOLSTON DAM, 703 SQ. MI.

Revised by Amendment 32

6-HOUR UNIT HYDROGRAPHS
SHEET 2 OF 11
FIGURE 2.4-46

Figure 2.4-46 6-Hour Unit Hydrographs Sheet 2 of 11



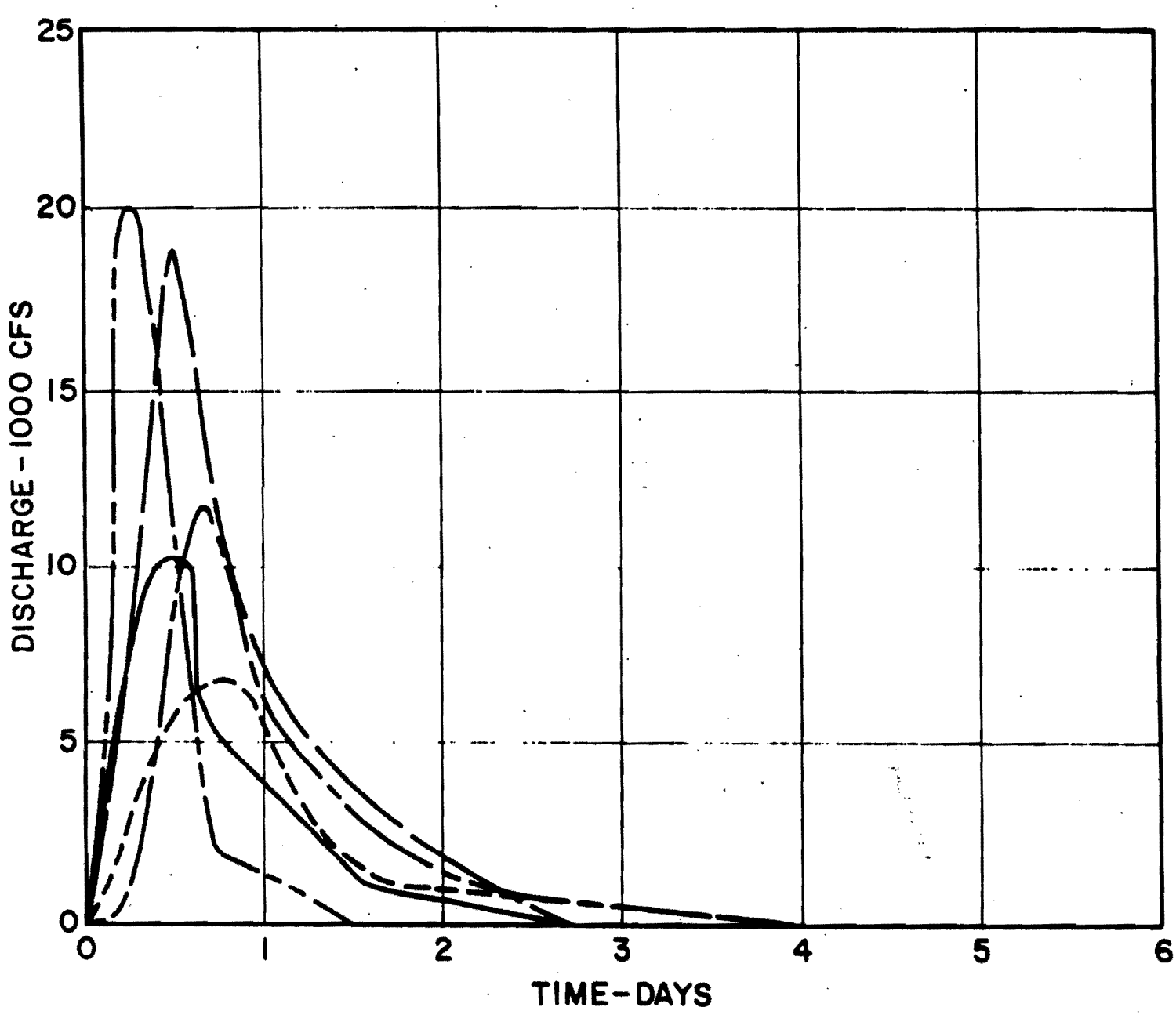
LEGEND:

AREA 10, WATAUGA DAM, 468 SQ. MI.
AREA 11, BOONE LOCAL, 669 SQ. MI.
AREA 12, FORT PATRICK HENRY LOCAL, 63 SQ. MI.
AREA 13, N. F. HOLSTON R. NR GATE CITY, 672 SQ. MI.

Revised by Amendment 32

6-HOUR UNIT HYDROGRAPHS
SHEET 3 OF 11
FIGURE 2.4 - 46

Figure 2.4-46 6-Hour Unit Hydrographs Sheet 3 of 11



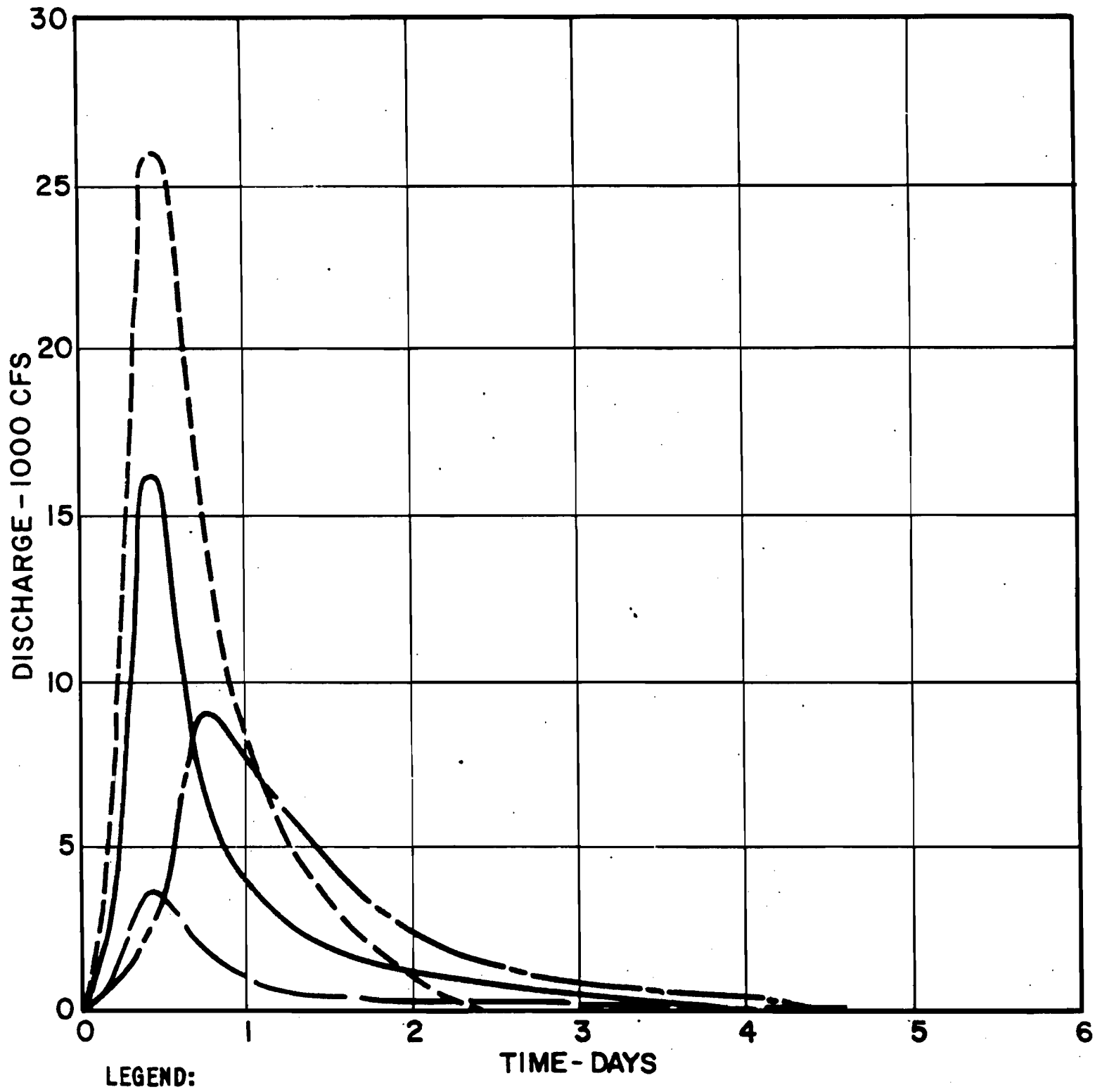
LEGEND:

AREA 14, SURGOINSVILLE LOCAL, 299 SQ. MI.
AREA 15, CHEROKEE LOCAL BELOW SURGOINSVILLE, 554 SQ. MI.
AREA 16, HOLSTON RIVER LOCAL, 289 SQ. MI.
AREA 17, LITTLE RIVER AT MOUTH, 379 SQ. MI.
AREA 18, FORT LOUDOUN LOCAL, 323 SQ. MI.

Revised by Amendment 32

6-HOUR UNIT HYDROGRAPHS
SHEET 4 OF 11
FIGURE 2.4-46

Figure 2.4-46 6-Hour Unit Hydrographs Sheet 4 of 11



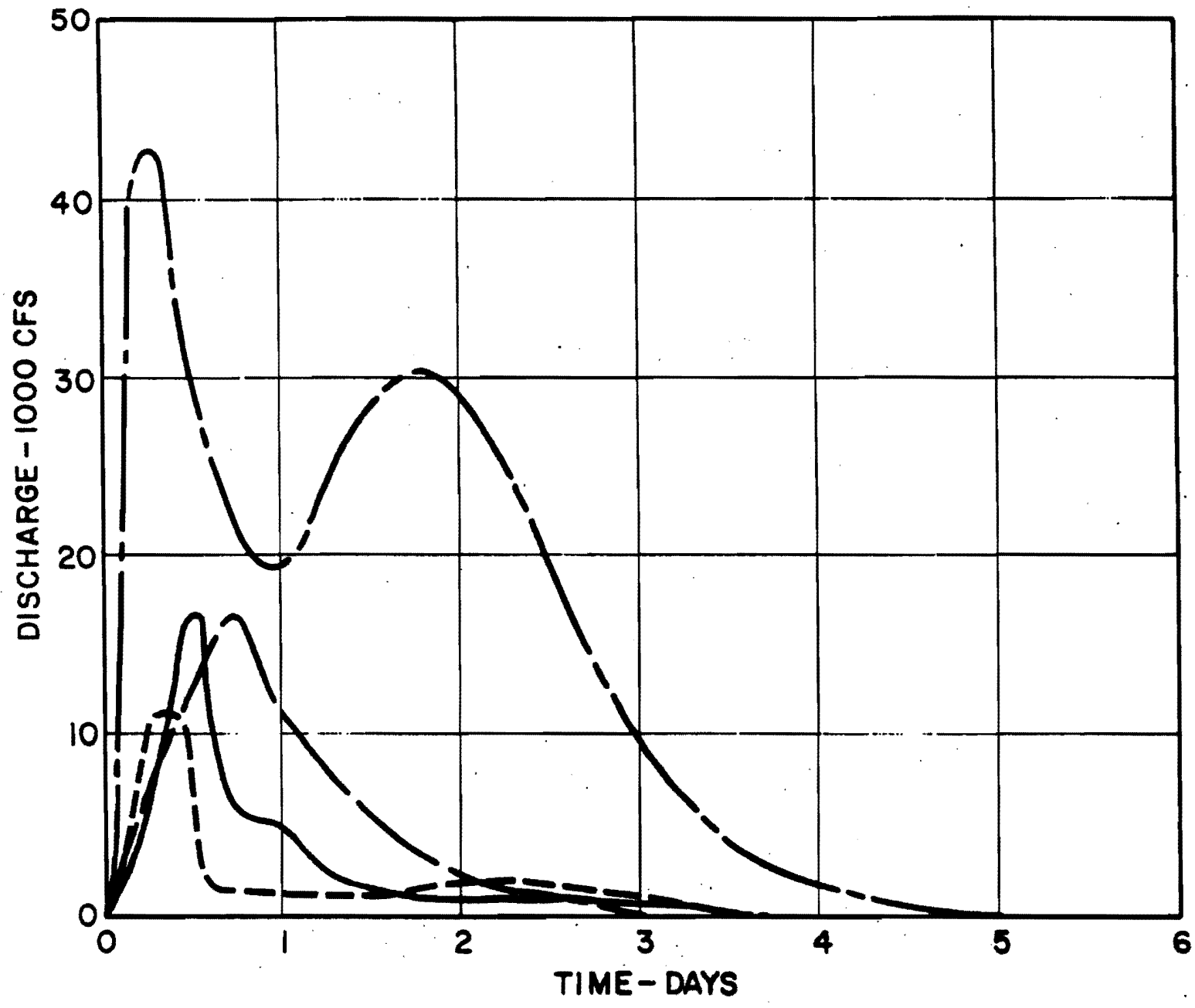
LEGEND:

AREA 19, LITTLE TENNESSEE R. AT NEEDMORE, 436 SQ. MI.
AREA 20, NANTAHALA, 91 SQ. MI.
AREA 21, TUCKASEGEE R. AT BRYSON CITY, 655 SQ. MI.
AREA 22, FONTANA LOCAL, 389 SQ. MI.

Revised by Amendment 32

6-HOUR UNIT HYDROGRAPHS
SHEET 5 OF 11
FIGURE 2.4-46

Figure 2.4-46 6-Hour Unit Hydrographs Sheet 5 of 11



LEGEND:

AREA 23, LITTLE TENNESSEE R. LOCAL, FONTANA TO CHILHOWEE, 406 SQ. MI.

AREA 24, LITTLE TENNESSEE R. LOCAL, CHILHOWEE TO TO TELLICO DAM, 650 SQ. MI.

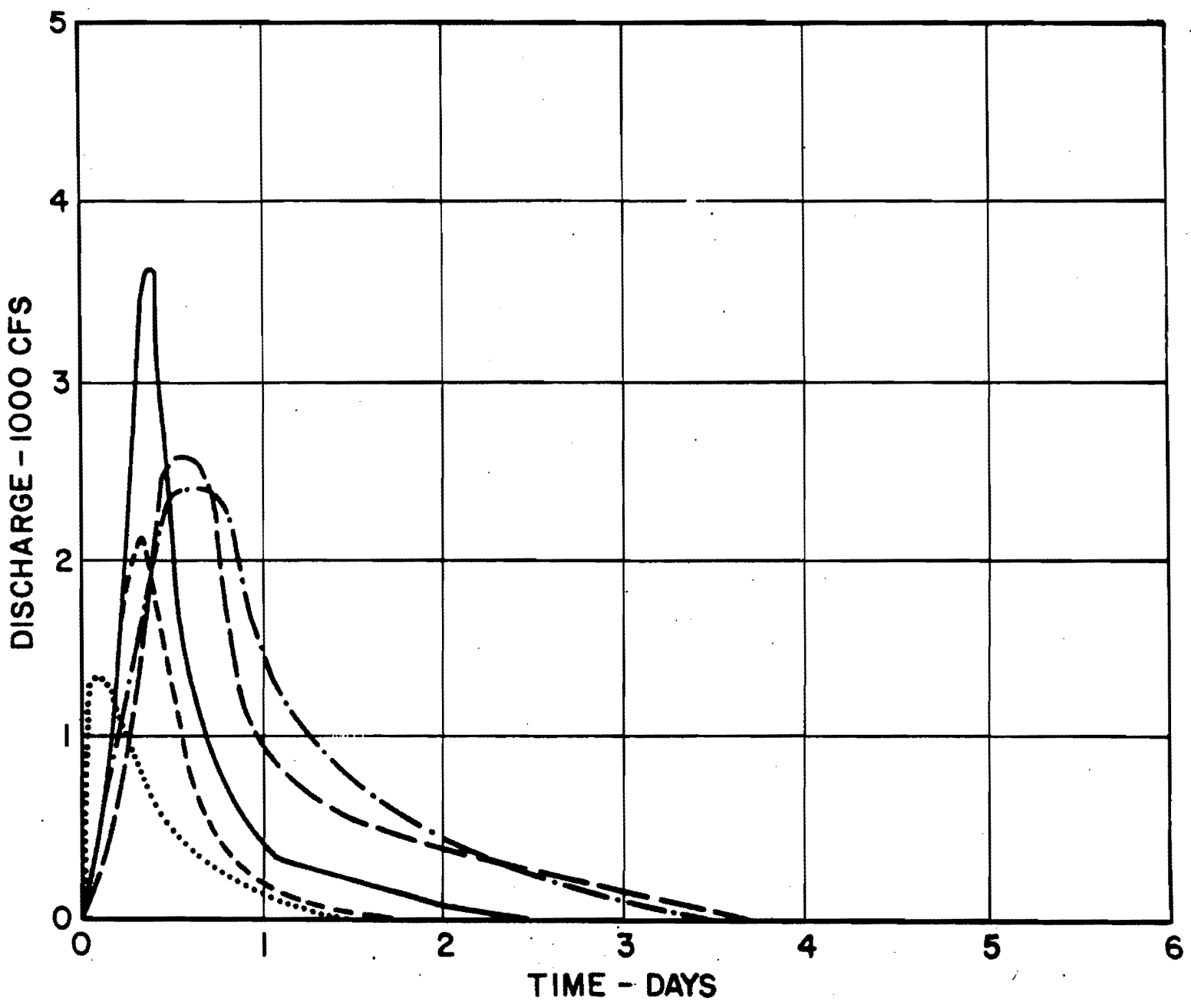
AREA 25, WATTS BAR LOCAL ABOVE CLINCH RIVER, 293 SQ. MI.

AREA 26, NORRIS DAM, 2912 SQ. MI.

Revised by Amendment 32

6-HOUR UNIT HYDROGRAPHS
SHEET 6 OF 11
FIGURE 2.4-46

Figure 2.4-46 6-Hour Unit Hydrographs Sheet 6 of 11



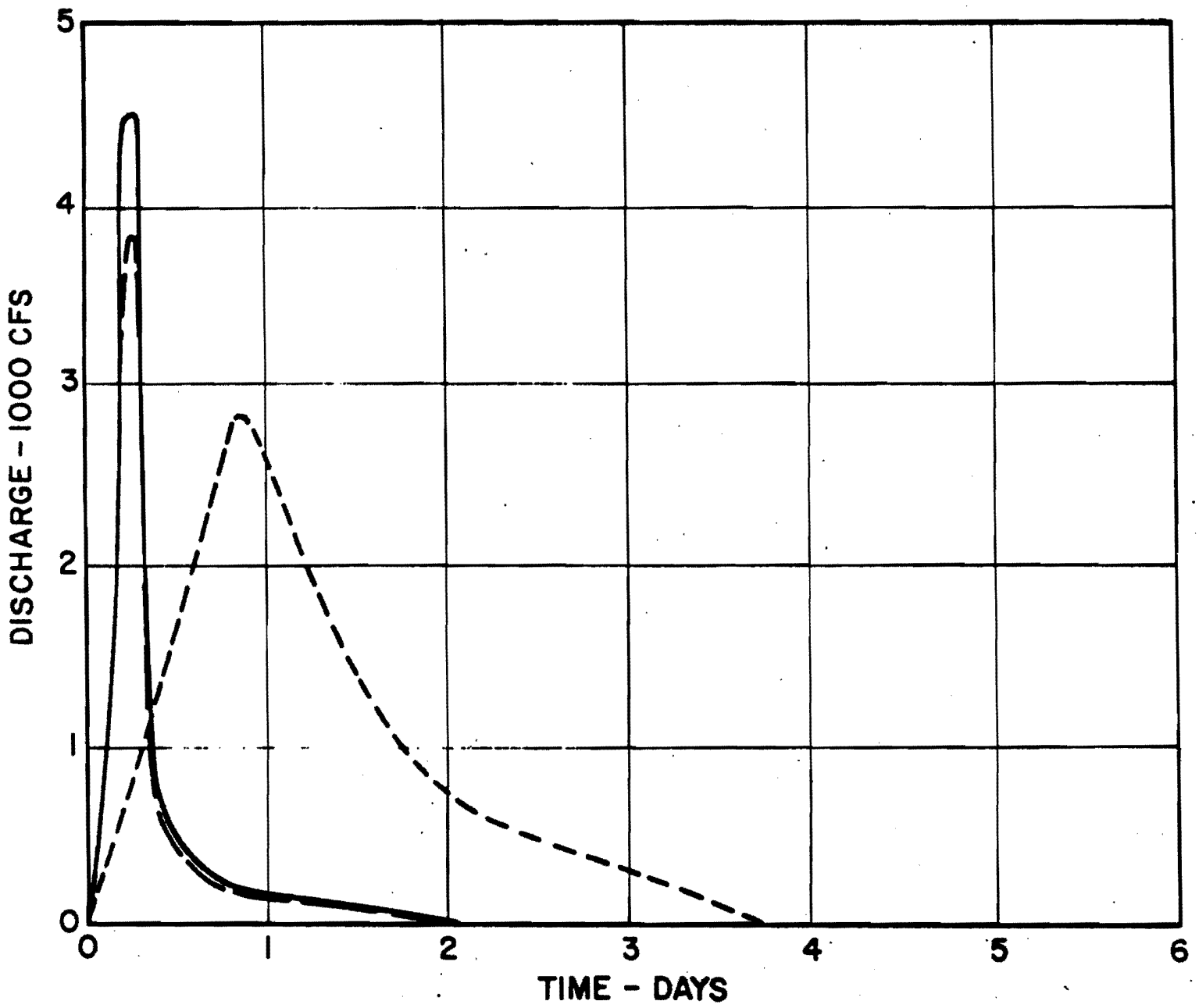
LEGEND:

- - - - - - AREA 27, COAL CREEK, 36.6 SQ. MI.
——— AREA 29, HINDS CREEK, 66.4 SQ. MI.
—·—·— AREA 30, BULLRUN CREEK, 104 SQ. MI.
— — — — AREA 31, BEAVER CREEK, 90.5 SQ. MI.
····· AREAS 28 AND 32, CLINCH RIVER LOCAL AREAS, 22. 2 SQ. MI.

Revised by Amendment 32

2-HOUR UNIT HYDROGRAPHS
SHEET 7 OF 11
FIGURE 2.4-46

Figure 2.4-46 2-Hour Unit Hydrographs Sheet 7 of 11



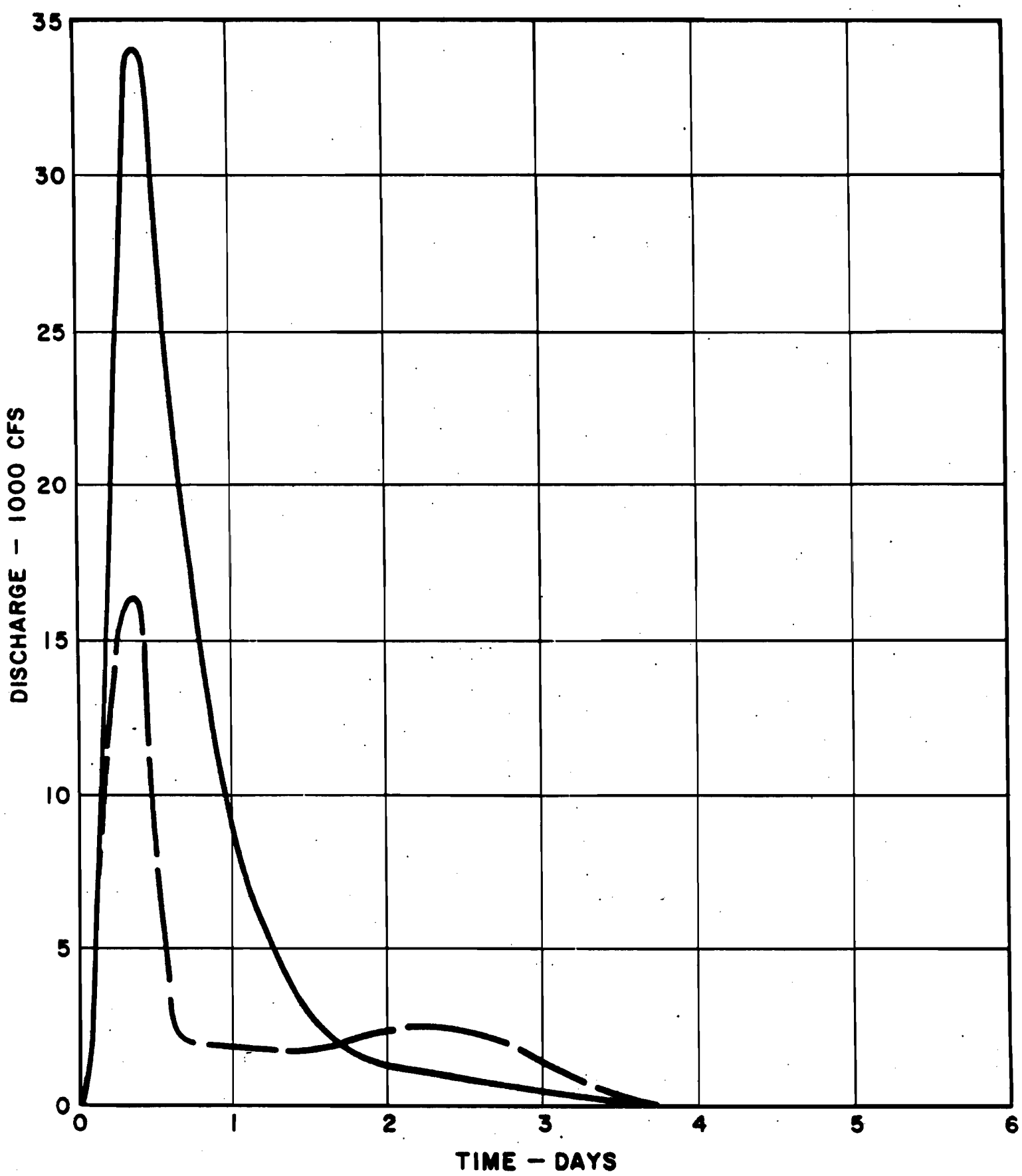
LEGEND:

AREA 33, LOCAL AREA ABOVE MI. 16, 37 SQ. MI.
AREA 34, POPLAR CREEK, 136 SQ. MI.
AREA 36, LOCAL AREA AT MOUTH, 32 SQ. MI.

Revised by Amendment 32

2-HOUR UNIT HYDROGRAPHS
SHEET 8 OF 11
FIGURE 2.4-46

Figure 2.4-46 2-Hour Unit Hydrographs Sheet 8 of 11



LEGEND:

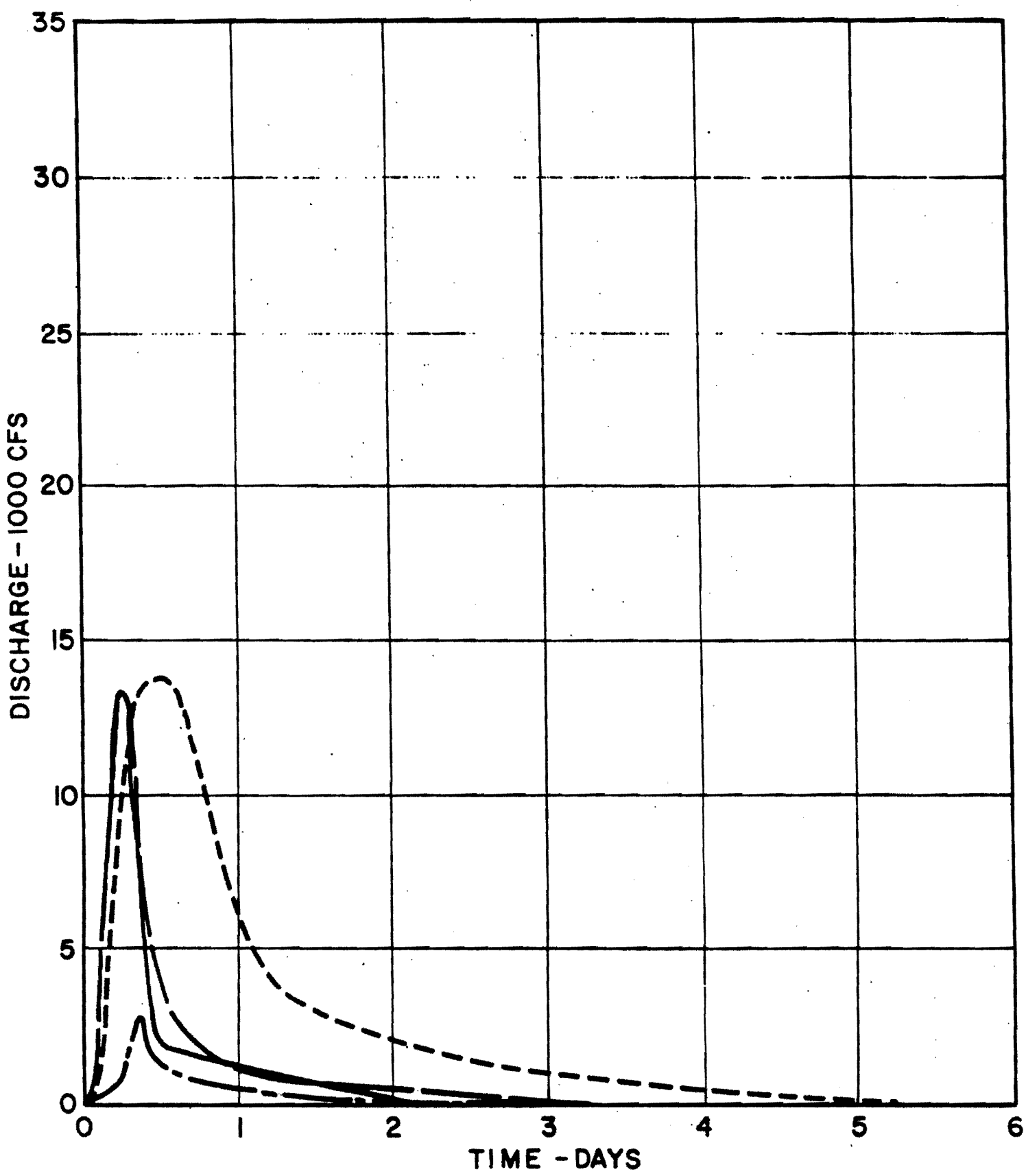
——— AREA 35, EMORY RIVER AT MOUTH, 865 SQ. MI.

— — AREA 37, WATTS BAR LOCAL BELOW CLINCH RIVER, 427 SQ. MI.

Revised by Amendment 32

6-HOUR UNIT HYDROGRAPHS
SHEET 9 OF 11
FIGURE 2.4 - 46

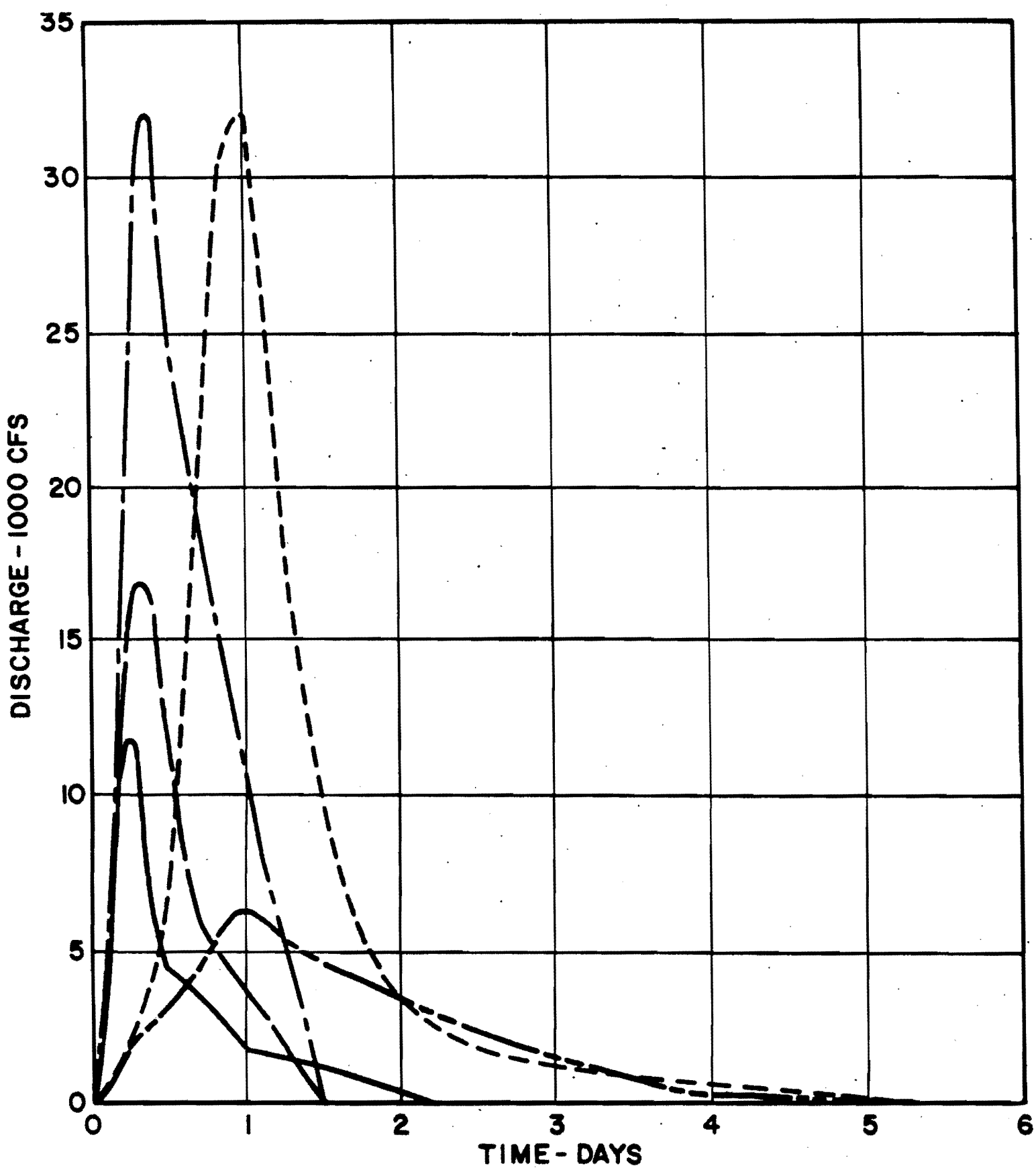
Figure 2.4-46 6-Hour Unit Hydrographs Sheet 9 of 11



Revised by Amendment 32

6-HOUR UNIT HYDROGRAPHS
SHEET 10 OF 11
FIGURE 2.4-46

Figure 2.4-46 6-Hour Unit Hydrographs Sheet 10 of 11



LEGEND:

AREA 42, BLUE RIDGE DAM, 232 SQ. MI.
AREA 43, OCOEE NO. 1 TO BLUE RIDGE DAM. 363 SQ. MI.
AREA 44, LOWER HIWASSEE LOCAL, 1087 SQ. MI.
AREA 45, CHICKAMAUGA LOCAL, 780 SQ. MI.

Revised by Amendment 32

6-HOUR UNIT HYDROGRAPHS
SHEET 11 OF 11
FIGURE 2.4 - 46

Figure 2.4-46 6-Hour Unit Hydrographs Sheet 11 of 11

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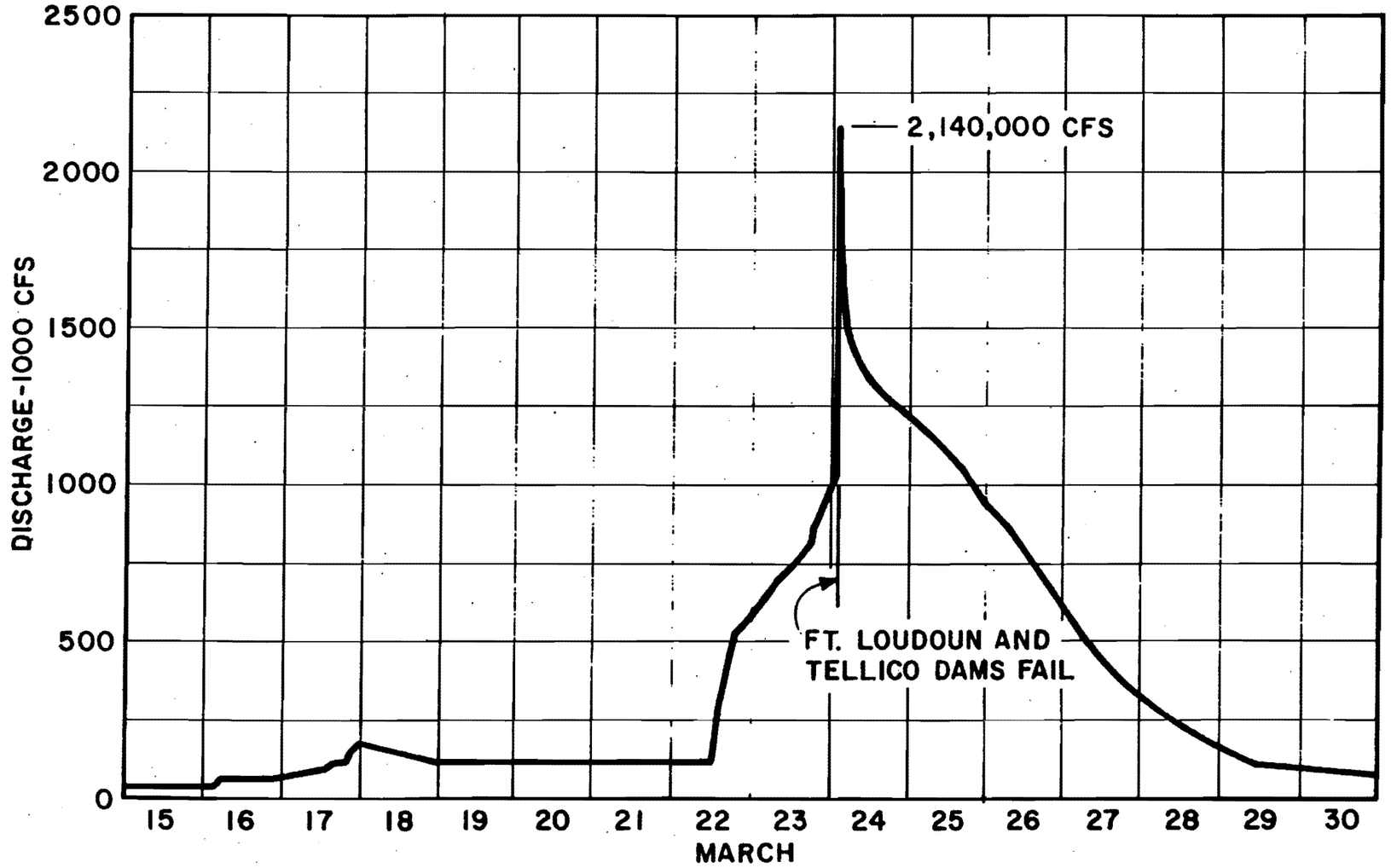
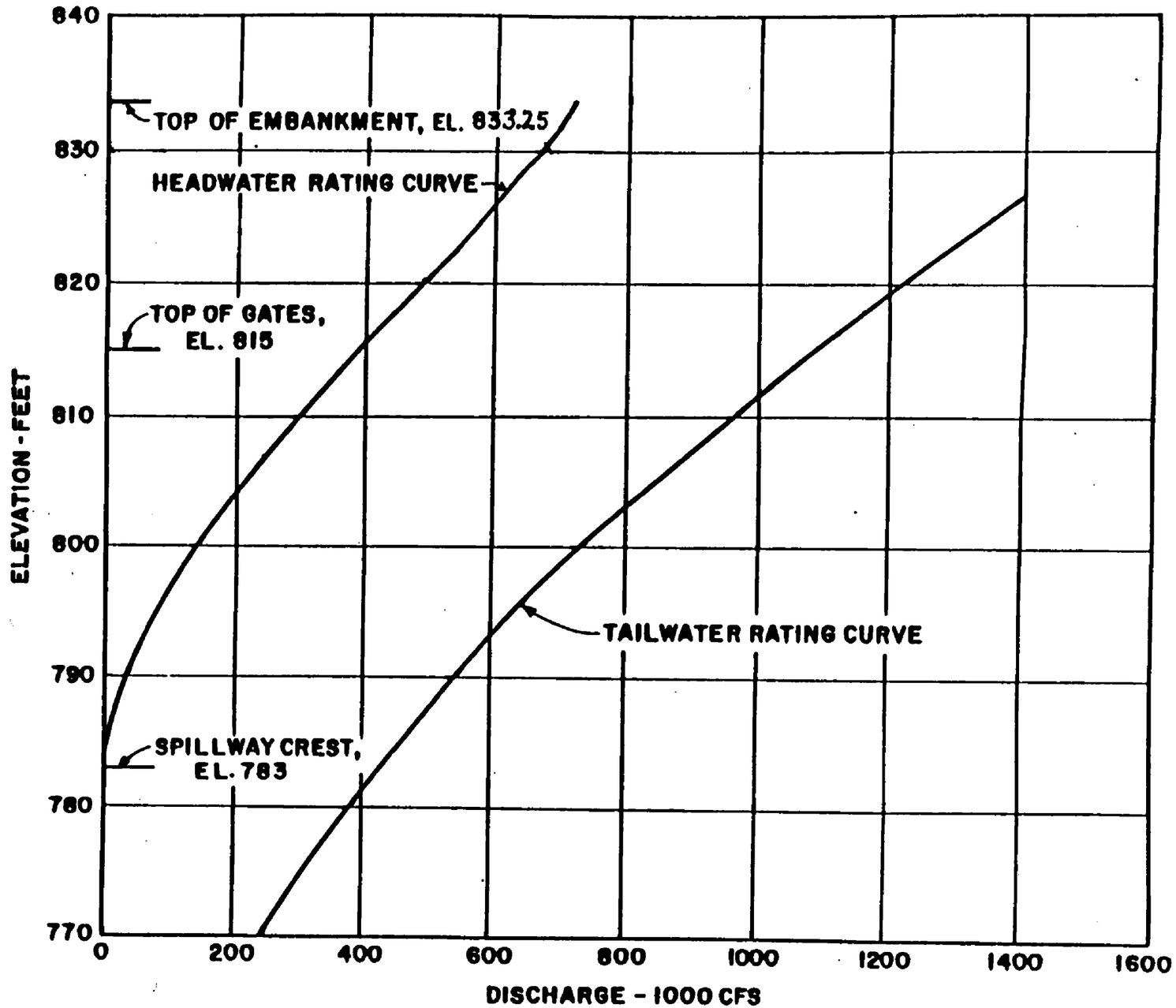


WATTS BAR PROBABLE MAXIMUM FLOOD
FT. LOUDOUN – TELLICO OUTFLOW
FIGURE 2.4-47a

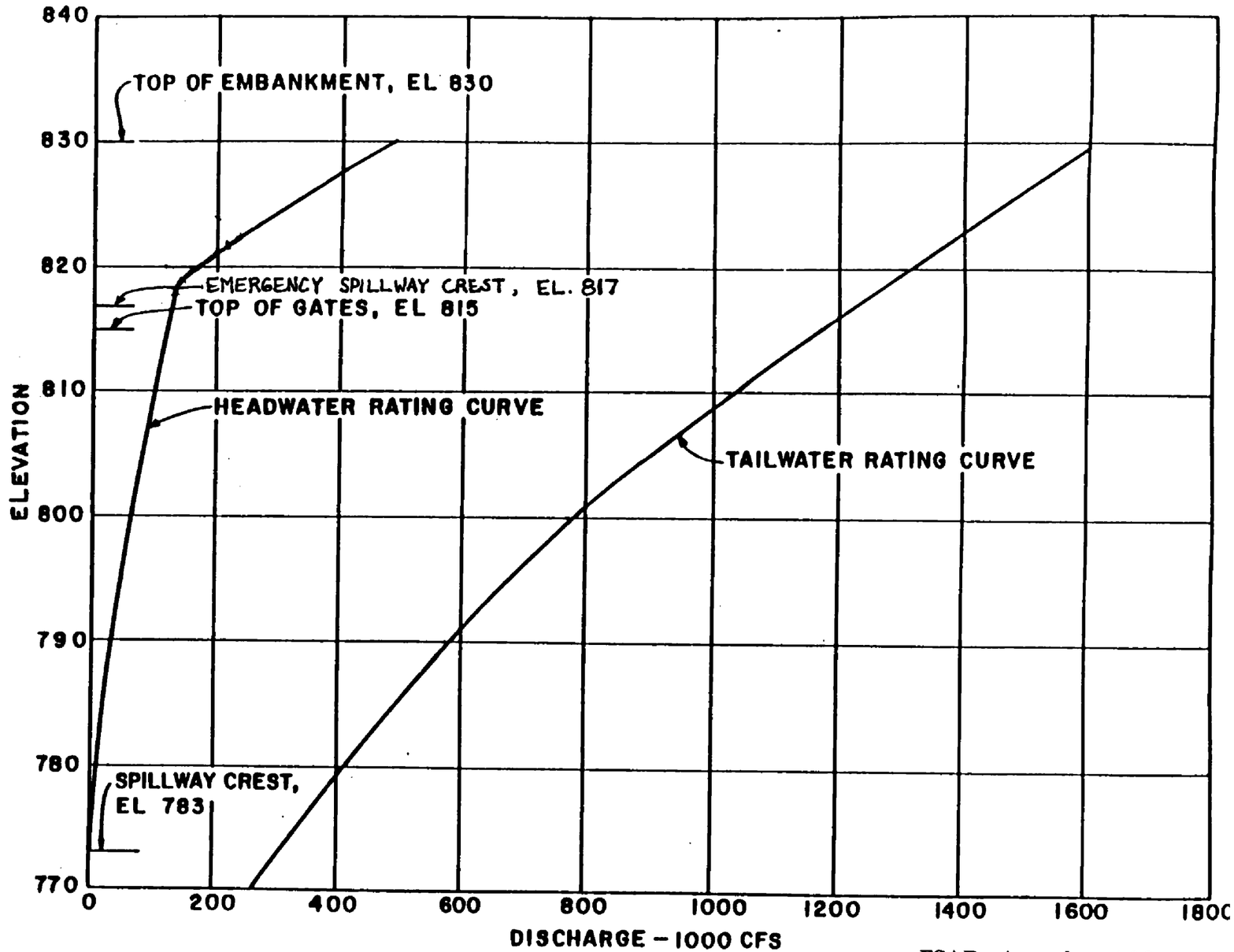
Added by Amendment 32

Figure 2.4-47a Watts Bar Probable Maximum Flood Fort Loudon - Tellico Outflow



FSAR - Amendment 92

Figure 2.4-47b Fort Loudon Dam Rating Curve

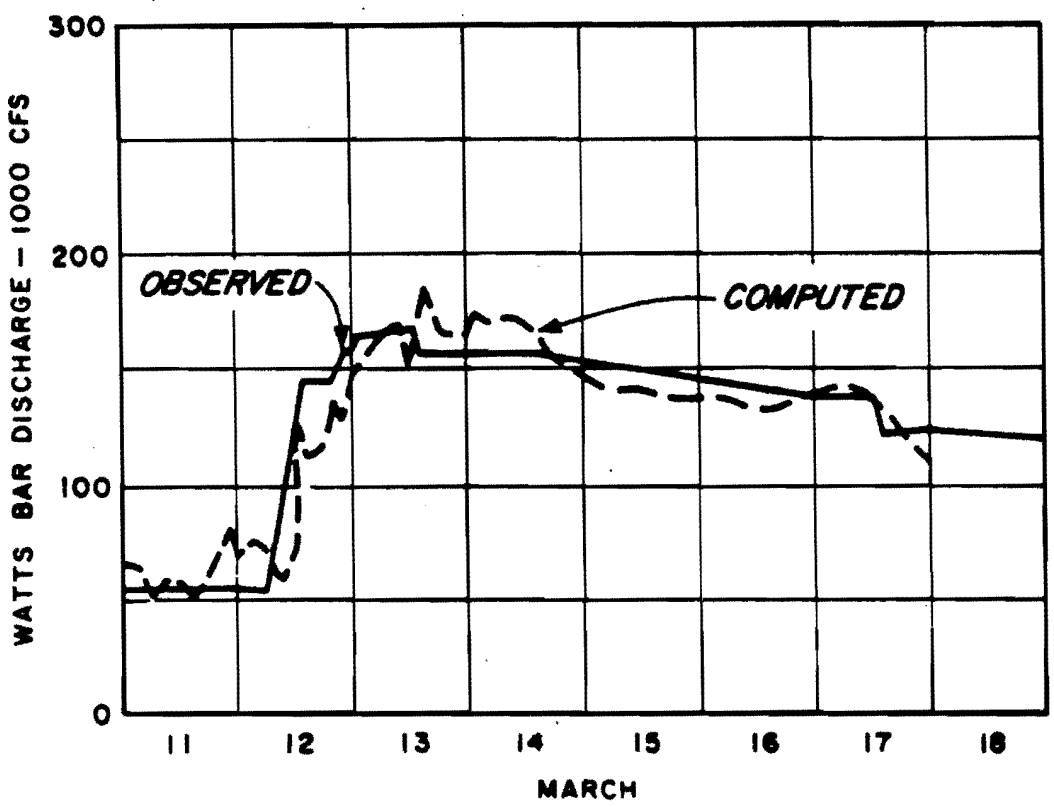


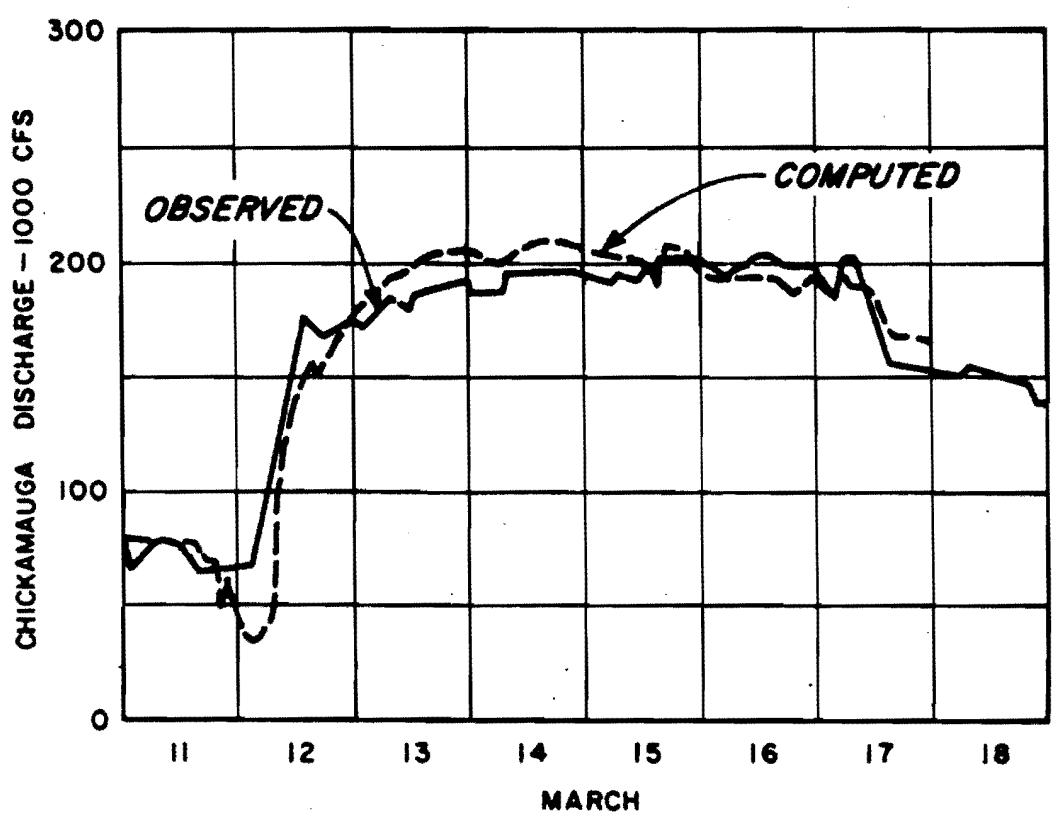
FSAR - Amendment 92

TELLICO DAM RATING CURVES

FIGURE 2.4-47c

Figure 2.4-47c Tellico Dam Rating Curve

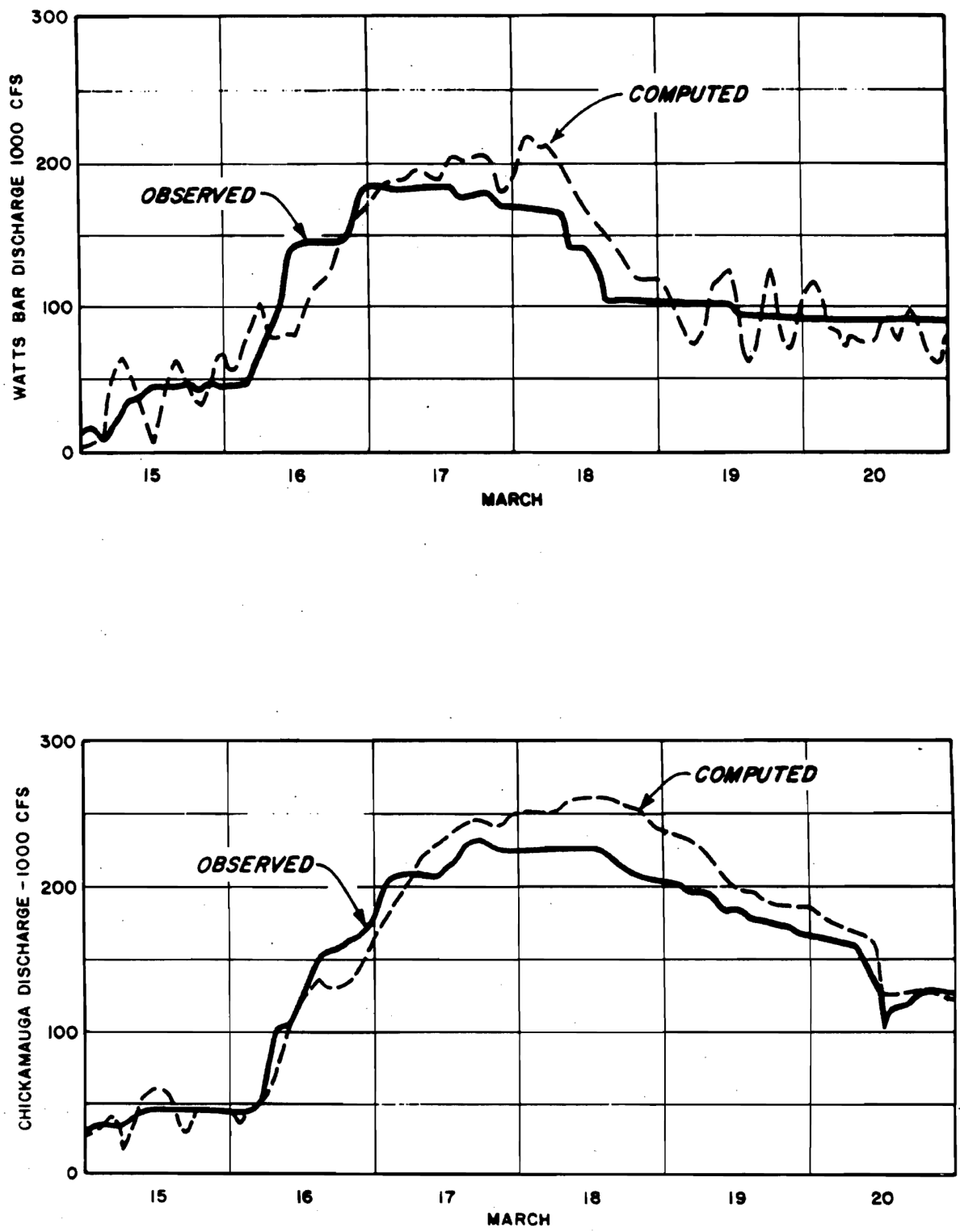




HYDROLOGIC MODEL VERIFICATION – 1963 FLOOD
FIGURE 2.4-48

Revised by Amendment 32

Figure 2.4-48 General Plan Elevation & Sections



HYDROLOGIC MODEL VERIFICATION - 1973 FLOOD

FIGURE 2.4-49

Revised by Amendment 32

Figure 2.4-49 Hydrologic Model Verification - 1973 Flood

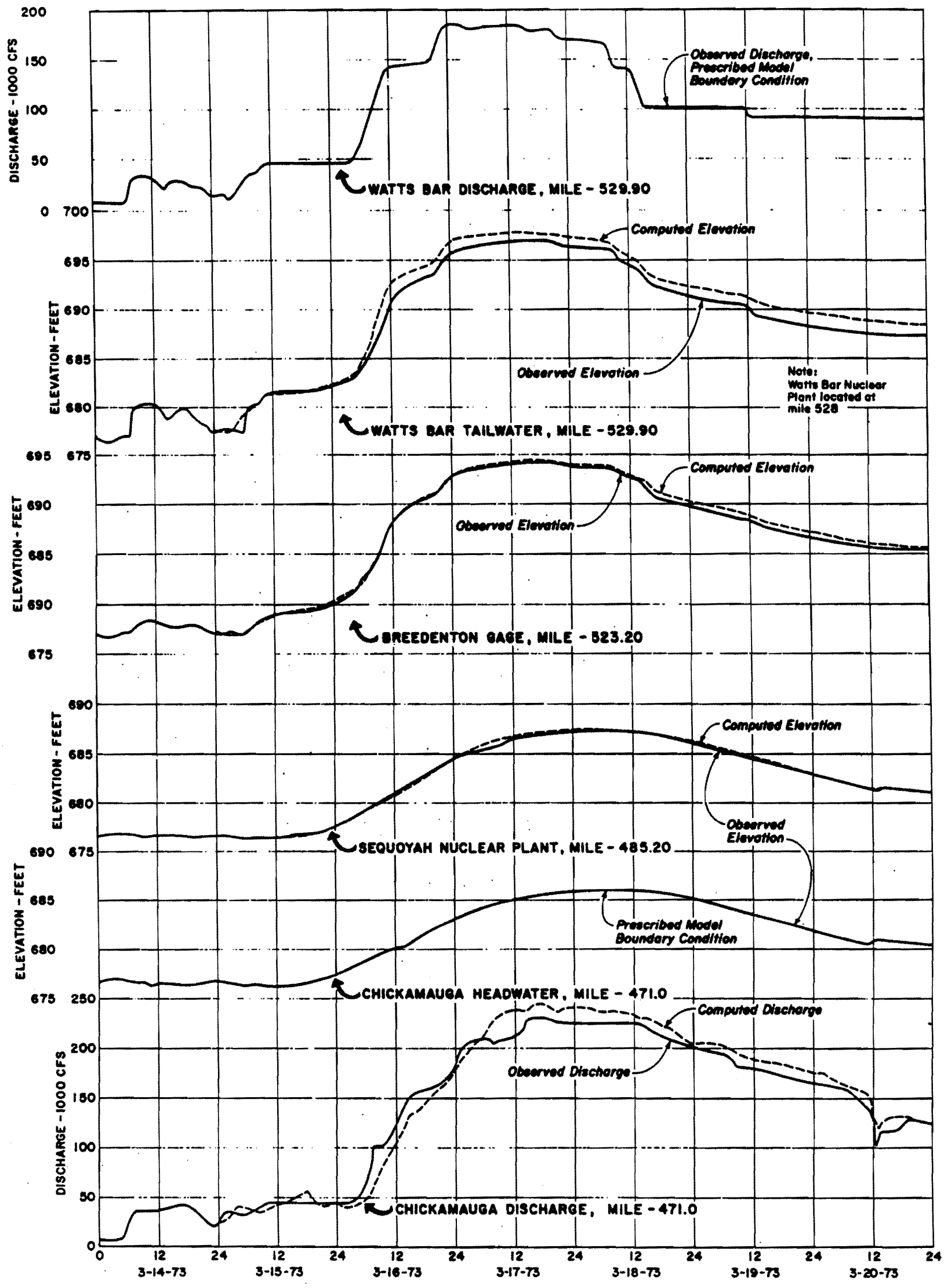
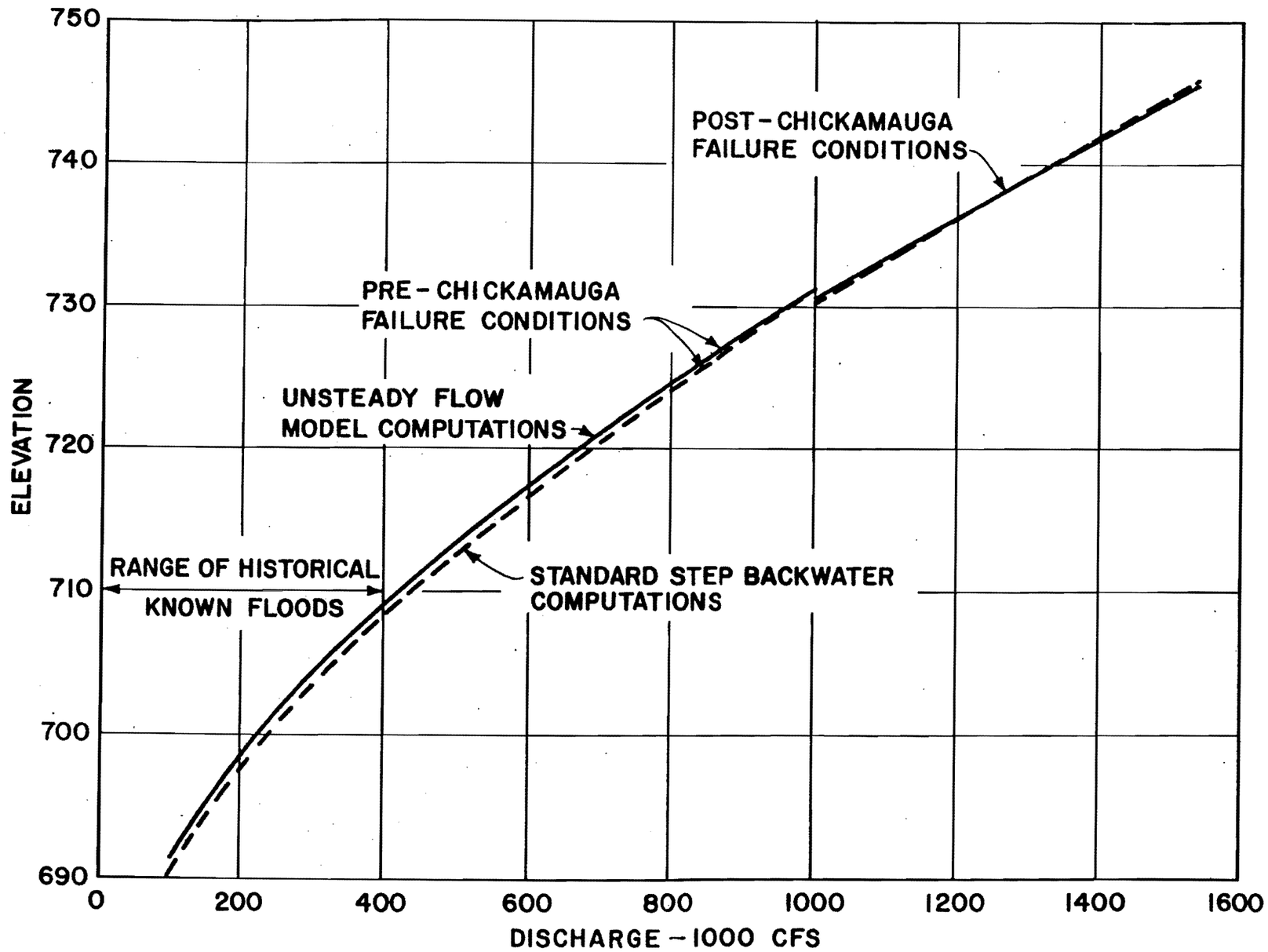


FIGURE 2.4-50 1973 FLOOD - CHICKAMAUGA RESERVOIR UNSTEADY FLOW MODEL VERIFICATION

Revised by Amendment 32

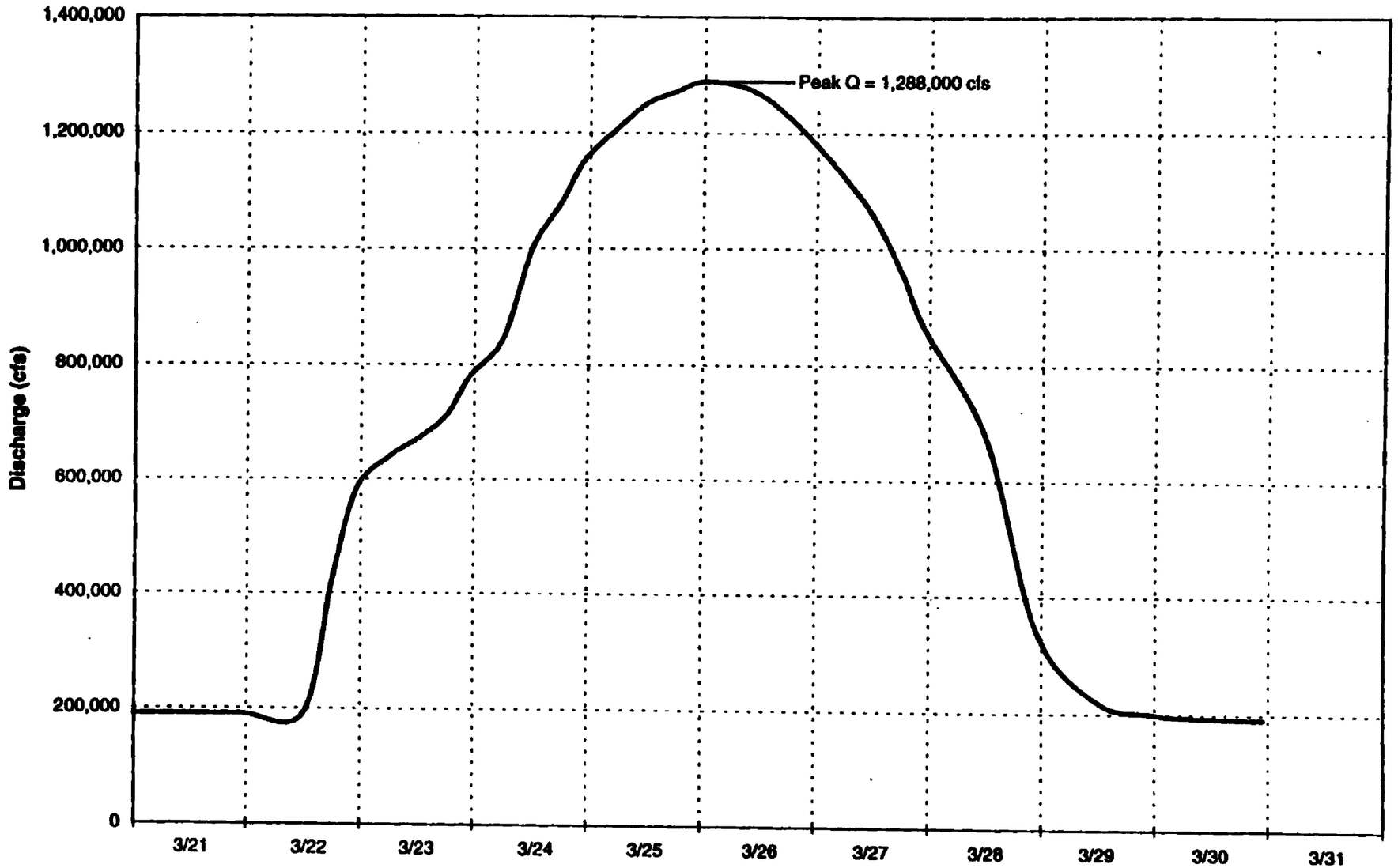
Figure 2.4-50 1973 Flood-Chickamauga Reservoir Unsteady Flow Model Verification



STEADY-STATE MODEL VERIFICATION
WATTS BAR DAM TAILWATER RATING CURVE
FIGURE 2.4-51

Revised by Amendment 32

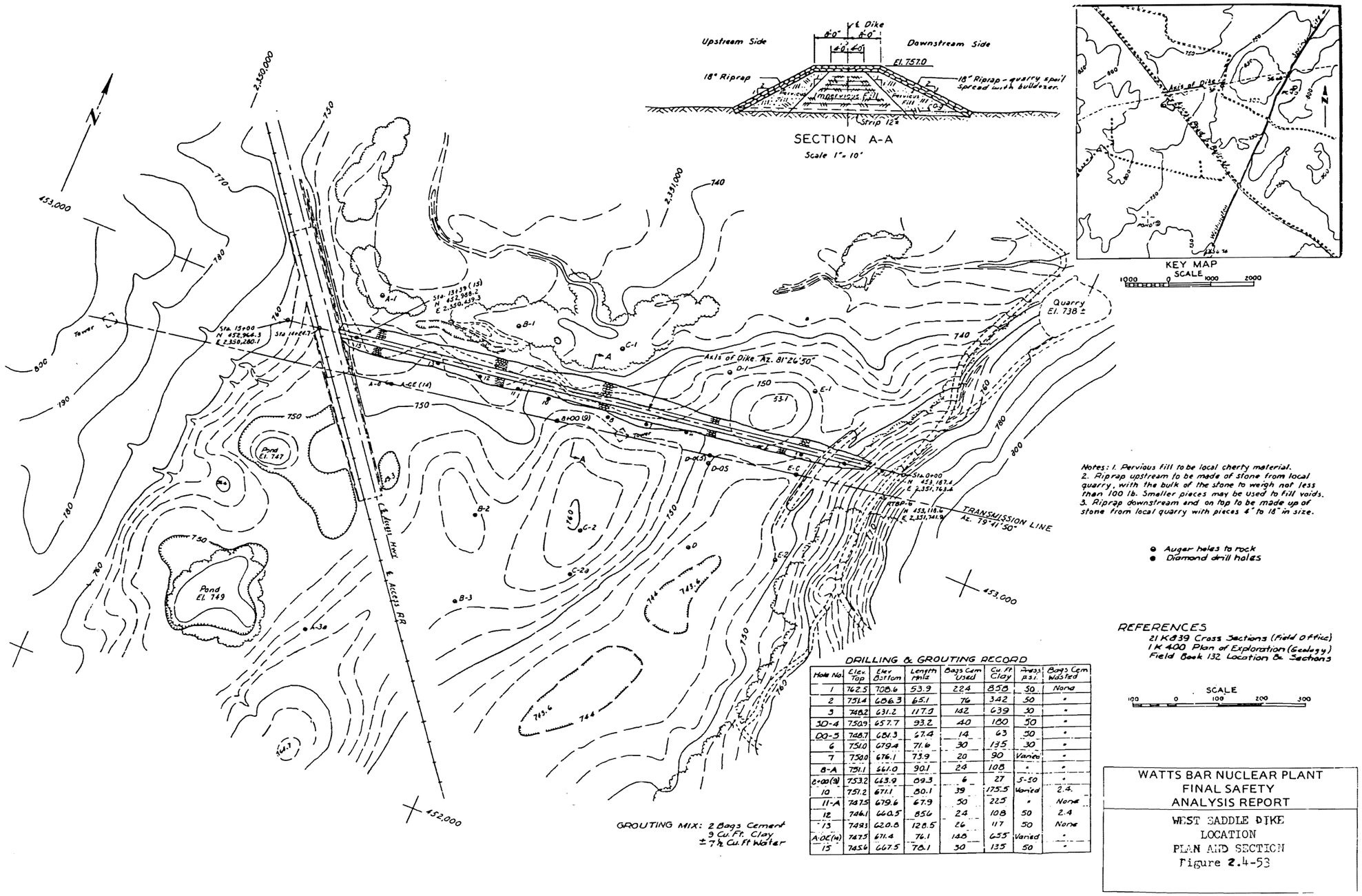
Figure 2.4-51 Steady- State Model Verification Watts Bar Dam Tailwater Rating Curve



Watts Bar Nuclear Plant Probable Maximum Flood Discharge
Figure 2.4 - 52

FSAR - Amendment 92

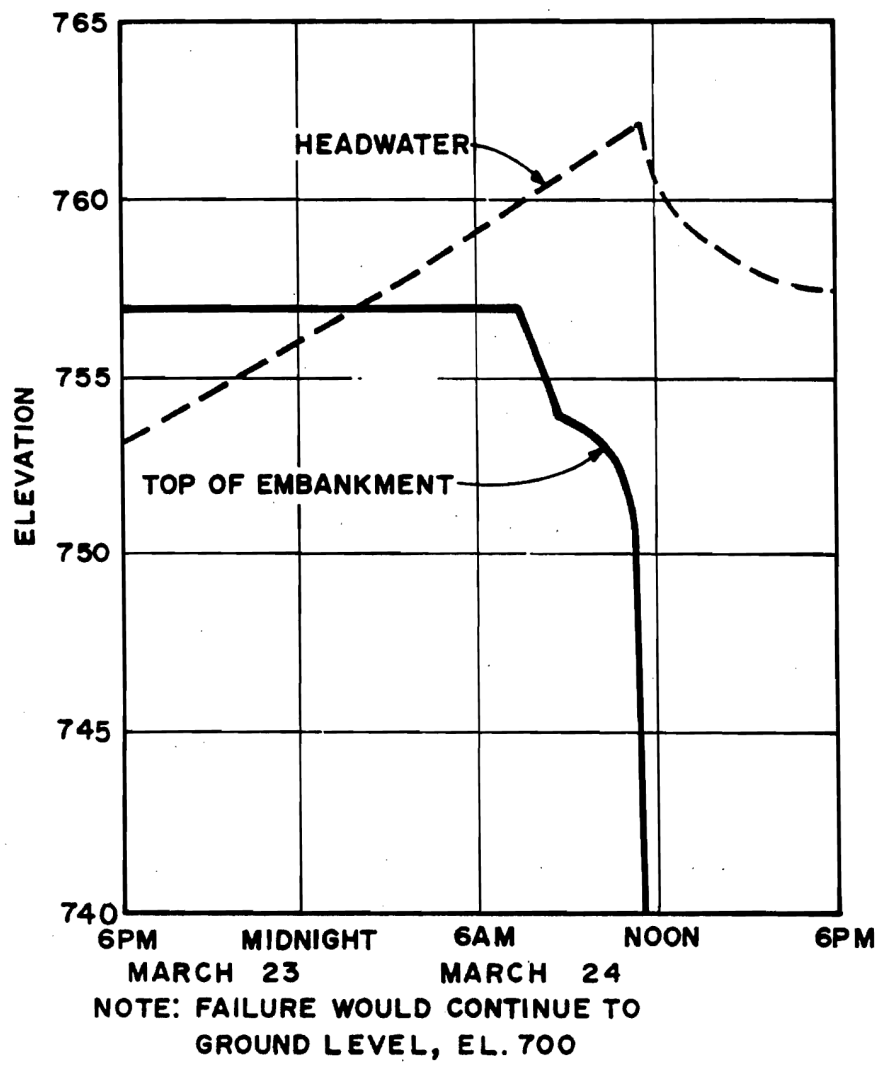
Figure 2.4-52 Watts Bar Nuclear Plant Probable Maximum Flood Discharge



DRILLING & GROUTING RECORD

Hole No.	Elev. Top	Elev. Bottom	Length rmlz	Bags Cem Used	Cu. Ft Clay	Press. p.s.i.	Bags Cem Wasted
1	762.5	708.6	53.9	224	858	50	None
2	751.4	686.3	65.1	76	342	50	"
3	748.2	631.2	117.0	142	639	50	"
30-4	750.9	657.7	93.2	40	180	50	"
DO-5	748.7	681.3	67.4	14	63	50	"
6	751.0	679.4	71.6	30	135	30	"
7	750.0	676.1	73.9	20	90	Varied	"
8-A	751.1	661.0	90.1	24	108	"	"
8+00(9)	753.2	643.9	09.3	6	27	5-50	"
10	751.2	671.1	80.1	39	175.5	Varied	2.4
11-A	747.5	679.6	67.9	50	225	"	None
12	746.1	660.5	85.6	24	108	50	2.4
13	749.3	620.8	128.5	26	117	50	None
A-0C(14)	747.5	671.4	76.1	145	655	Varied	"
15	745.6	667.5	78.1	30	135	50	"

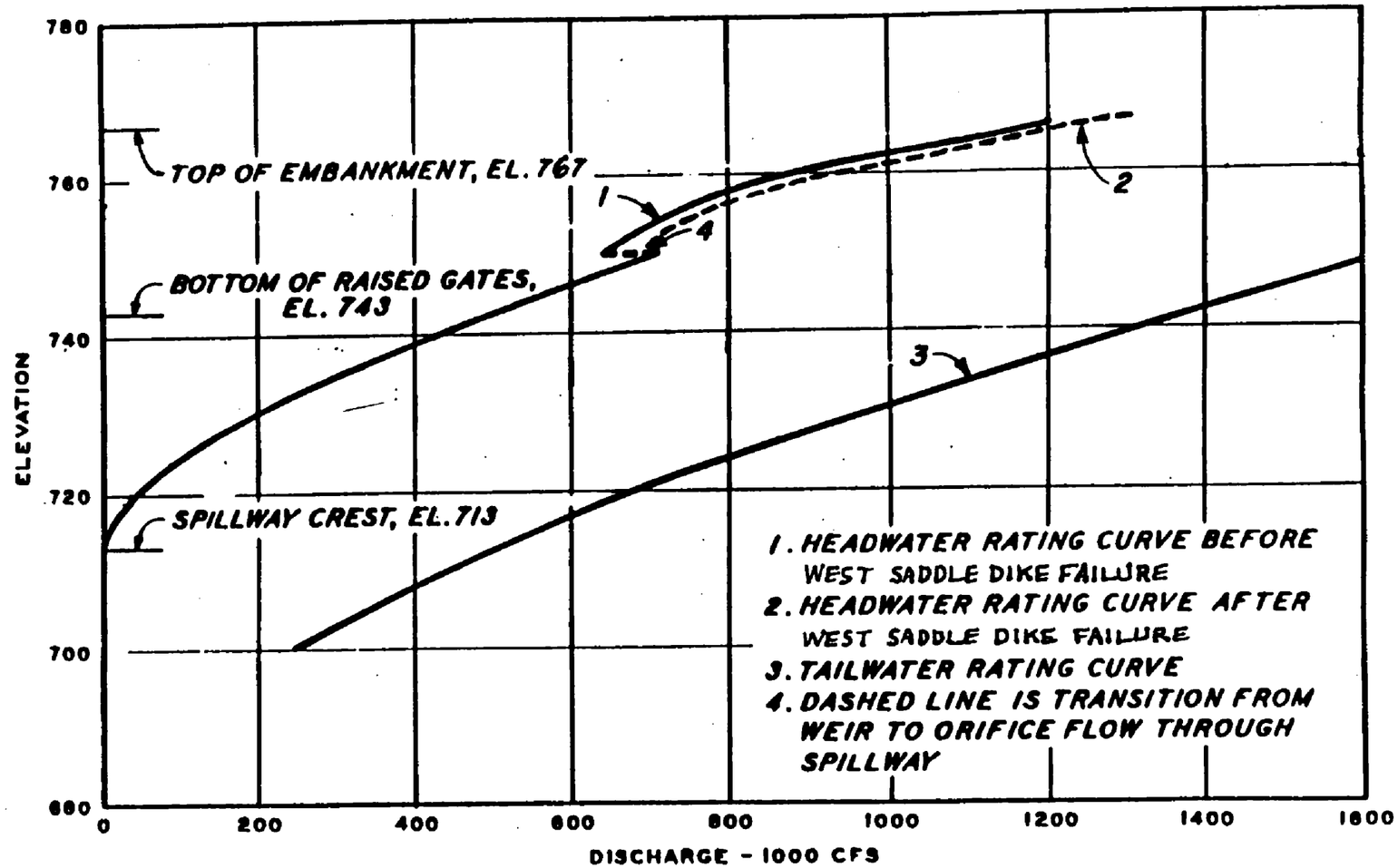
Figure 2.4-53 West Saddle Dike Location Plan and Section



WATTS BAR PROBABLE MAXIMUM FLOOD
WATTS BAR EMBANKMENT FAILURE
FIGURE 2.4-54

Revised by Amendment 32

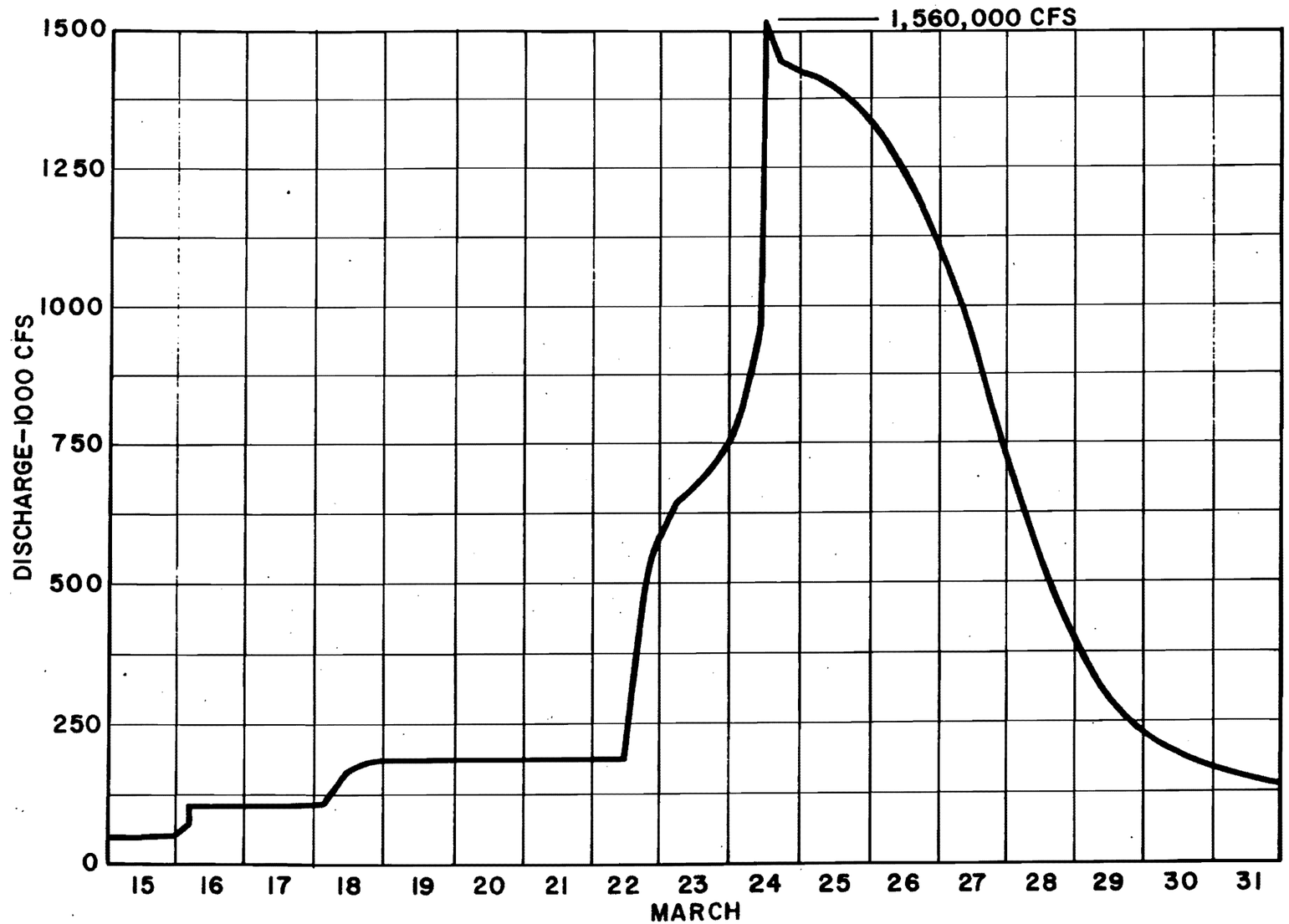
Figure 2.4-54 Watts Bar Probable Maximum Flood -Watts Bar Embankment Failure



WATTS BAR DAM RATING CURVES
FIGURE 2.4-55

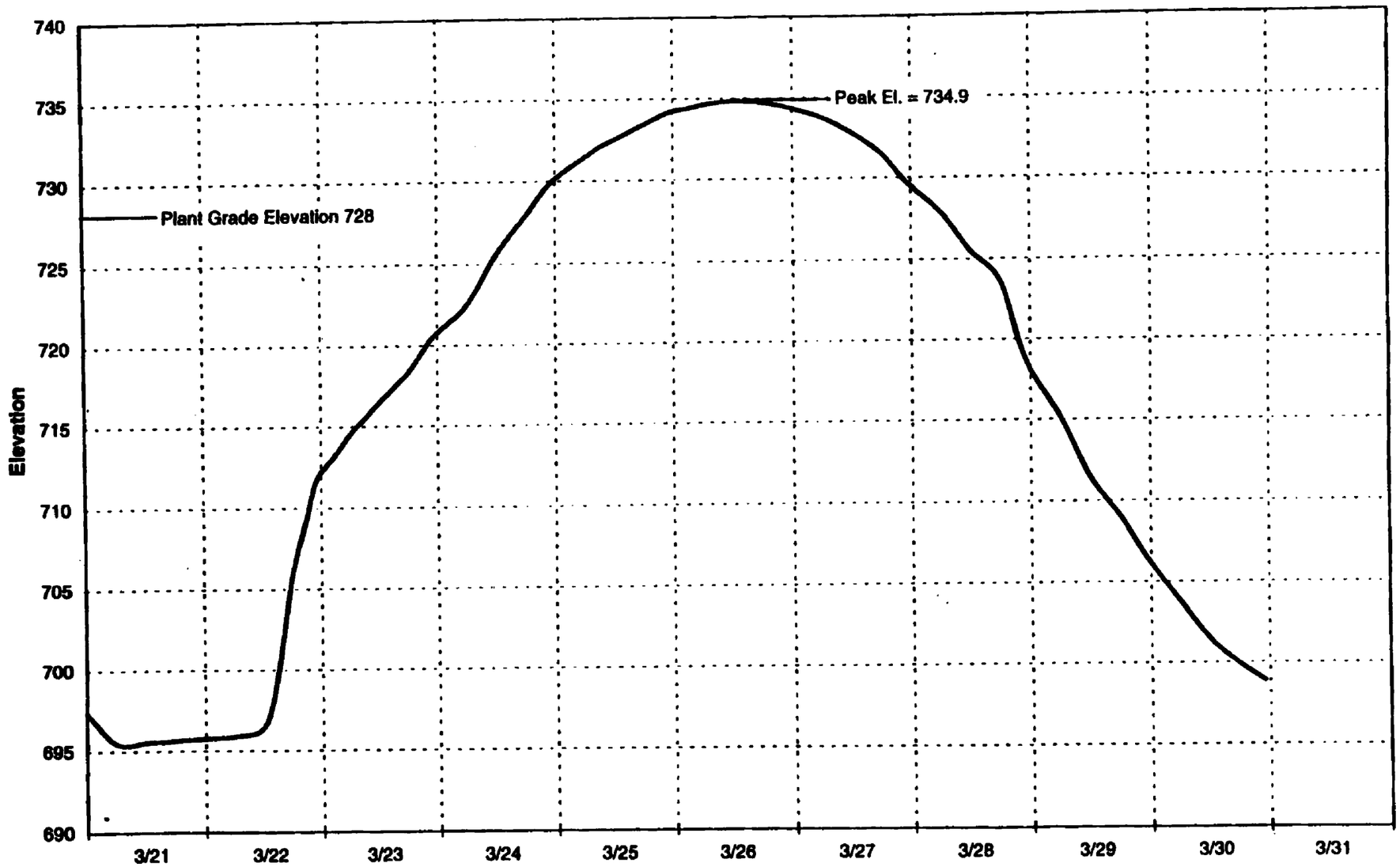
FSAR - Amendment 92

Figure 2.4-55 Watts Bar Dam Rating Curves



WATTS BAR PROBABLE MAXIMUM FLOOD
WATTS BAR DAM OUTFLOW
FIGURE 2.4-56
Revised by Amendment 32

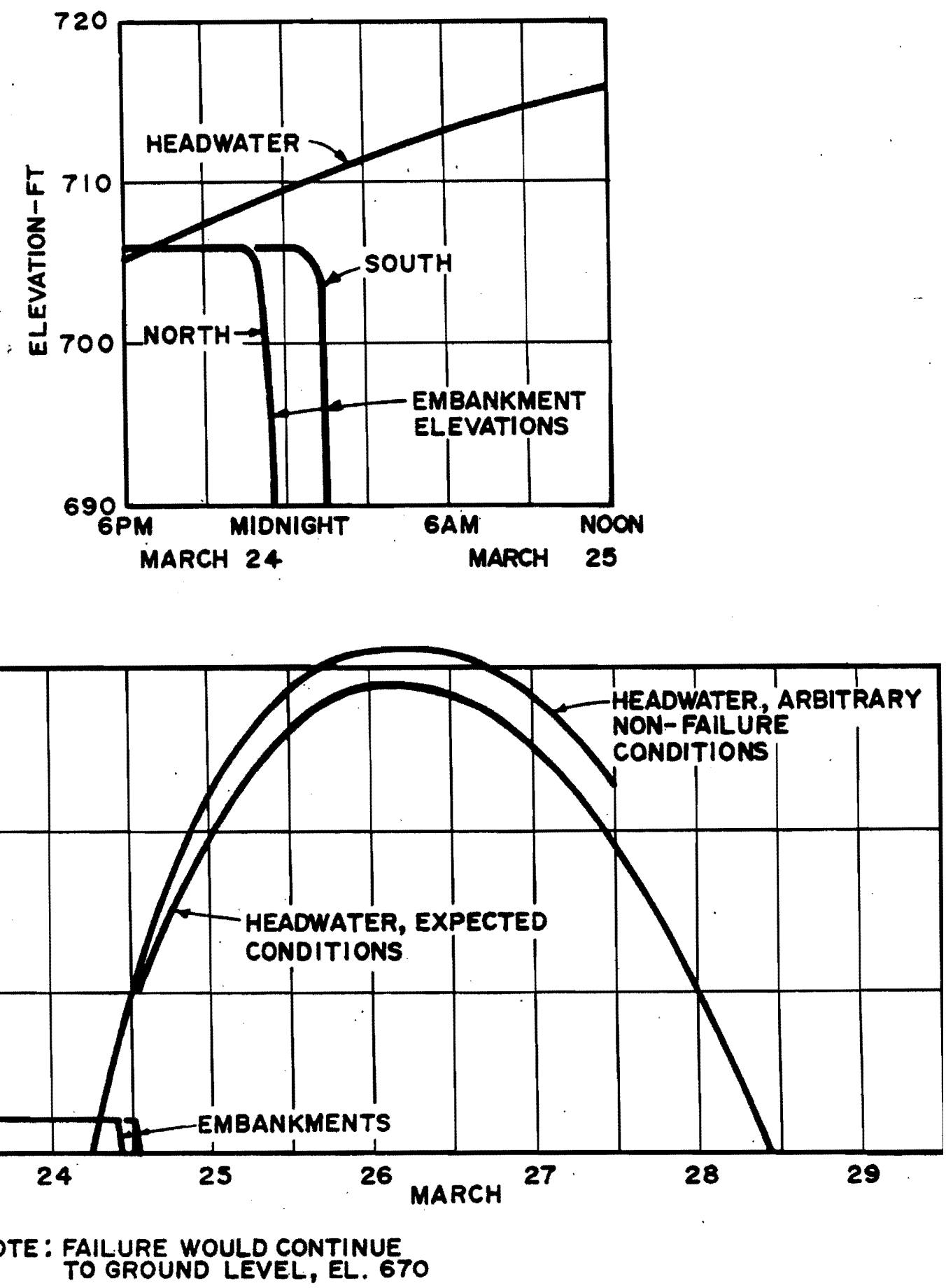
Figure 2.4-56 Watts Bar Probable Maximum Flood Watts Bar Dam Outflow



Watts Bar Nuclear Plant Probable Maximum Flood Elevation
Figure 2.4 - 57

FSAR - Amendment 92

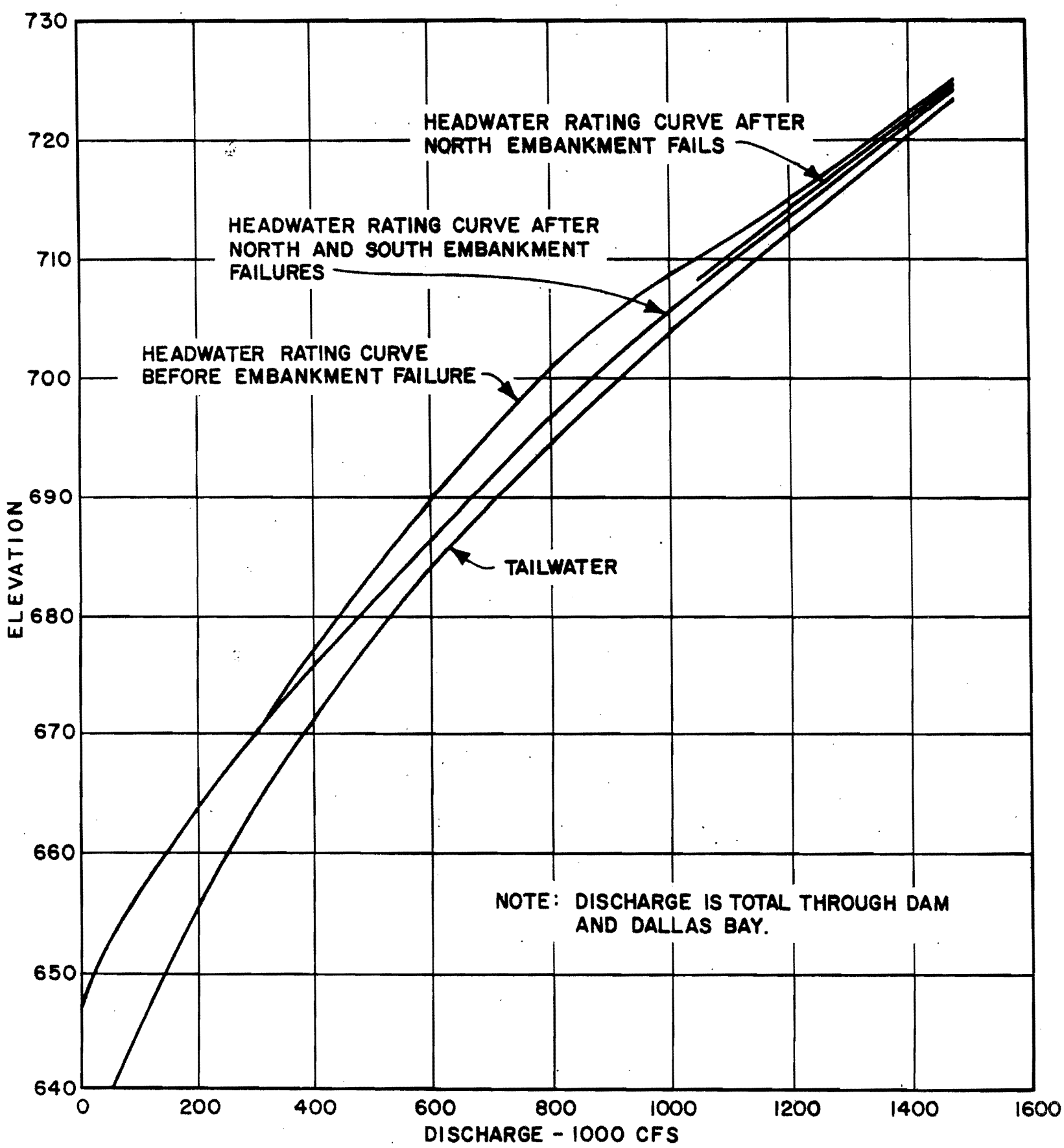
Figure 2.4-57 Watts Bar Probable Maximum Flood Watts Bar Headwater Elevation



WATTS BAR PROBABLE MAXIMUM FLOOD
CHICKAMAUGA HEADWATER ELEVATIONS
FIGURE 2.4-58

Revised by Amendment 32

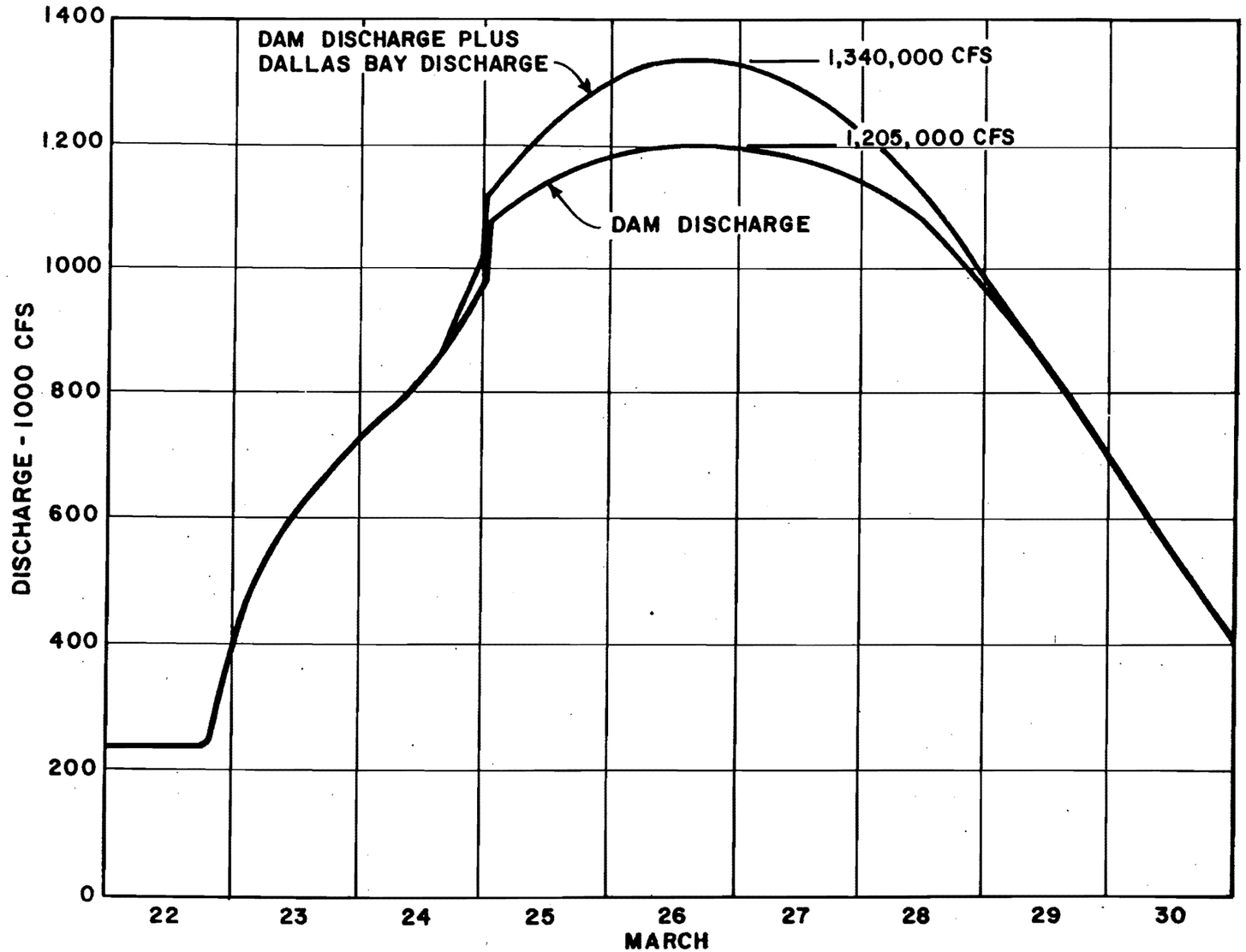
Figure 2.4-58 Watts Bar Probable Maximum Flood Chickamauga Headwater Elevations



CHICKAMAUGA DAM RATING CURVES
FIGURE 2.4-59

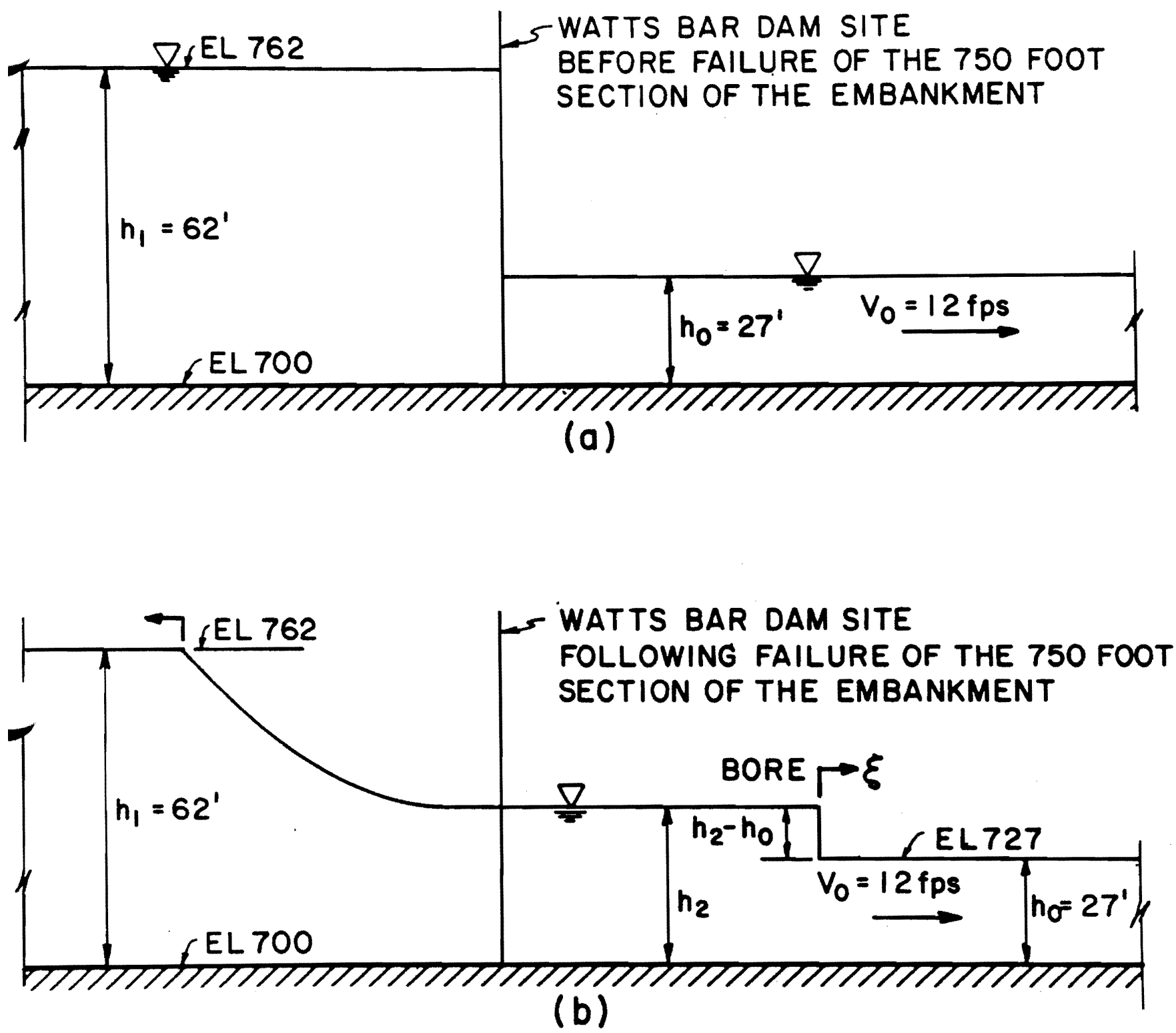
Revised by Amendment 32

Figure 2.4-59 Chickamauga Darn Rating Curves



WATTS BAR PROBABLE MAXIMUM FLOOD
CHICKAMAUGA OUTFLOW
FIGURE 2.4 - 60
Revised by Amendment 32

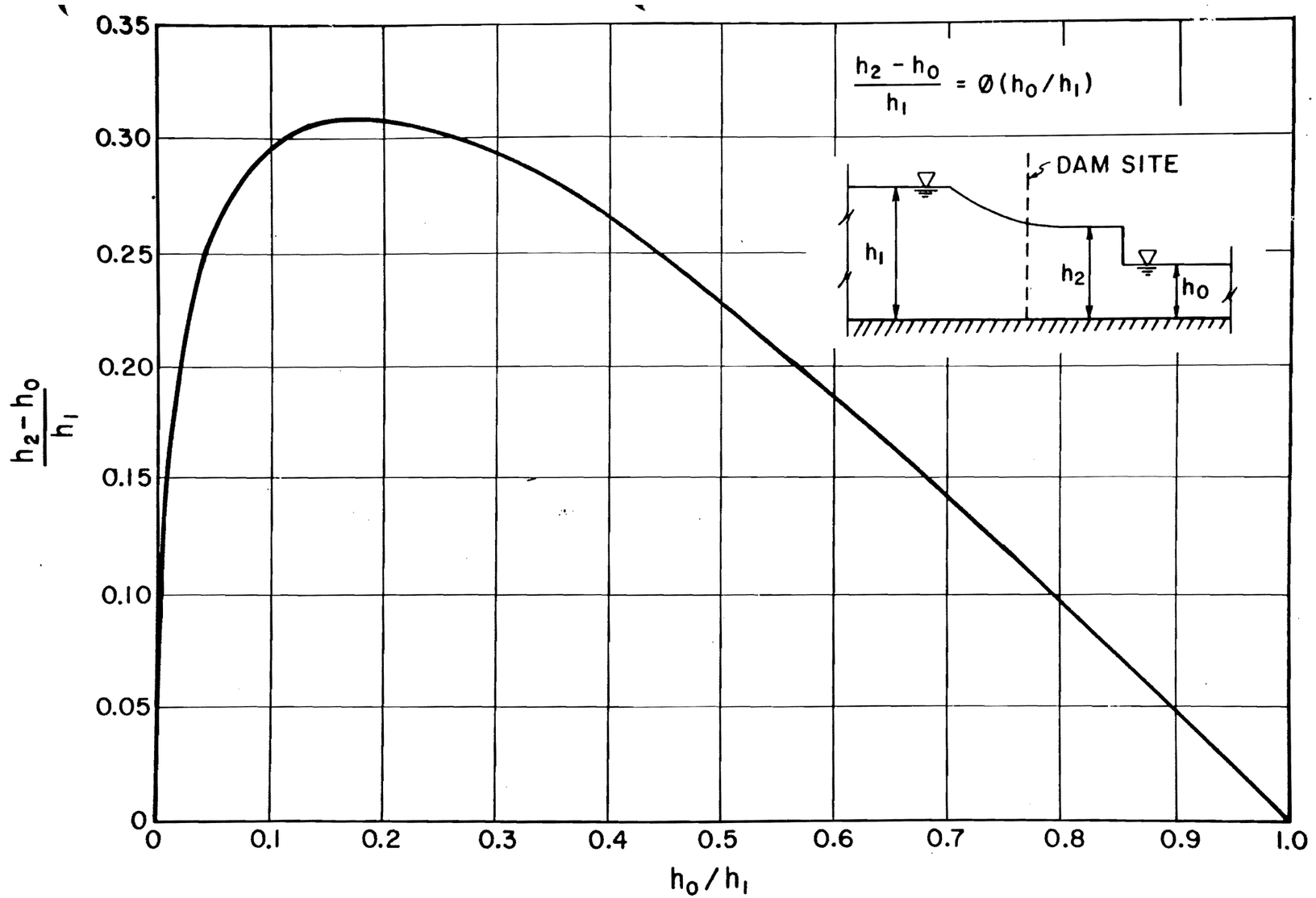
Figure 2.4-60 Watts Bar Probable Maximum Flood Chickamauga Outflow



WATTS BAR PROBABLE MAXIMUM FLOOD
WATER LEVELS BEFORE & AFTER EMBANKMENT FAILURE
FIGURE 2.4-61

Revised by Amendment 39

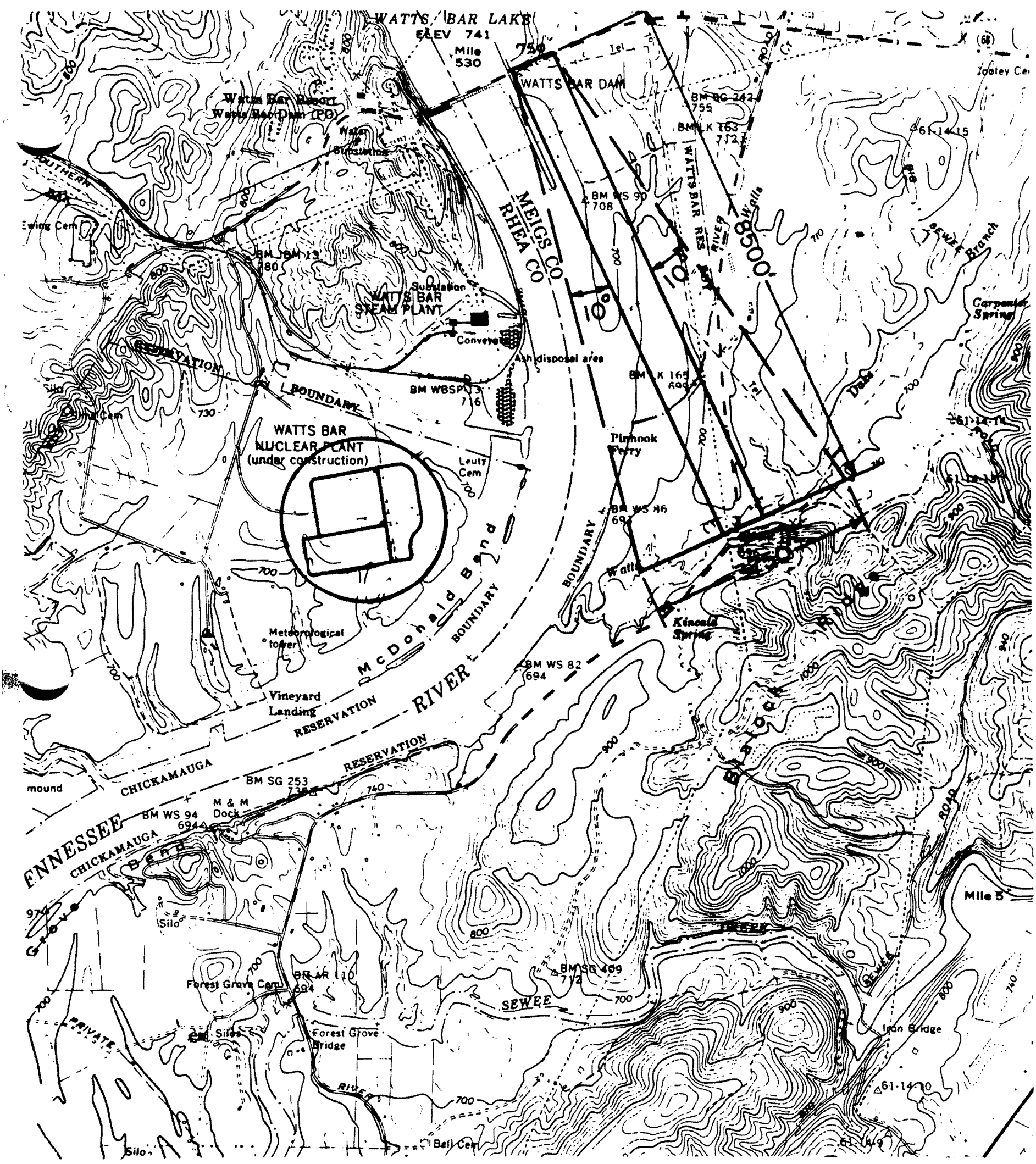
Figure 2.4-61 Watts Bar Probable Maximum Flood Water Levels Before and After Embankment Failure



RELATIVE BORE HEIGHT (AFTER J. J. STROKER, REF. 31)
FIGURE 2.4-62

Revised by Amendment 39

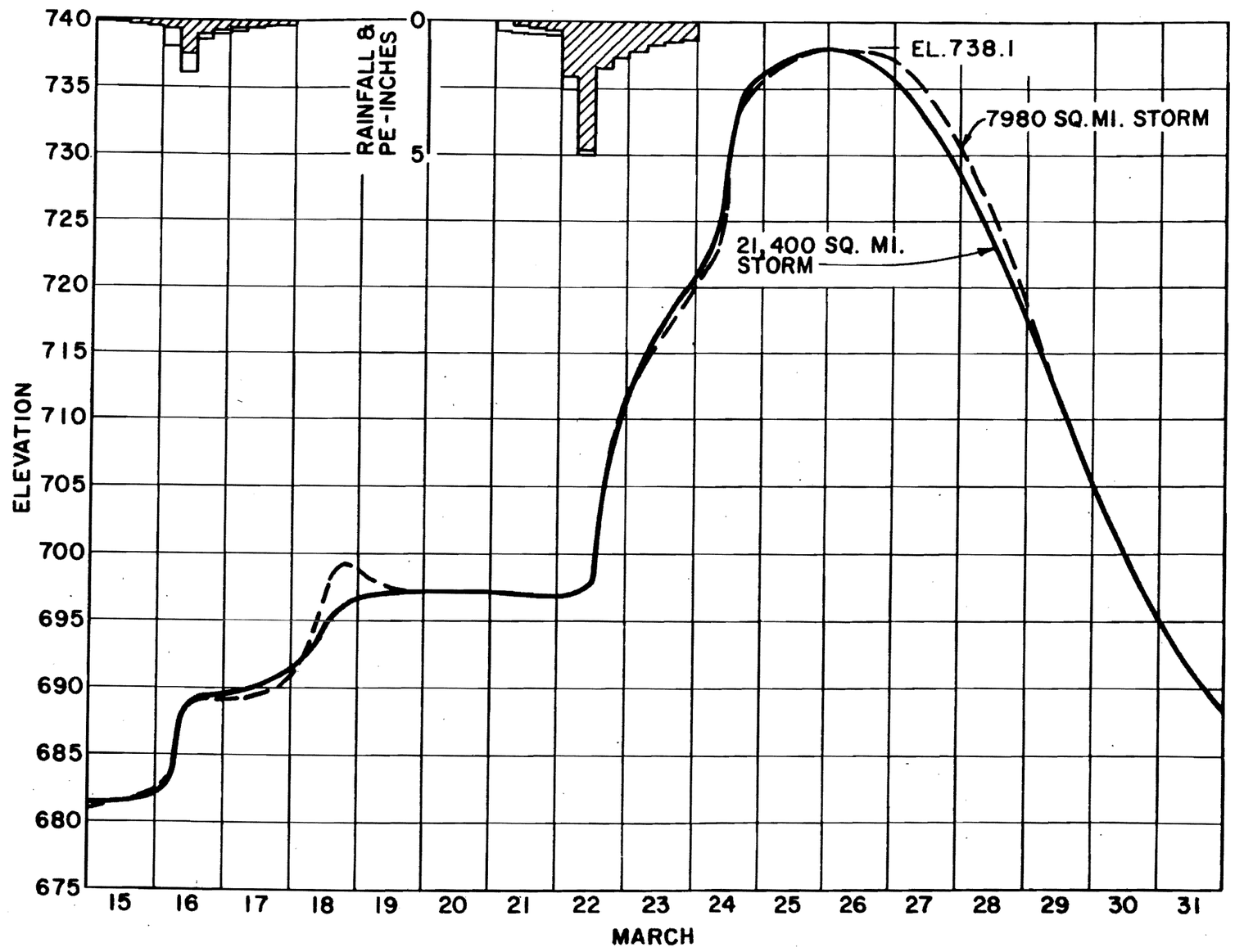
Figure 2.4-62 Relative Bore Height (After J. J. Stroker, REF. 31)



ASSUMED LIMITS OF EMBANKMENT FAILURE
WAVE EXPANSION
FIGURE 2.4-63

Revised by Amendment 39

Figure 2.4-63 Assumed Limits of Embankment Failure Wave Expansion



WATTS BAR NUCLEAR PLANT PROBABLE MAXIMUM FLOOD ELEVATION
FIGURE 2.4-64

Revised by Amendment 32

Figure 2.4-64 Watts Bar Nuclear Plant Probable Maximum Flood Elevation

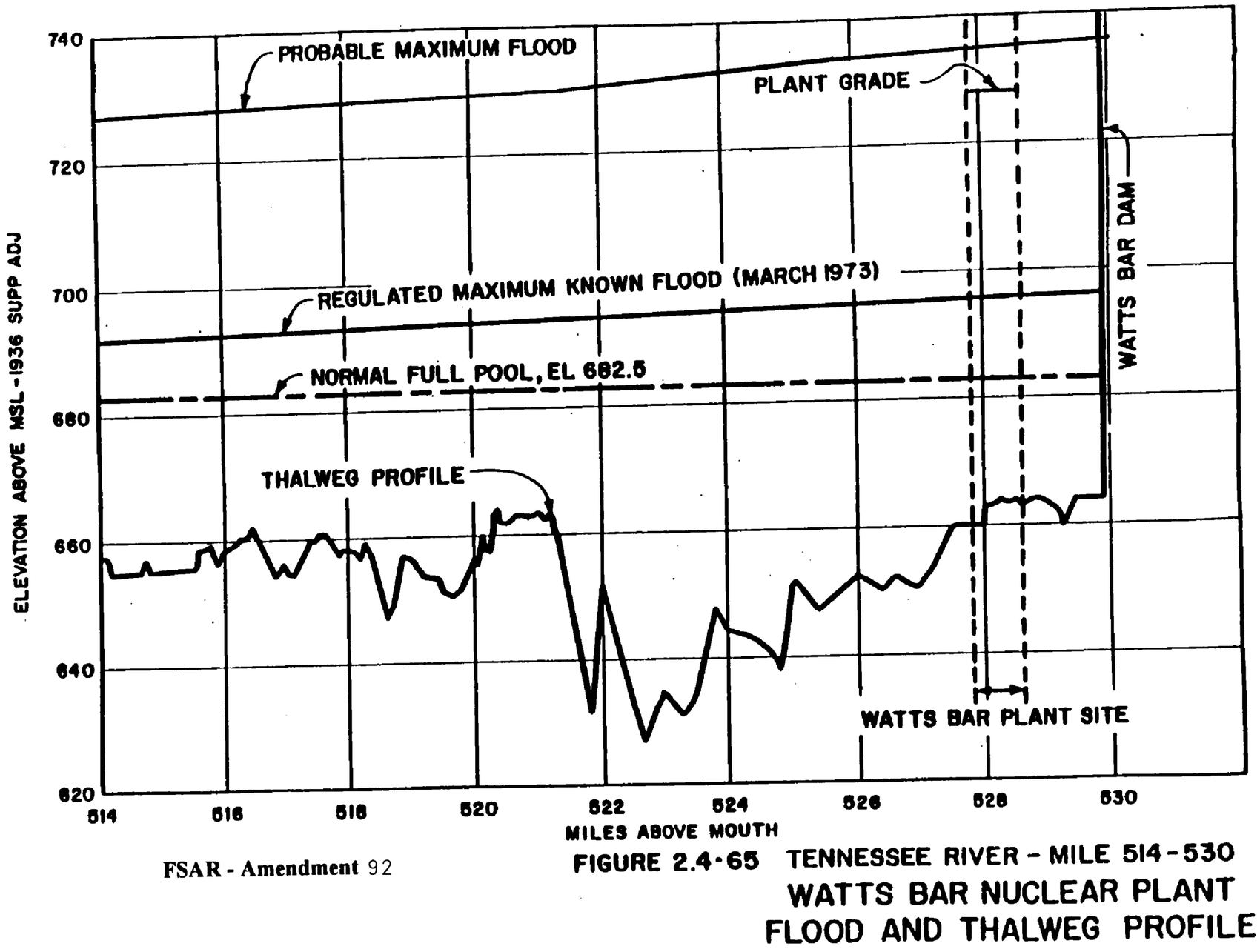


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FIGURE 2.4-65 TENNESSEE RIVER - MILE 514-530
WATTS BAR NUCLEAR PLANT
FLOOD AND THALWEG PROFILE

Figure 2.4-65 Tennessee River - Mile 514-530 - Watts Bar Nuclear Plant Flood and Thalweg Profile

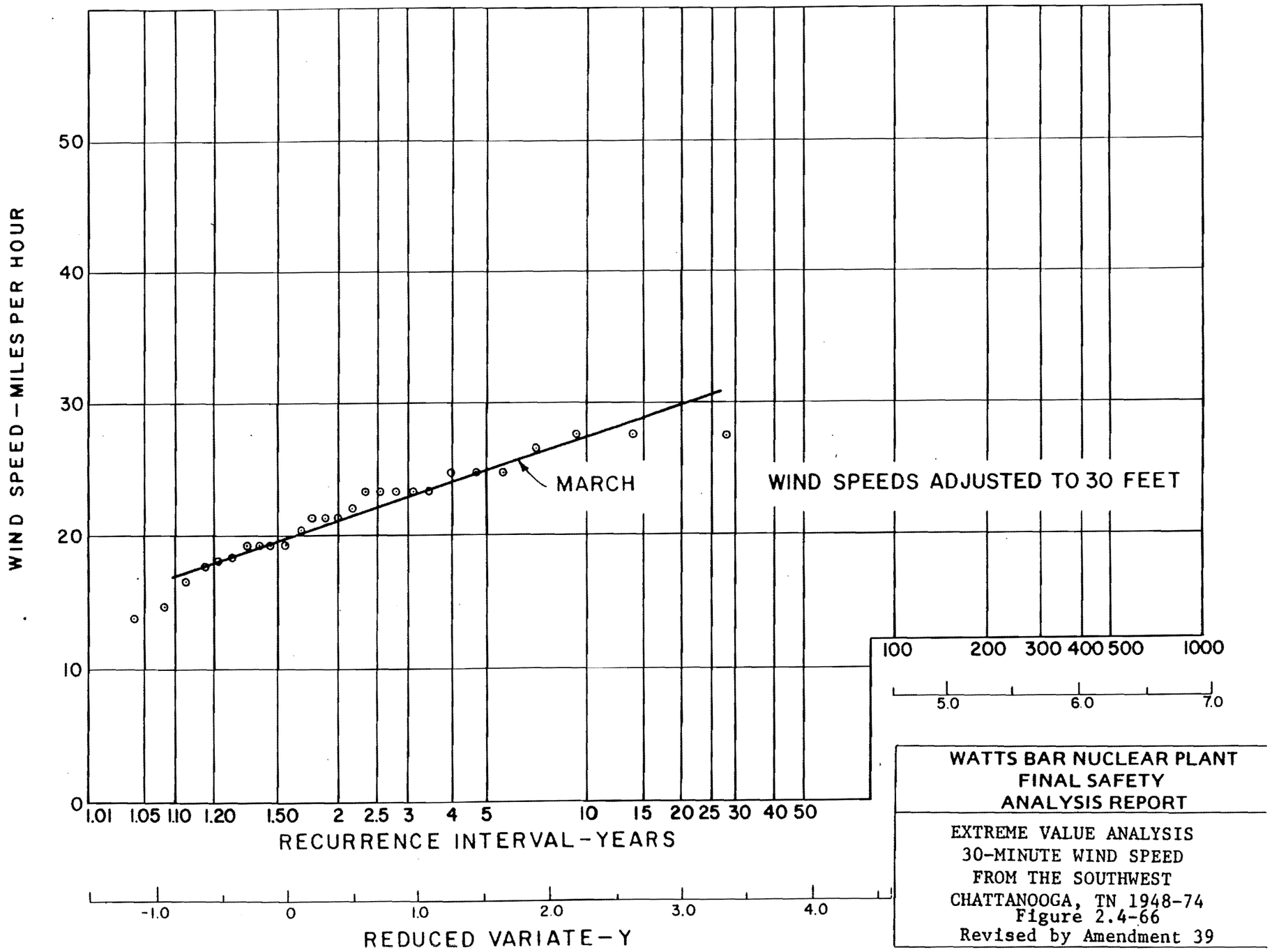
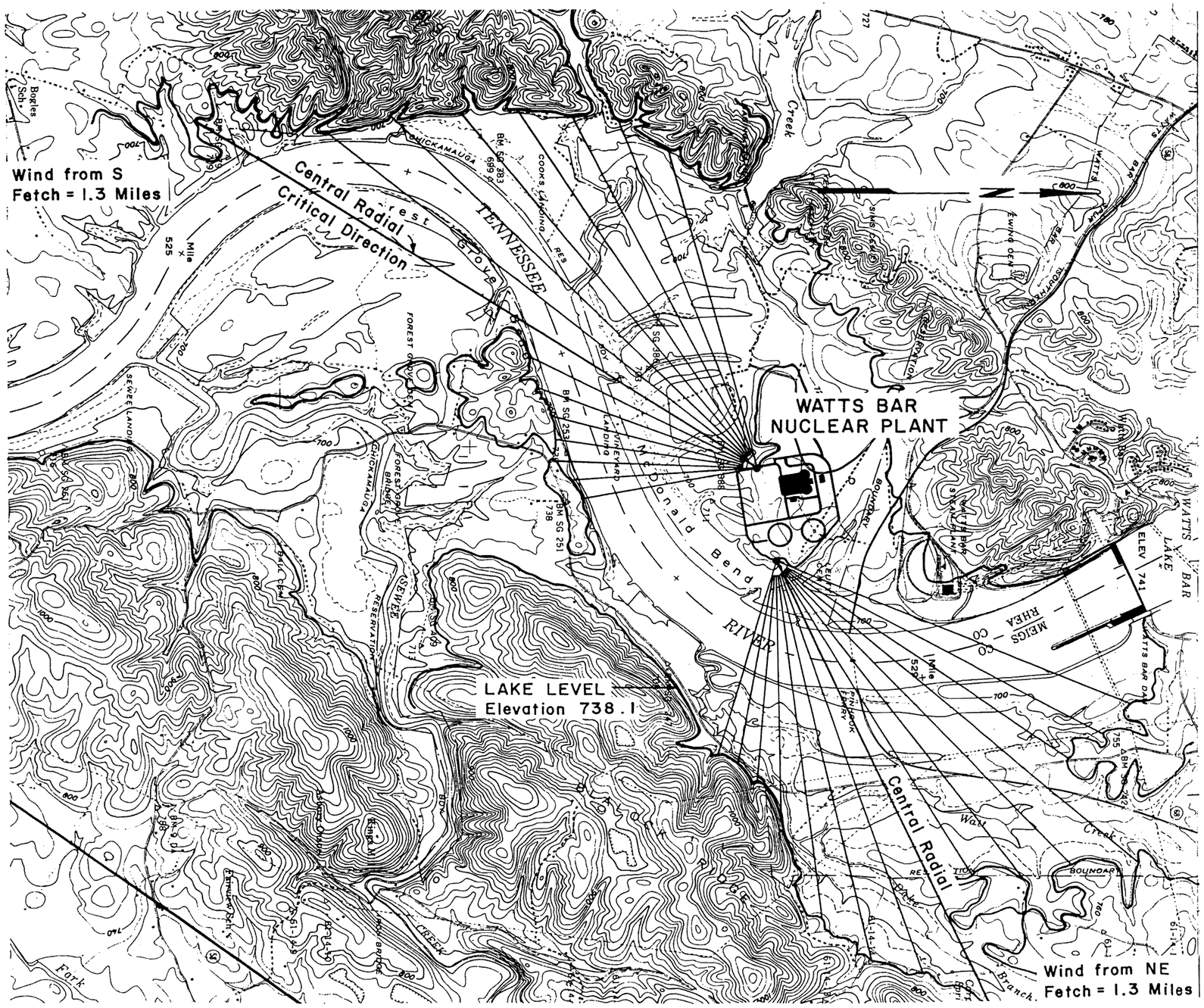


Figure 2.4-66 Extreme Value Analysis 30-Minute Wind Speed From The Southwest Chattanooga, TN 1948-74



WATTS BAR NUCLEAR PLANT
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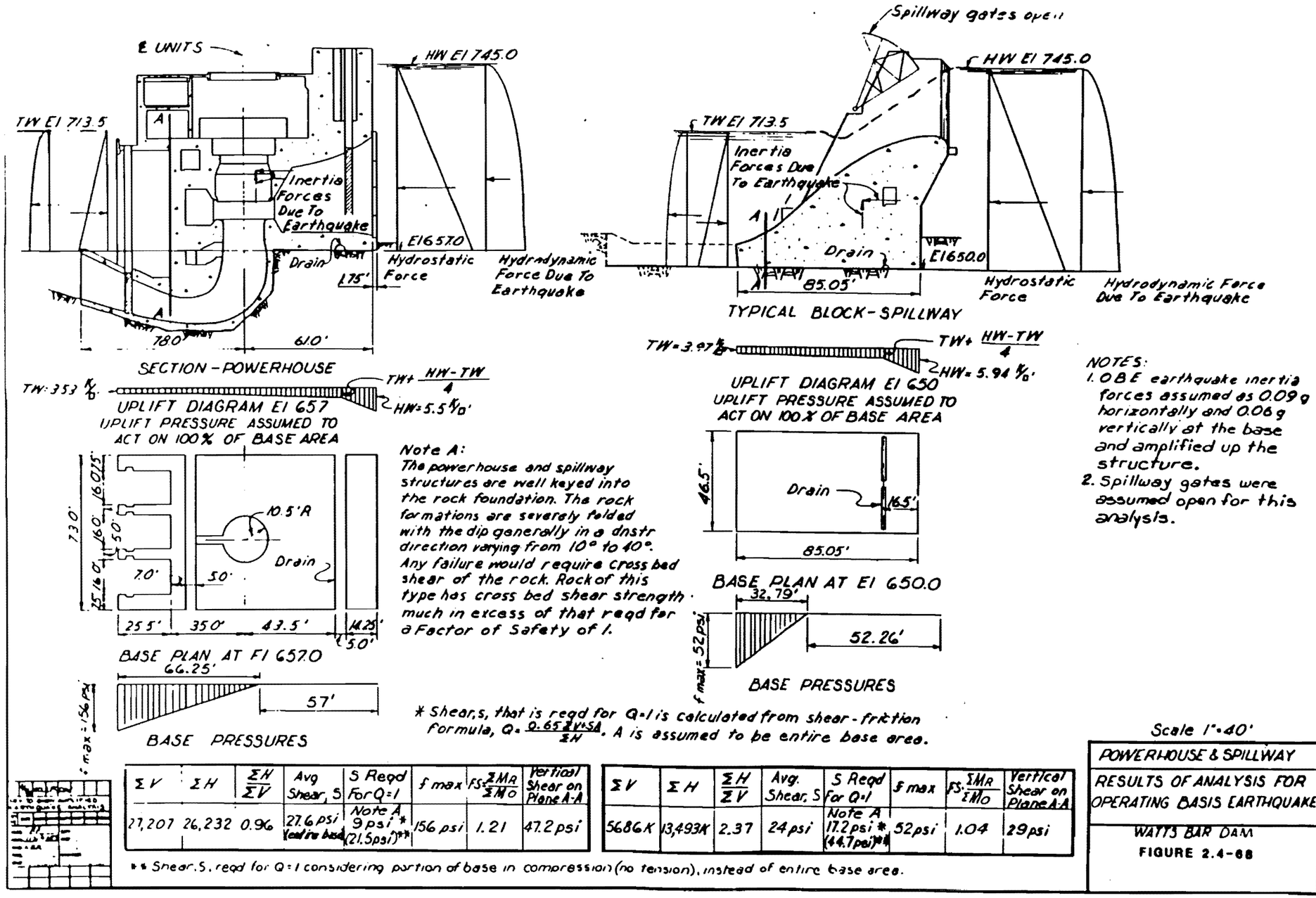
WATTS BAR NUCLEAR PLANT
WIND WAVE FETCH
Figure 2.4-67

SCALE 0 1000 2000 3000 4000 FEI

Figure 2.4-67 Watts Bar Nuclear Plant Wind Wave Fetch



Note A:
The powerhouse and spillway structures are well keyed into the rock foundation. The rock formations are severely folded with the dip generally in a dnstr direction varying from 10° to 40°. Any failure would require cross bed shear of the rock. Rock of this type has cross bed shear strength much in excess of that reqd for a Factor of Safety of 1.

NOTES:
1. OBE earthquake inertia forces assumed as 0.09 g horizontally and 0.06 g vertically at the base and amplified up the structure.
2. Spillway gates were assumed open for this analysis.

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

ΣV	ΣH	ΣH/ΣV	Avg Shear, S	S Reqd For Q=1	f max	FS: ΣMR/ΣMO	Vertical Shear on Plane A-A
27,207	26,232	0.96	27.6 psi (cut fro base)	Note A 9 psi * (21.5 psi)**	156 psi	1.21	47.2 psi

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

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

Scale 1" = 40'

POWERHOUSE & SPILLWAY
RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE
WATTS BAR DAM
FIGURE 2.4-68

Amendment 63

Figure 2.4-68 Powerhouse & Spillway Results of Analysis For Operating Basis Earthquake - Watts Bar Dam

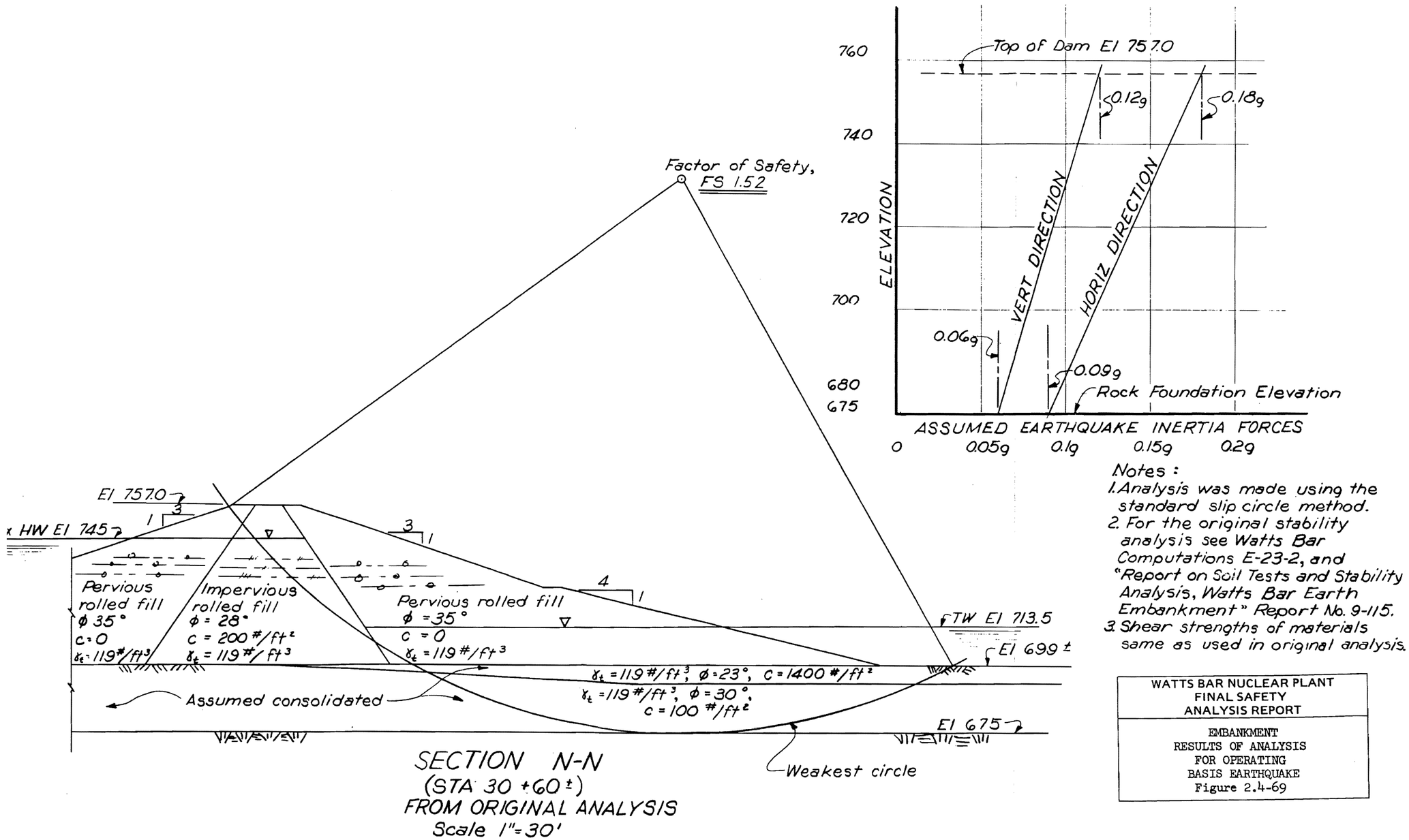
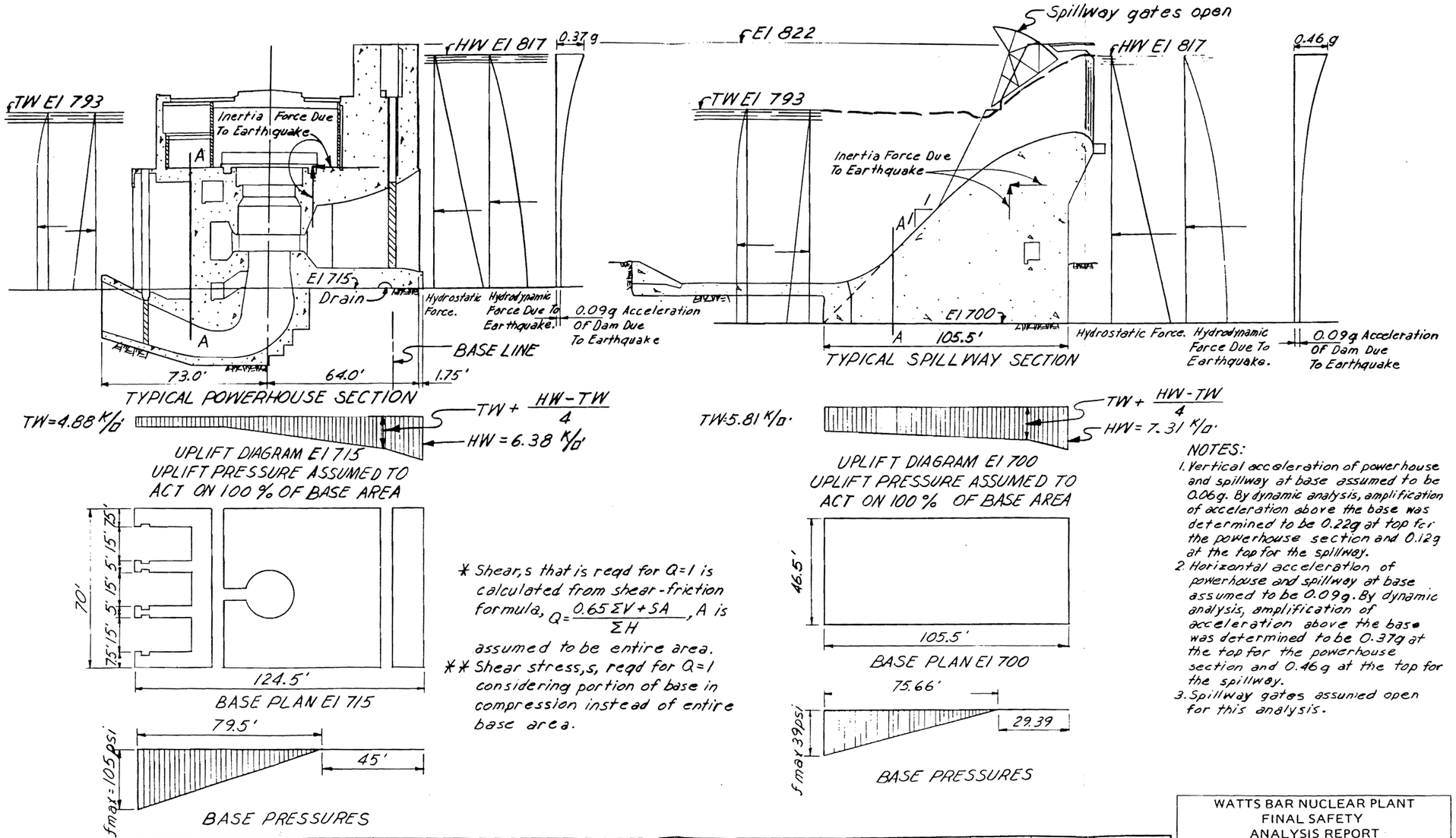


Figure 2.4-69 Embankment Results of Analysis For Operating Basis Earthquake

Figure 2.4-70 Deleted by Amendment 83



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

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

NOTES:

1. Vertical acceleration of powerhouse and spillway at base assumed to be 0.06g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.22g at top for the powerhouse section and 0.12g at the top for the spillway.
2. Horizontal acceleration of powerhouse and spillway at base assumed to be 0.09g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.37g at the top for the powerhouse section and 0.46g at the top for the spillway.
3. Spillway gates assumed open for this analysis.

ΣV	ΣH	ΣH/ΣV	Avg shear	S reqd for Q=1	f max	F.S. = ΣMR/ΣMo	Vertical shear on plane A-A
22,762K	22,541K	0.99	24.2psi (entire base)	8.3psi * (14.6psi)**	105psi	1.18	26 psi

ΣV	ΣH	ΣH/ΣV	Avg shear	S reqd for Q=1	f max	F.S. = ΣMR/ΣMo	Vertical shear on plane A-A
19,488K	14,382K	0.75	20psi (entire base)	2.38psi * (3.30psi)**	39psi	1.26	17psi

WATTS BAR NUCLEAR PLANT
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POWERHOUSE & SPILLWAY
RESULTS OF ANALYSIS FOR
OPERATING BASIS EARTHQUAKE
Fort Loudoun Dam
Figure 2.4-71

Figure 2.4-71 Powerhouse & Spillway Results of Analysis For Operating Basis Earthquake - Fort Loudoun Dam

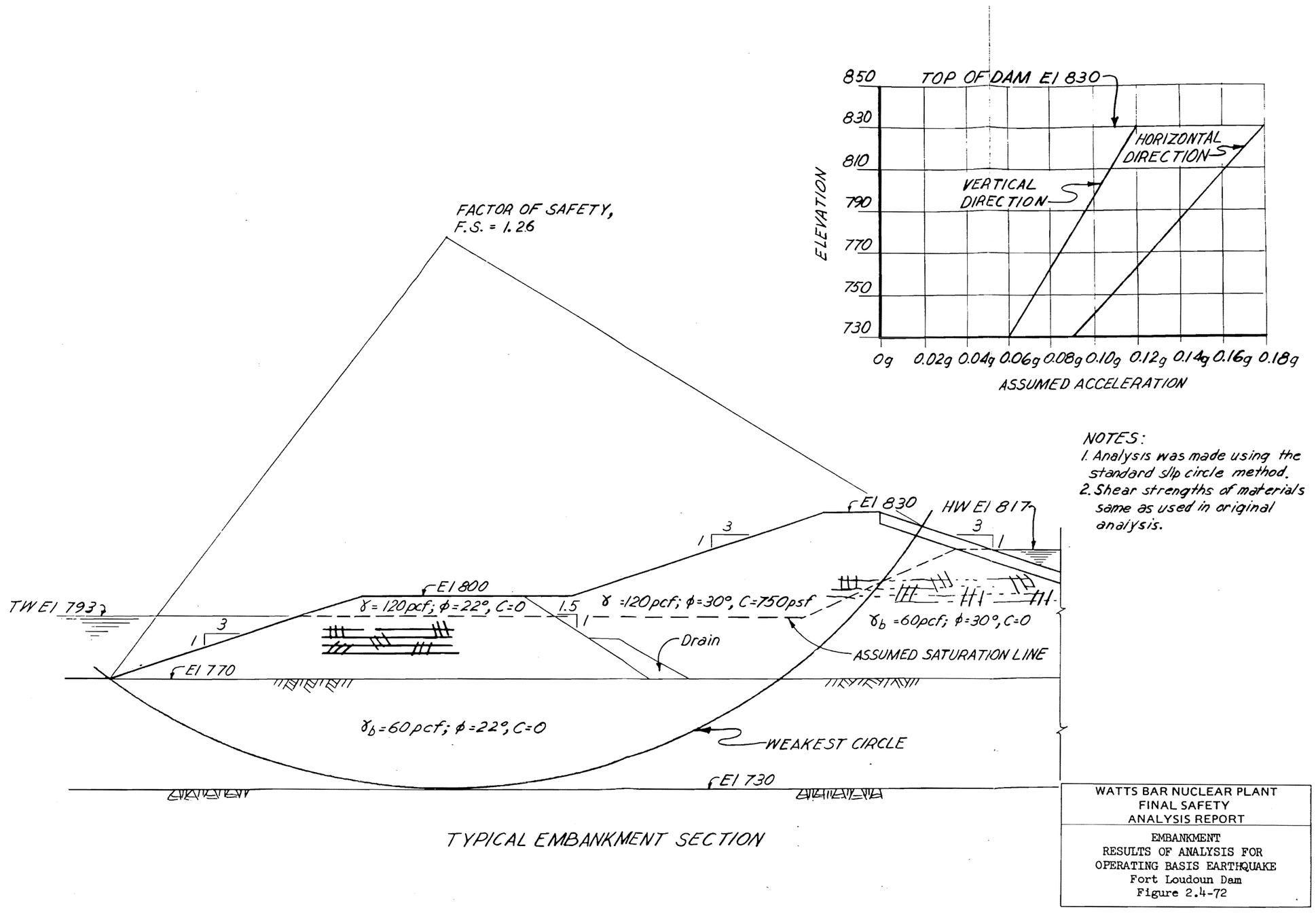
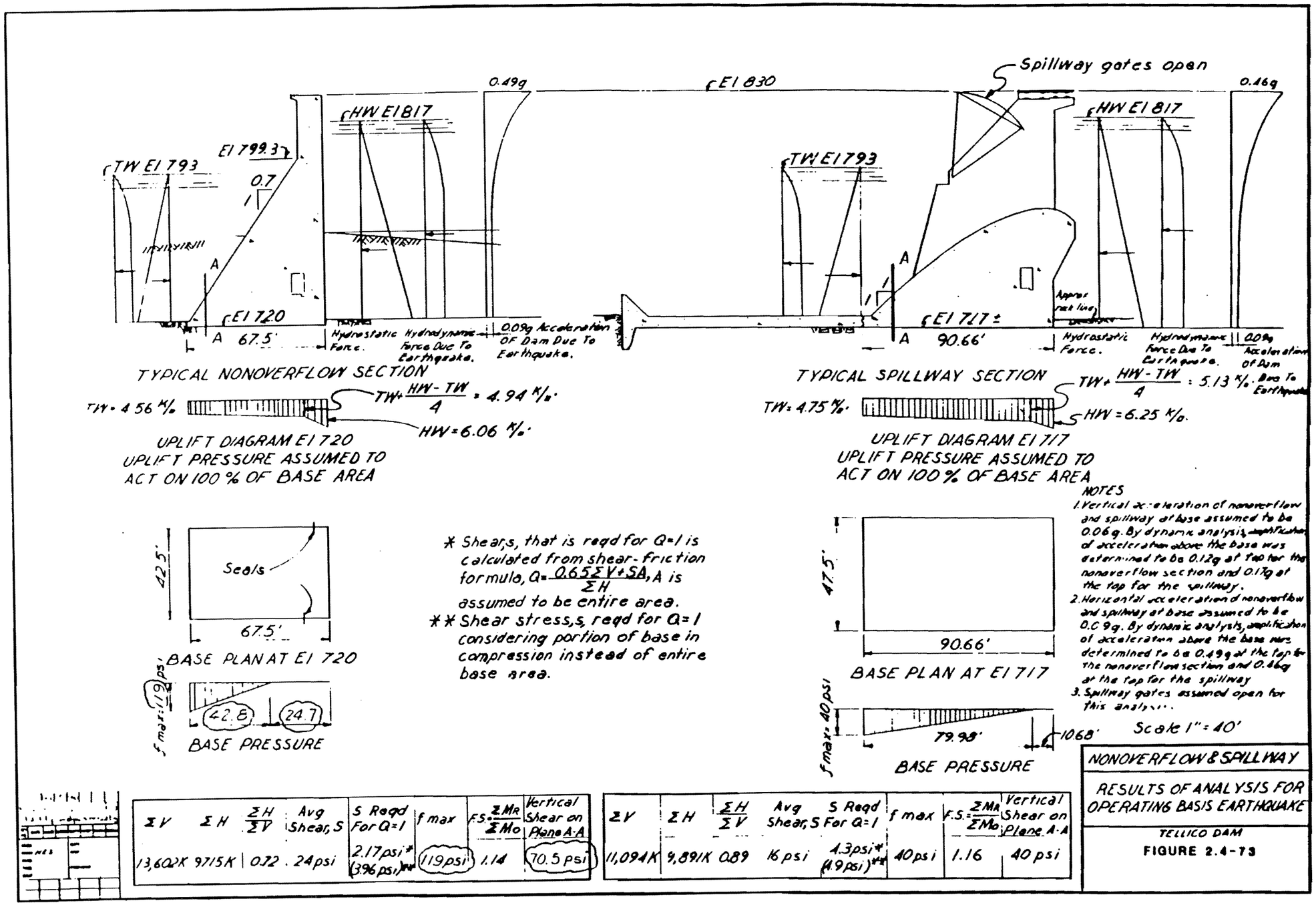


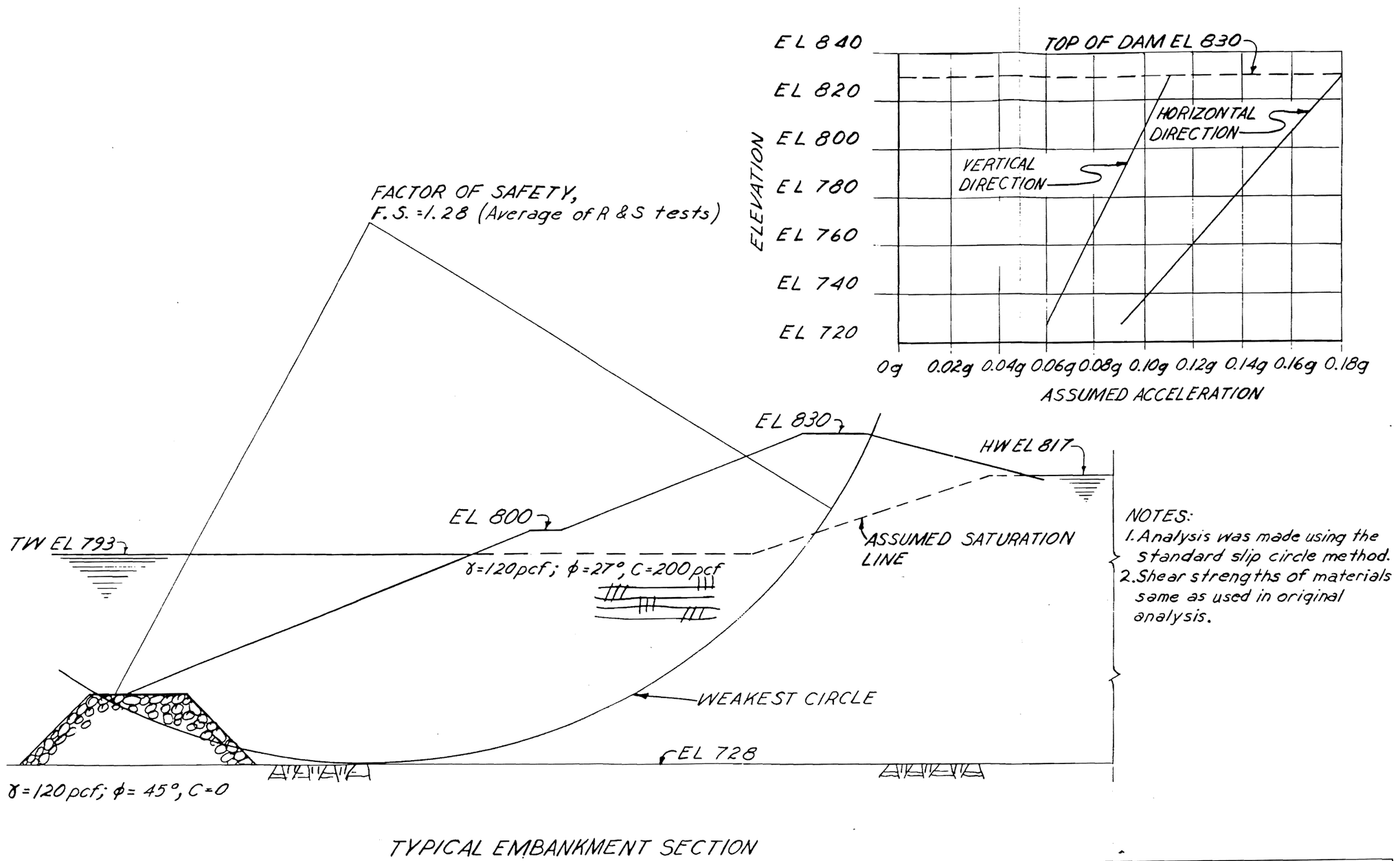
Figure 2.4-72 Embankment Results Of Analysis For Operating Basis Earthquake - Fort Loudoun Darn



ΣV	ΣH	ΣH/ΣV	Avg Shear, S	S Reqd For Q=1	f max	F.S. = ΣMR/ΣMo	Vertical Shear on Plane A-A
13,602K	9715K	0.72	24psi	2.17psi* (3.96psi)**	119psi	1.14	70.5 psi

ΣV	ΣH	ΣH/ΣV	Avg Shear, S	S Reqd For Q=1	f max	F.S. = ΣMR/ΣMo	Vertical Shear on Plane A-A
11,094K	9,891K	0.89	16 psi	4.3psi* (4.9psi)**	40psi	1.16	40 psi

Figure 2.4-73 Nonoverflow & Spillway Results of Analysis For Operating Basis Earthquake - Tellico Dam

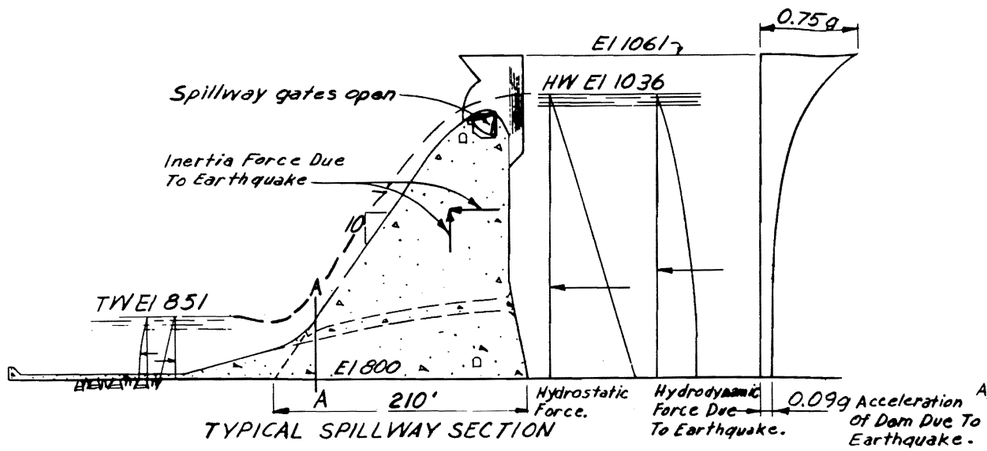


TYPICAL EMBANKMENT SECTION

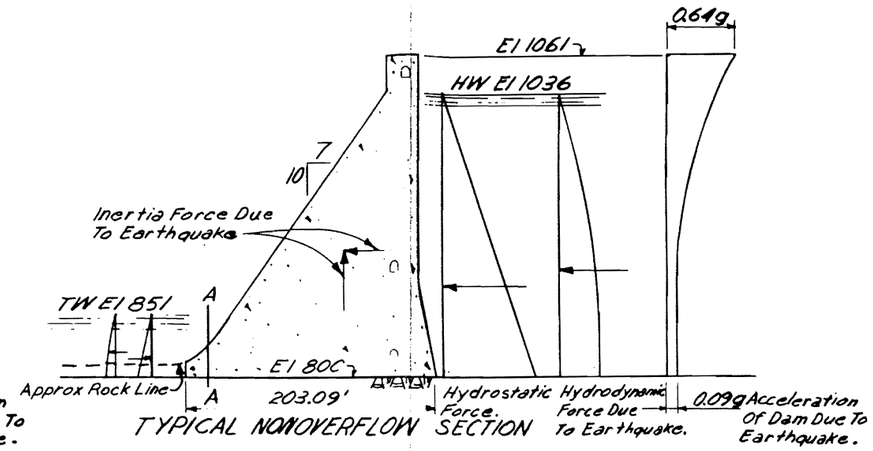
WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

EMBANKMENT
RESULTS FOR ANALYSIS FOR
OPERATING BASIS EARTHQUAKE
Tellico Dam
Figure 2.4-74

Figure 2.4-74 Embankment Results For Analysis For Operating Basis Earthquake - Tellico Darn

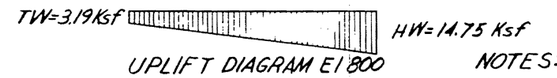


TYPICAL SPILLWAY SECTION



TYPICAL NONOVERFLOW SECTION

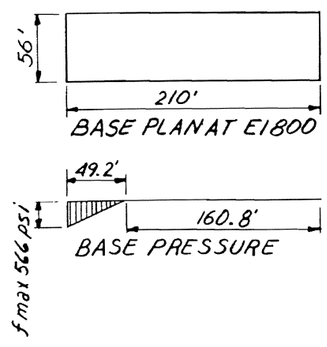
UPLIFT DIAGRAM El 800
UPLIFT PRESSURE ASSUMED
TO ACT ON 100 % OF BASE AREA

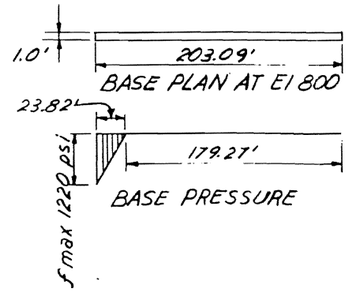


UPLIFT DIAGRAM El 800
UPLIFT PRESSURE ASSUMED
TO ACT ON 100 % OF BASE AREA

NOTES:

1. Vertical acceleration of nonoverflow and spillway at base assumed to be 0.06g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.14g at the top for the nonoverflow section and 0.14g at the top for the spillway.
2. Horizontal acceleration of nonoverflow and spillway at base assumed to be 0.09g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.64g at the top for the nonoverflow section and 0.75g at the top for the spillway.
3. Spillway gates assumed open for this analysis.





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

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

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, s	S Reqd For Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on plane A-A
112,616 K	143,587 K	1.28	85 psi (entire base)	42 psi* (177 psi)**	566 psi	1.25	247 psi

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, s	S Reqd For Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on plane A-A
2101 K	2786 K	1.33	95 psi (entire base)	49 psi* (415 psi)**	1220 psi	1.03	535 psi

WATTS BAR NUCLEAR PLANT
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ANALYSIS REPORT

SPILLWAY & NONOVERFLOW
RESULTS OF ANALYSIS FOR
OBE + ½ PMF
Norris Dam
Figure 2.4-75

Figure 2.4-75 Spillway & Nonoverflow Results of Analysis For OBE & 1/2 PMF-Norris Dam

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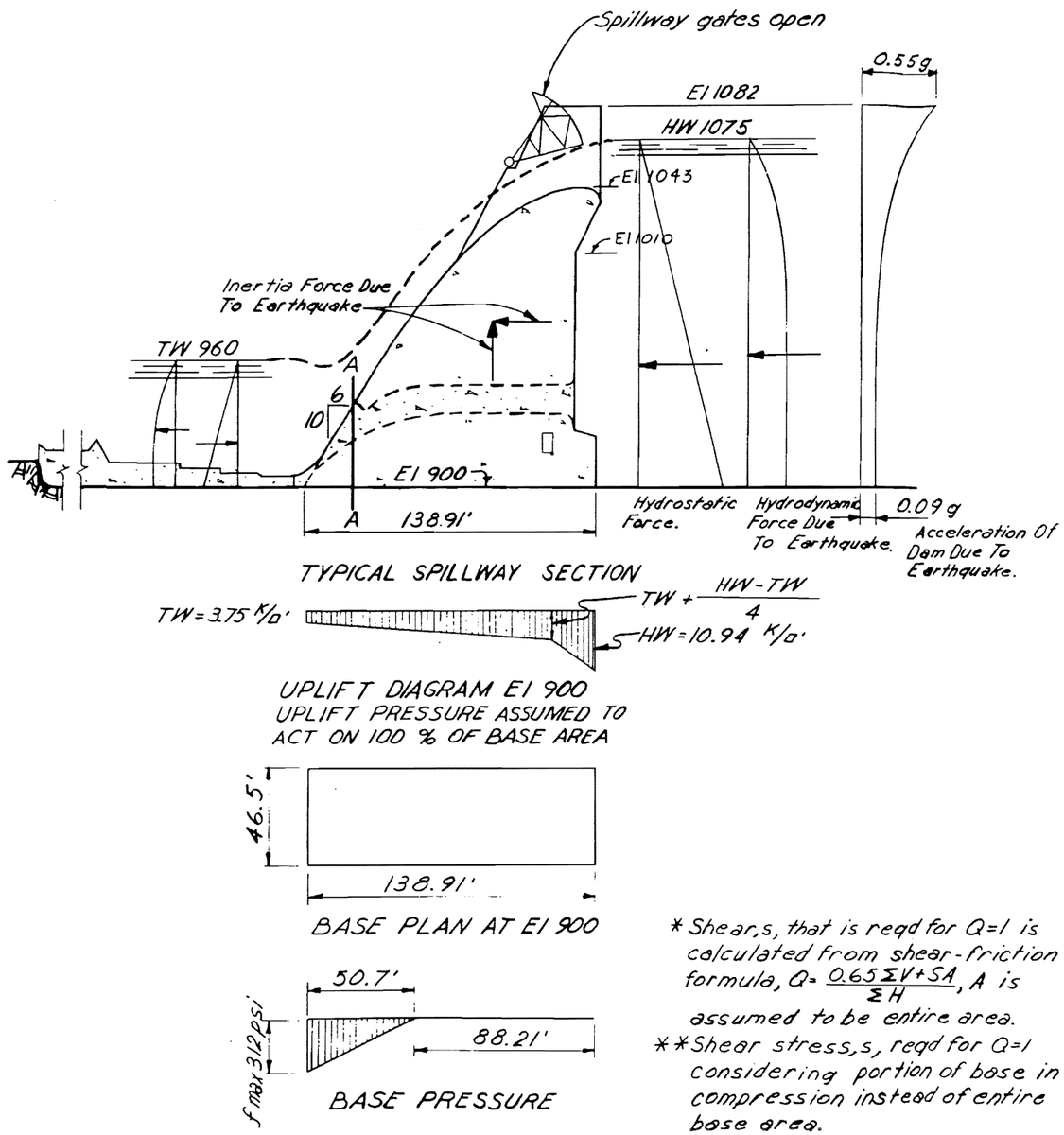
Figure 2.4-76 Analysis For OBE & 1/2 PMF Assumed Condition of Dam After Failure Norris Dam



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

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

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, S	S Reqd For Q=1	f max	F.S. $\frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on Plane A-A
53,007K	57,276K	1.08	61 psi (entire base)	25 psi * (67 psi)**	312 psi	1.13	173 psi

NOTES:

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

WATTS BAR NUCLEAR PLANT
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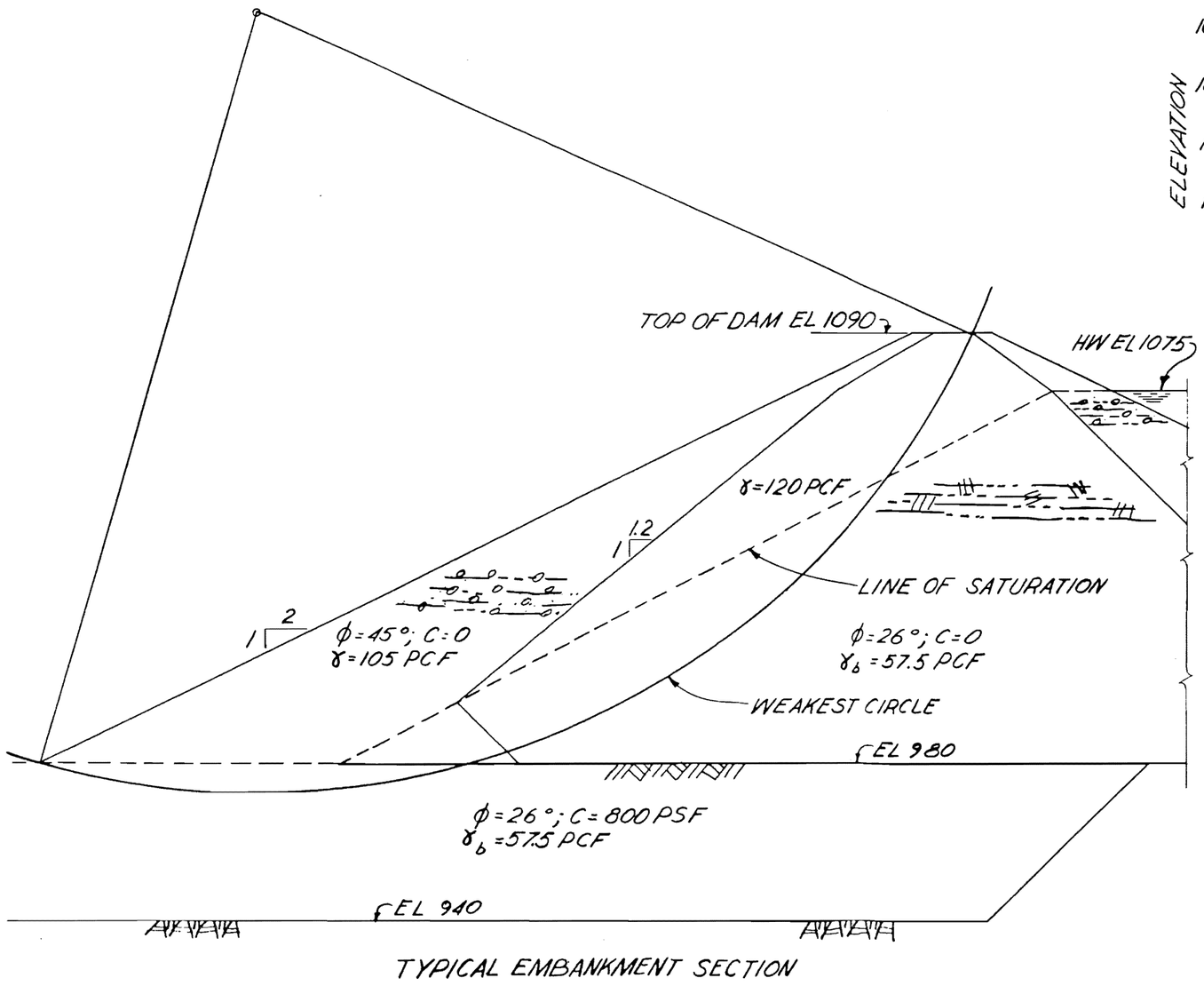
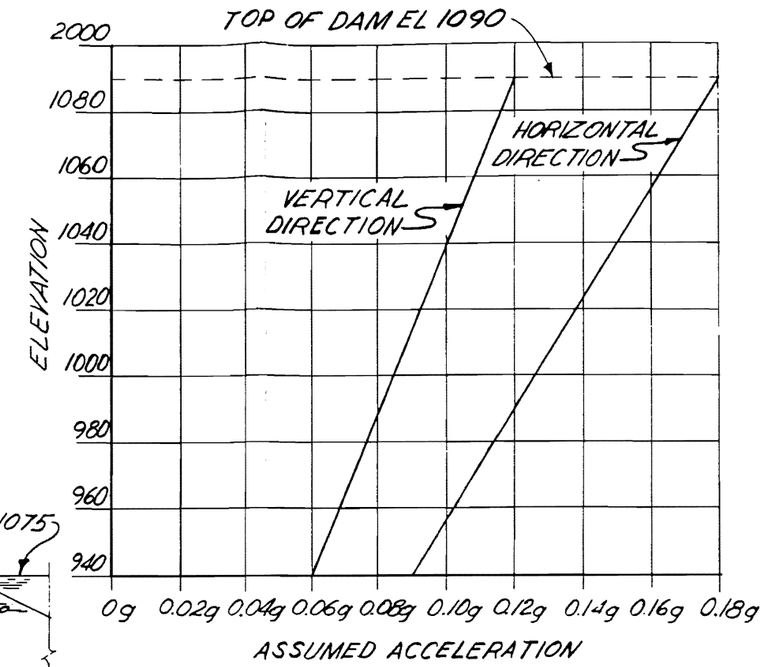
Cherokee Dam

Figure 2.4-77 Spillway & Nonoverflow Results of Analysis For Operating Basis Earthquake -Cherokee Dam



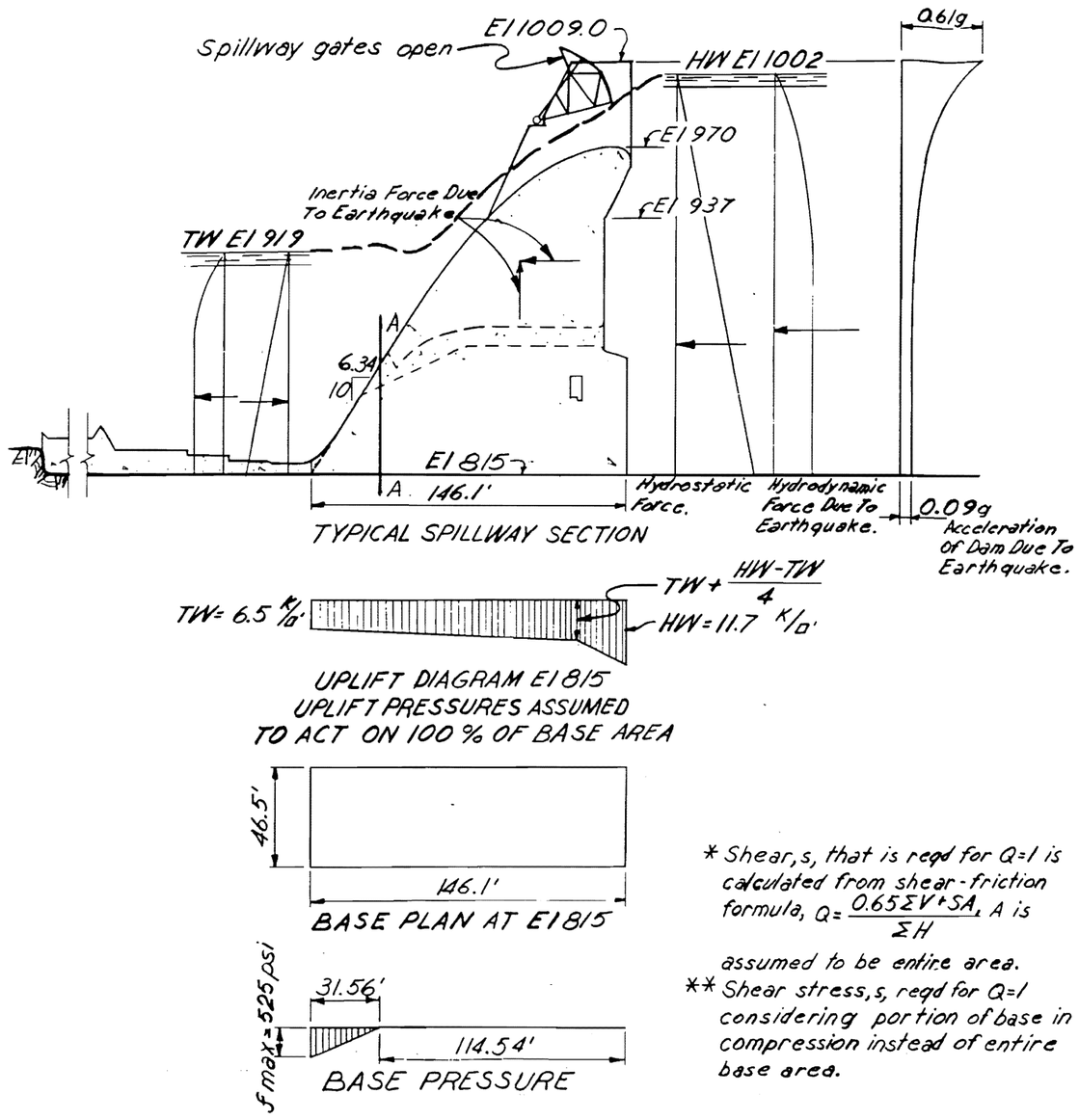


NOTES:
1. Analysis was made using the standard slip circle method.
2. Shear strengths of materials same as used in original analysis.

WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
EMBANKMENT RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE Cherokee Dam Figure 2.4-78

Figure 2.4-78 Embankment Results of Analysis For Operating Basis Earthquake - Cherokee Dam

Figure 2.4-79 Assumed Condition of Dam After Failure PBE And 1/2 Probable Max Flood - Cherokee Dam



NOTES:

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

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

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

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, S	S Reqd For Q=1	f max	$F.S. = \frac{\Sigma MR}{\Sigma Mo}$	Vertical Shear on Plane A-A
55,483 K	60,245 K	1.09	61.6 psi (entire base)	25 psi * (170 psi) **	525 psi	1.06	156 psi

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

SPILLWAY & NONOVERFLOW
RESULTS OF ANALYSIS FOR
OPERATING BASIS EARTHQUAKE
Douglas Dam
Figure 2.4-80

Figure 2.4-80 Spillway & Nonoverflow Results of Analysis For Operating Basis Earthquake - Douglas Dam

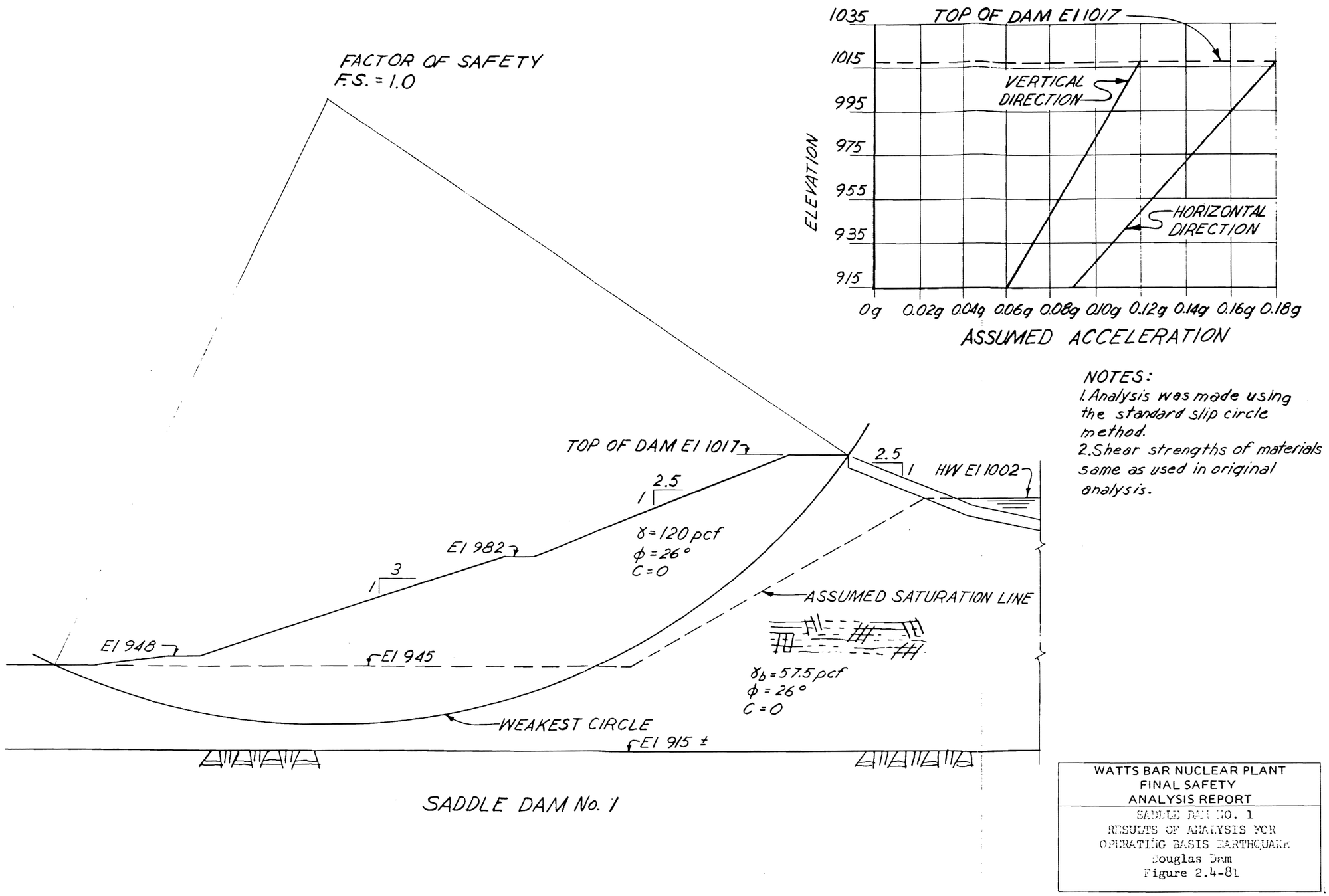


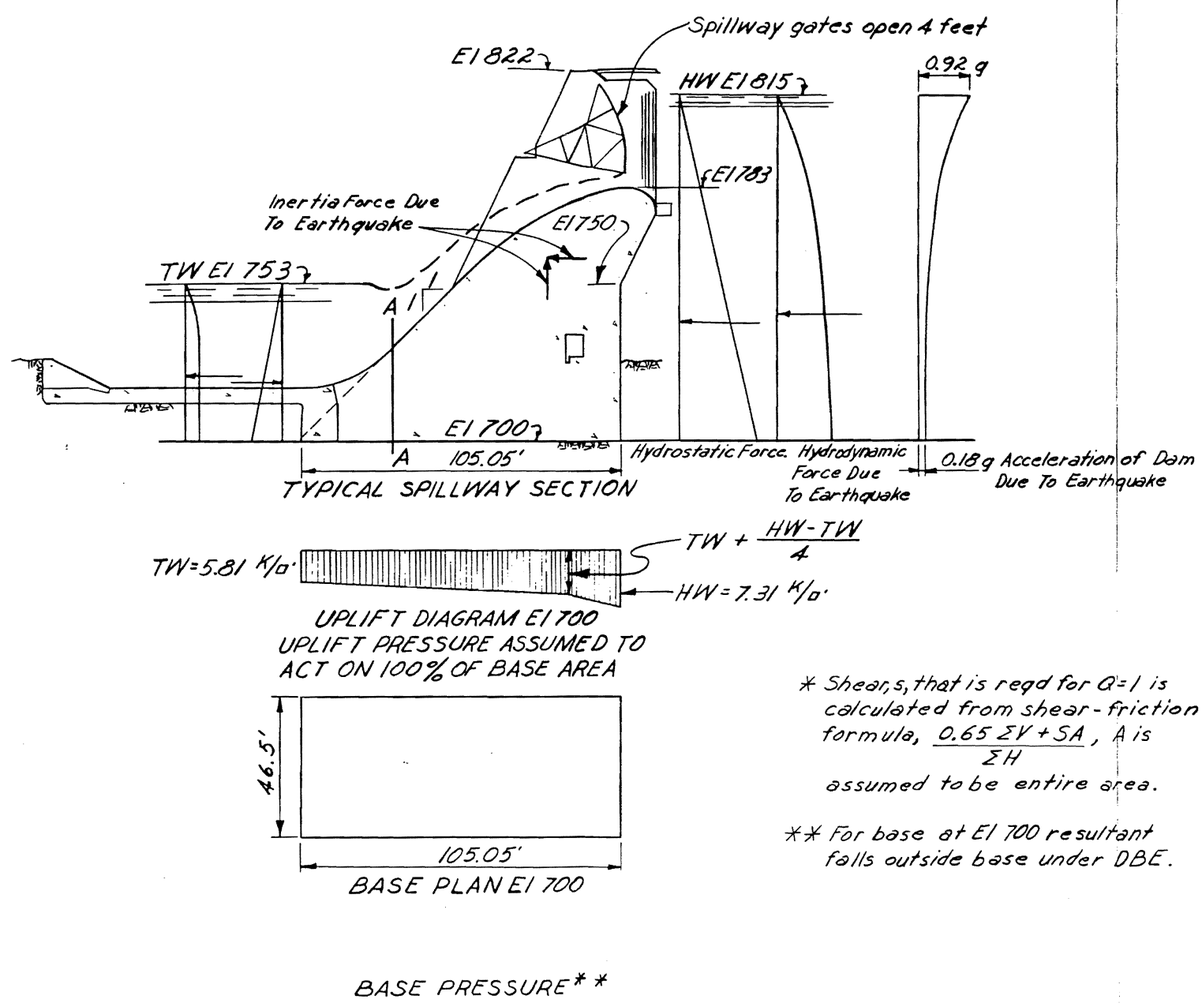
Figure 2.4-81 Saddle Dam No. 1 Results of Analysis For Operating Basis Earthquake - Douglas Dam

Figure 2.4-82 Douglas Dam Assumed Condition of Dam After Failure aBE And 1/2 Probable Maximum Flood - Douglas Project

Figure 2.4-83 Fontana Dam Assumed Condition of Dam after Failure aBE And 1/2 Probable Maximum Flood - Fontana Dam

Figure 2.4-84 Deleted by Amendment 63

Figure 2.4-85 Deleted by Amendment 63



NOTES:

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

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

** For base at El 700 resultant falls outside base under DBE.

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg shear	S reqd for Q=1	f max	$FS \frac{\Sigma M_R}{\Sigma M_O}$
18,254 K	29,534 K	1.62	42 psi (entire base)	25 psi*	**	0.9

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

SPILLWAY
RESULTS OF ANALYSIS FOR
SSE EARTHQUAKE
Fort Loudoun Dam
Figure 2.4-86

Figure 2.4-86 Spillway Results of Analysis For SSE Earthquake Fort Loudoun Dam

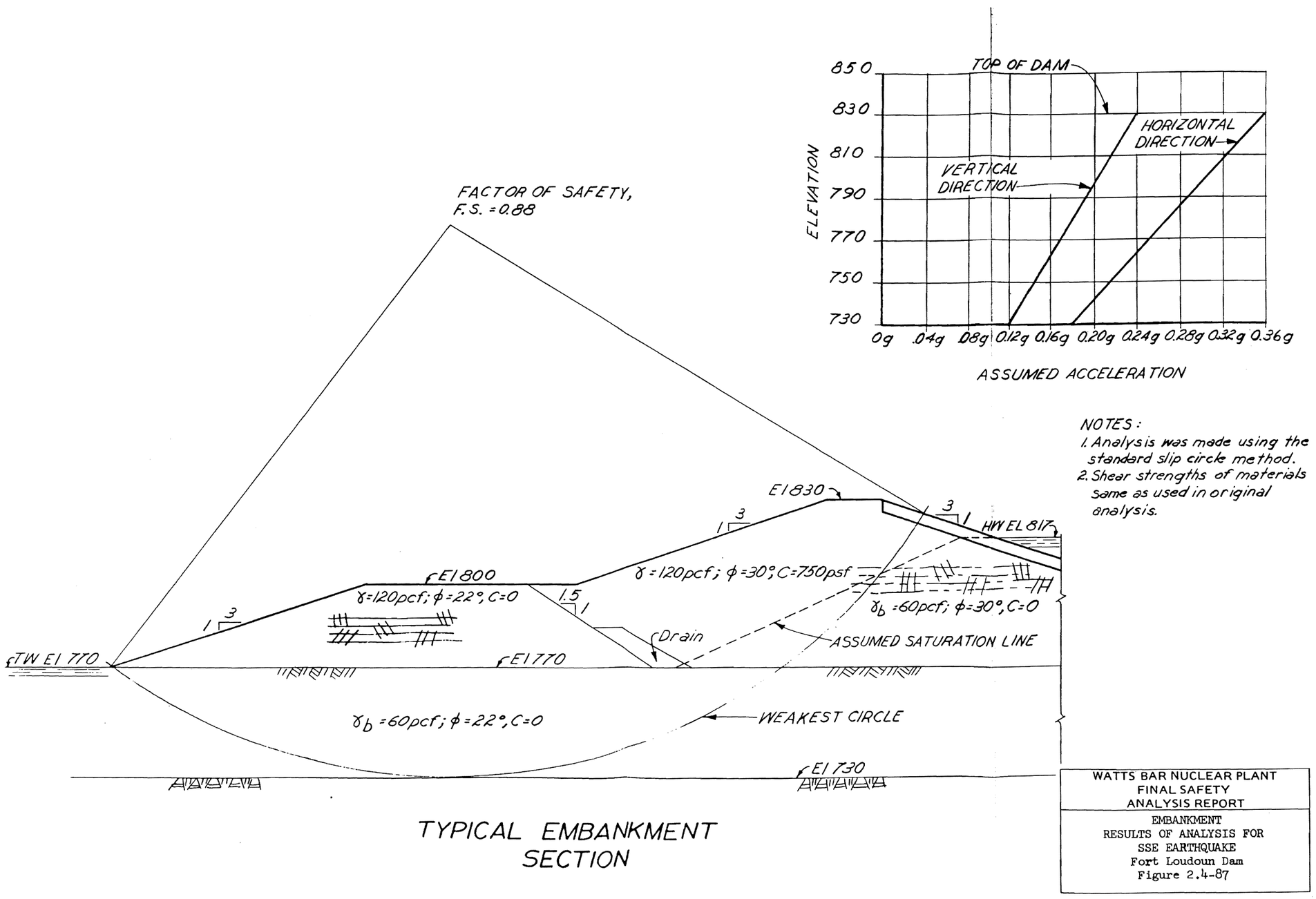


Figure 2.4-87 Embankment Results of Analysis For SSE Earthquake Fort Loudoun Dam

Figure 2.4-88 Fort Loudoun Dam Assumed Condition of Dam After Failure SSE Combined With a 25 Year Flood - Fort Loudoun Dam

Figure 2.4-89 Tellico Dam Assumed Condition of Dam After Failure SSE Combined With a 25 Year Flood Tellico Project

Figure 2.4-90 Norris Dam SSE + 25 Year Flood Judged Condition of Dam After Failure - Norris Dam

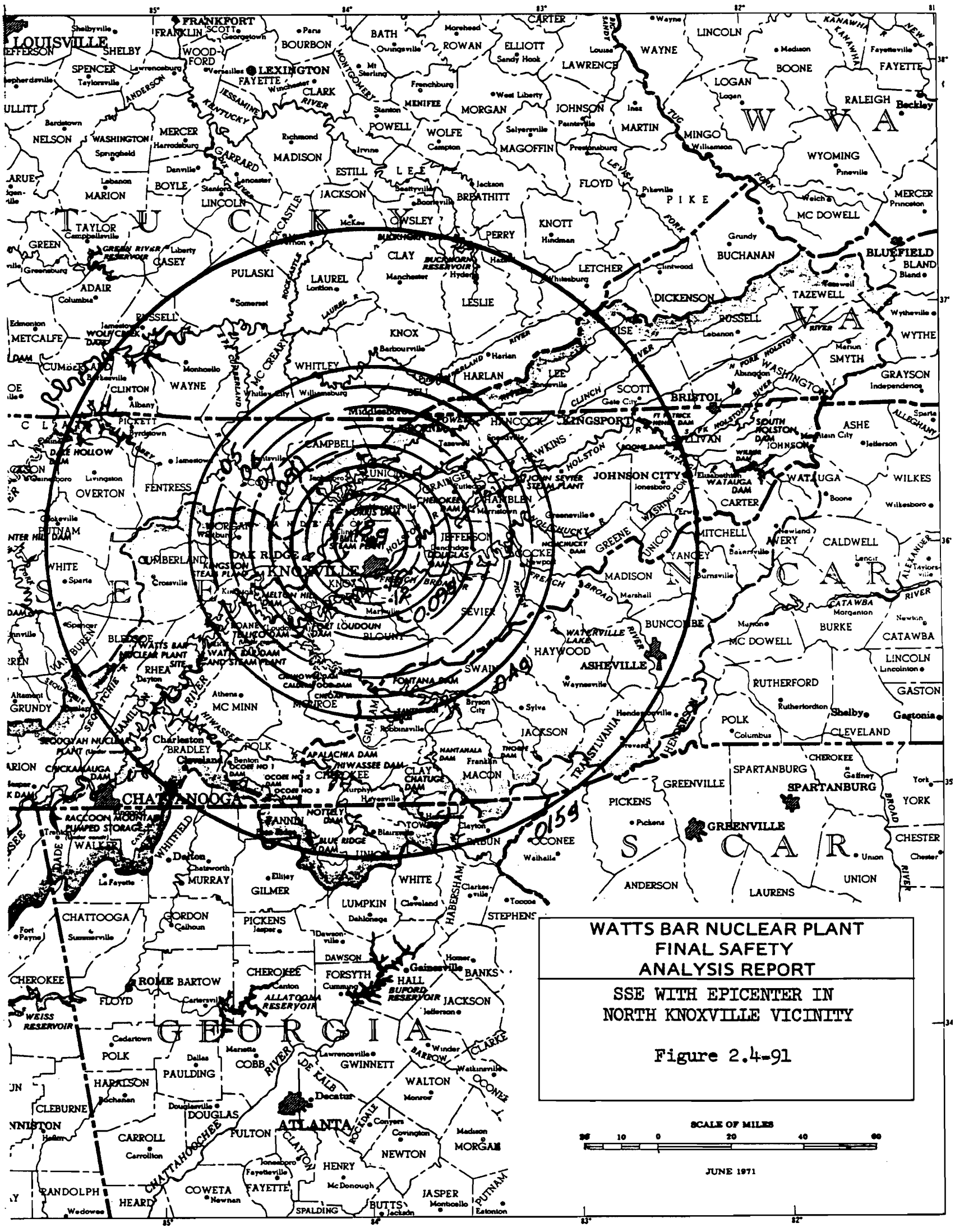
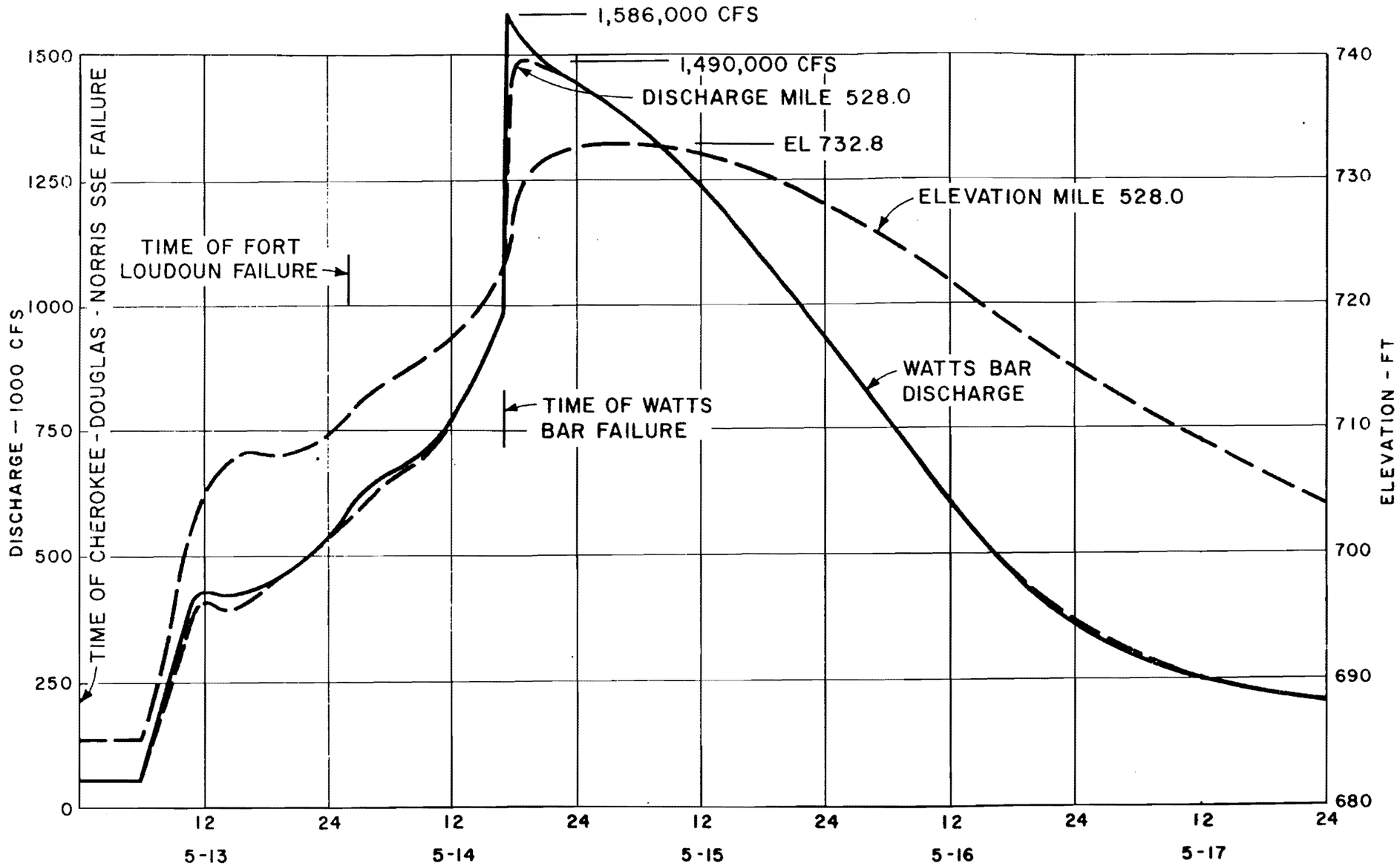


Figure 2.4-91 SSE With Epicenter In North Knoxville Vicinity



FLOOD HYDROGRAPHS

Figure 2.4-92

Revised by
Amendment 35

Figure 2.4-92 Time and Date Flood Hydrographs

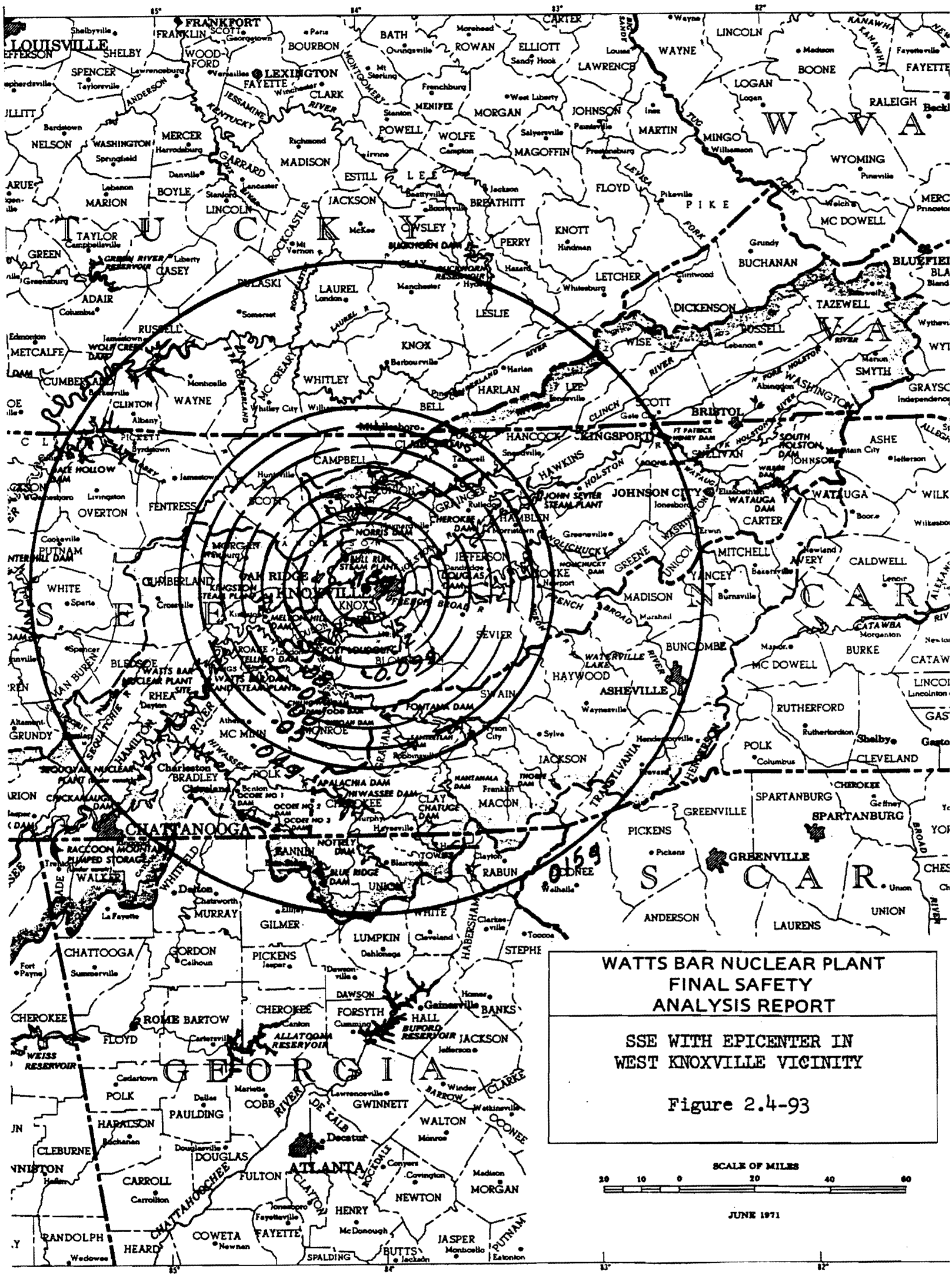
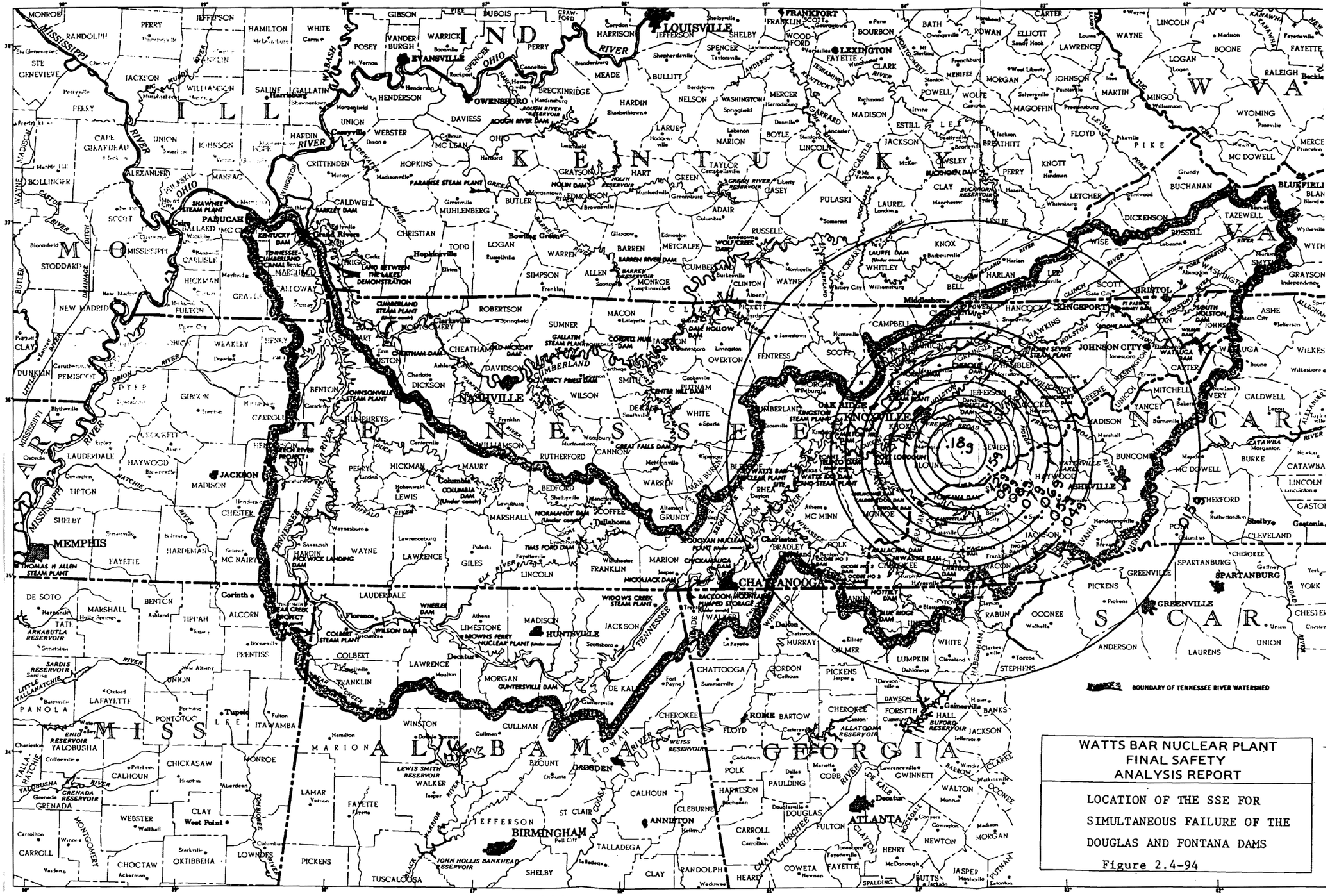


Figure 2.4-93 SSE With Epicenter In West Knoxville Vicinity



WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

LOCATION OF THE SSE FOR
SIMULTANEOUS FAILURE OF THE
DOUGLAS AND FONTANA DAMS
Figure 2.4-94

Figure 2.4-94 Location of SSE For Simultaneous Failure of The Douglas and Fontana Dams

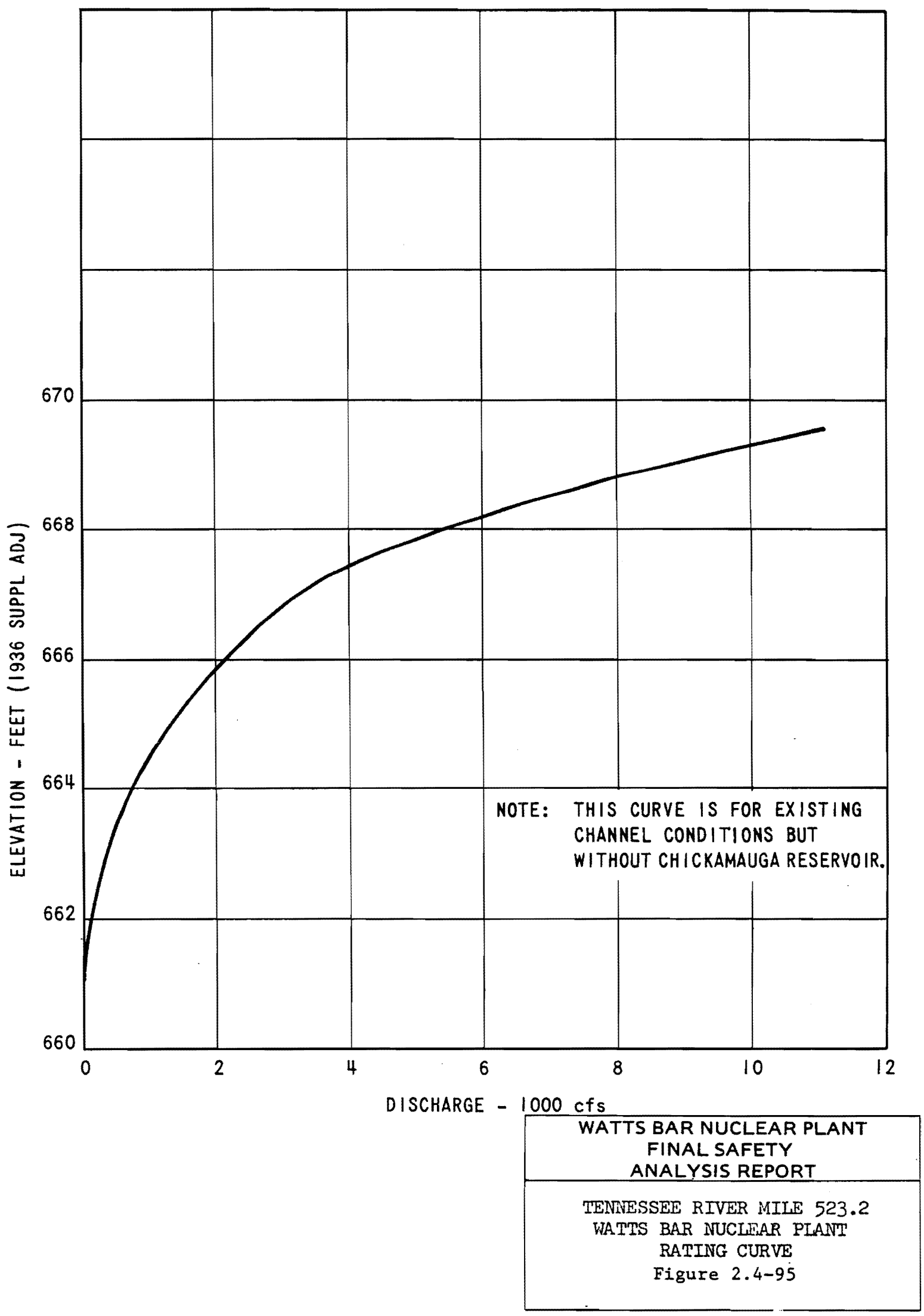


Figure 2.4-95 Tennessee River Mile 523.2 Watts Bar Nuclear Plant Rating Curve

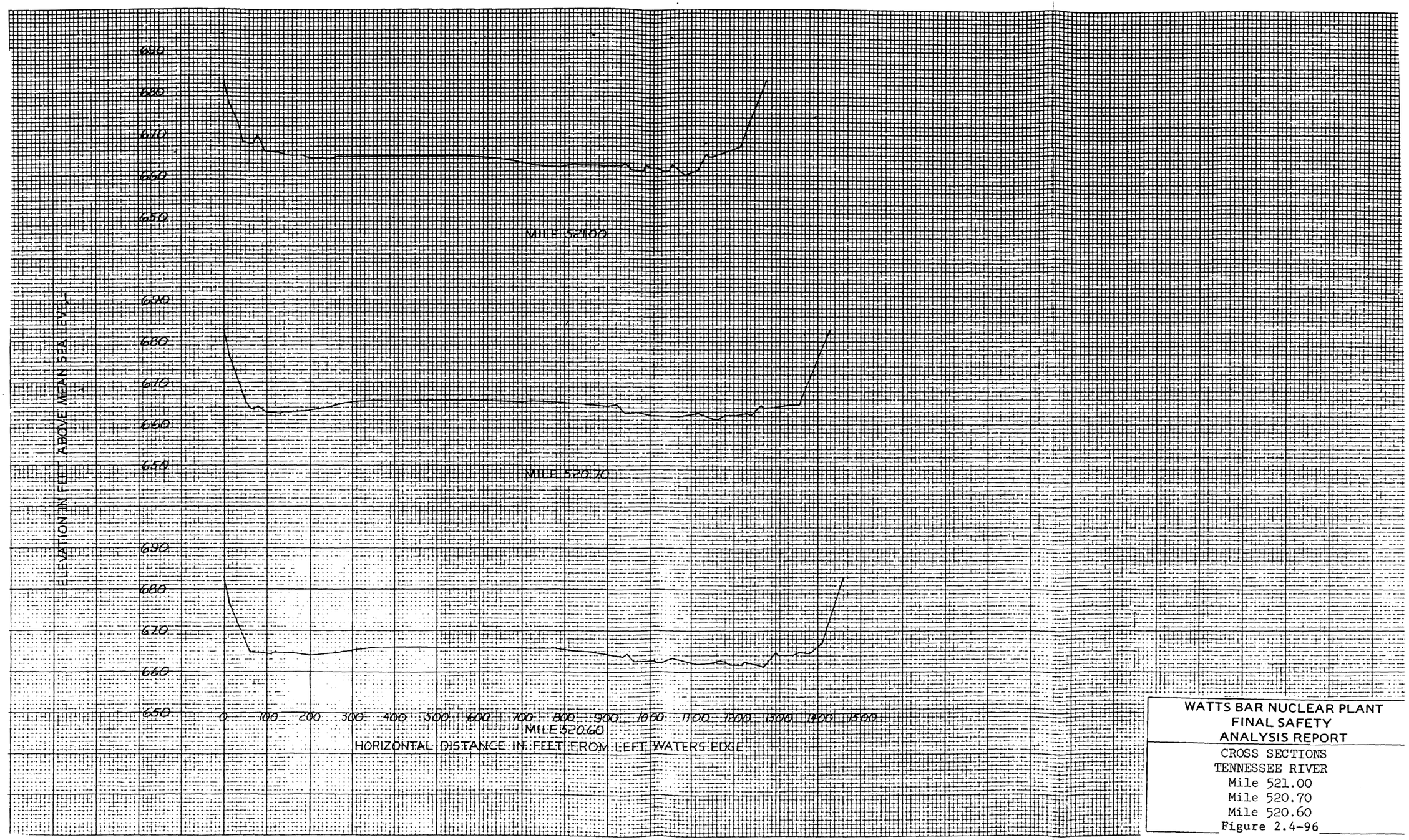


Figure 2.4-96 Cross Sections Tennessee River (mile 521.00) (mile 520.70) (mile 520.60)

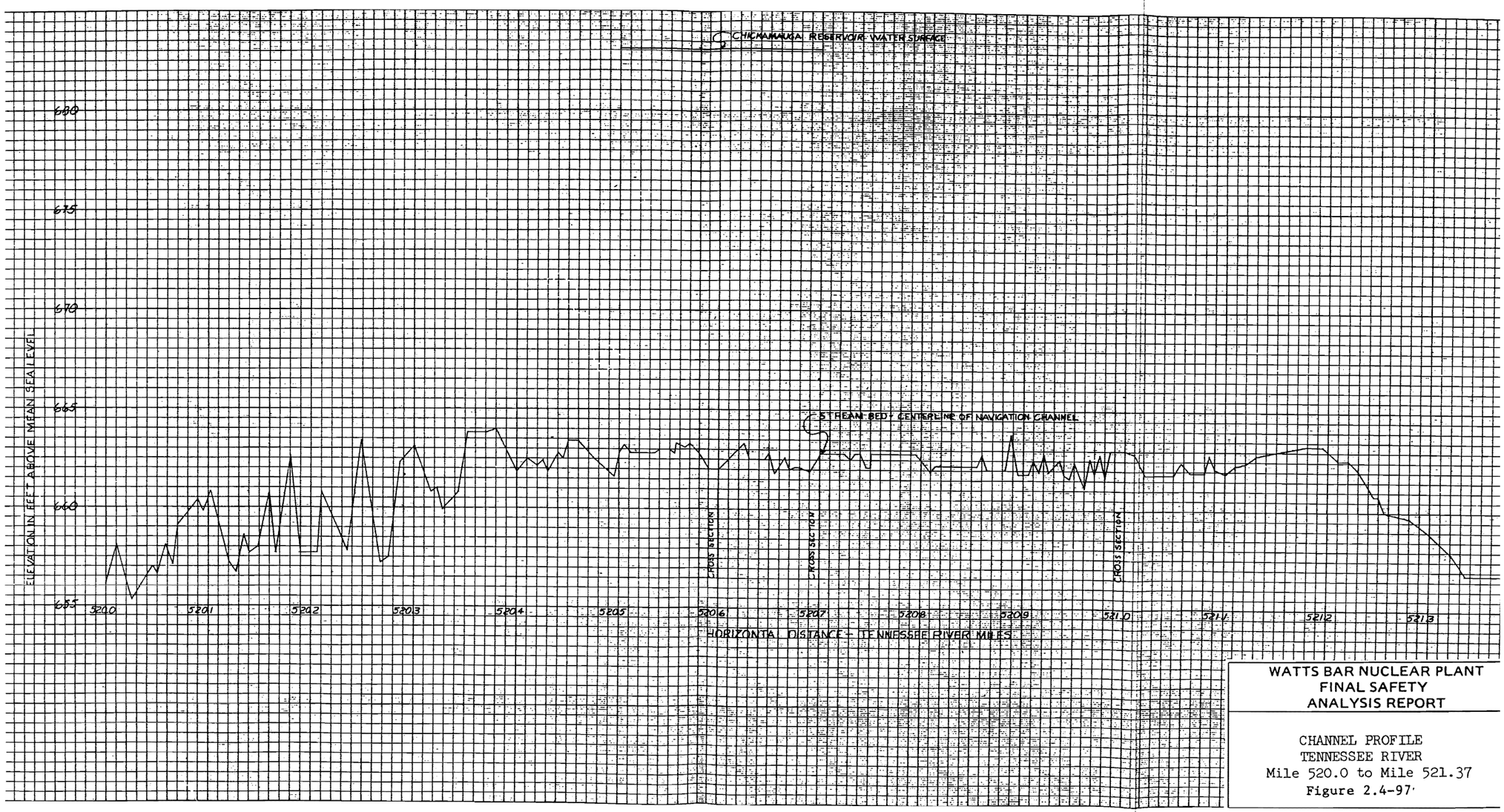
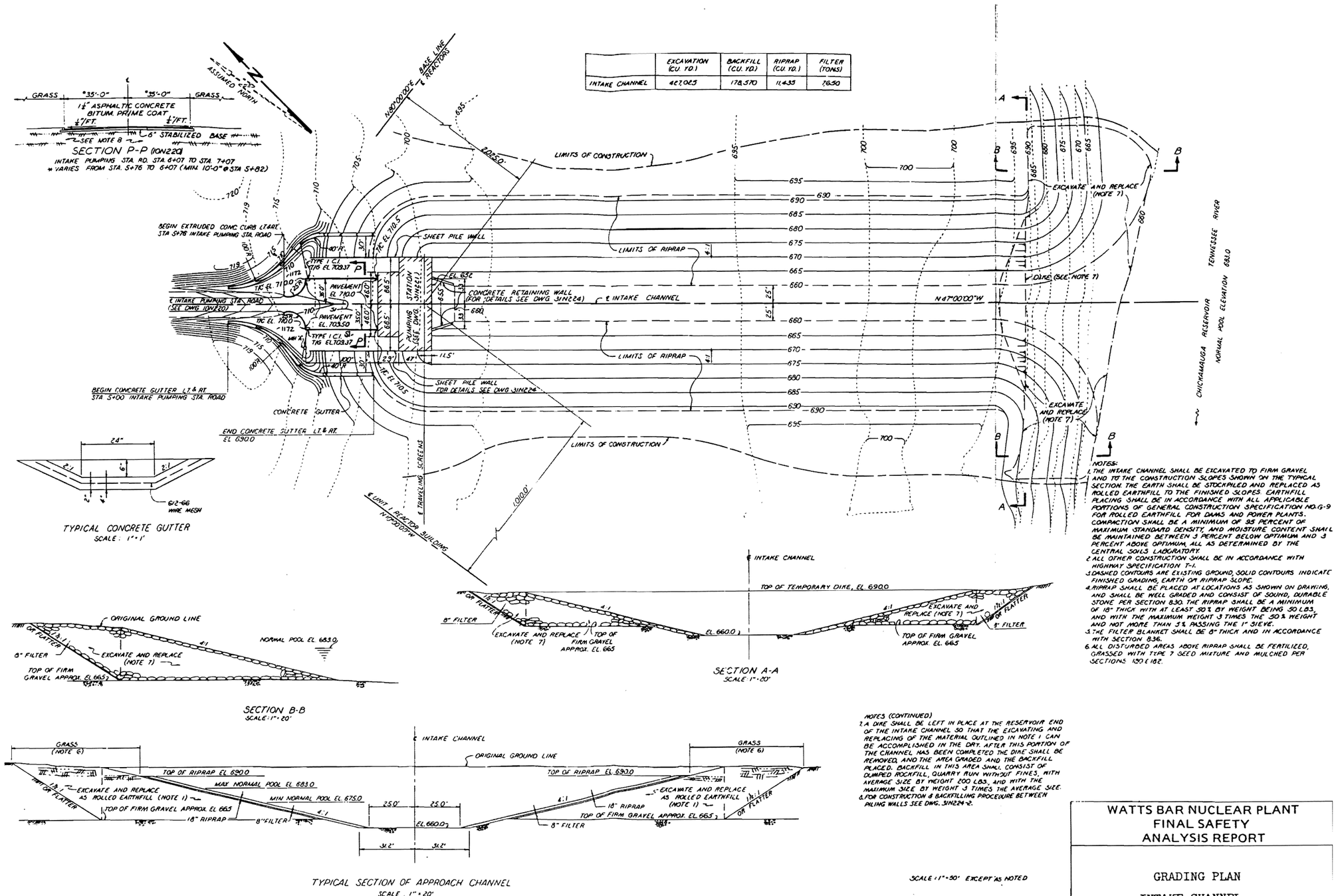


Figure 2.4-97 Channel Profile Tennessee River (mile 520.0 to mile 521.37)

Figure 2.4-98 Main Plant General Grading Plan

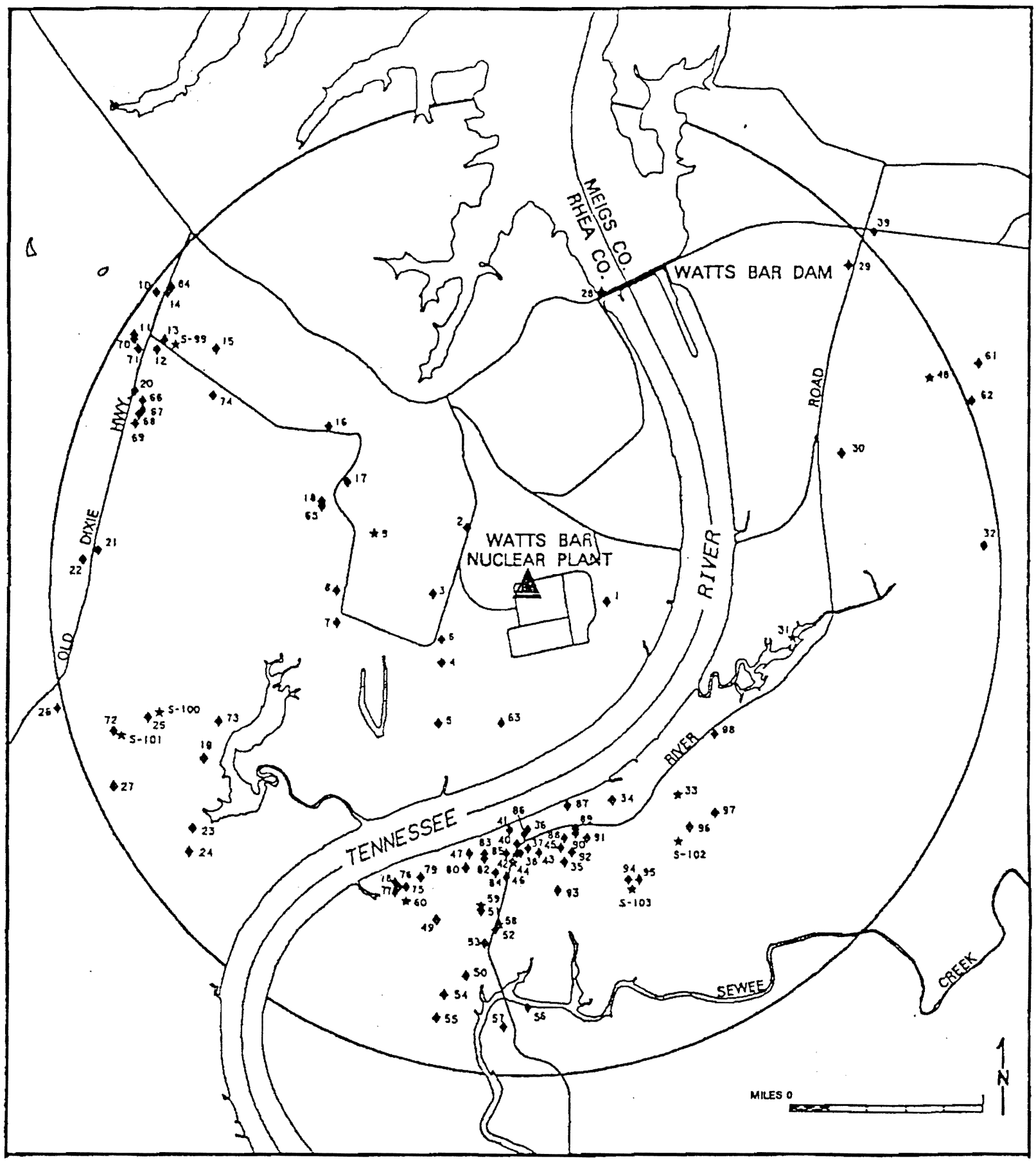


	EXCAVATION (CU. YD.)	BACKFILL (CU. YD.)	RIPRAP (CU. YD.)	FILTER (TONS)
INTAKE CHANNEL	427,063	178,570	11,435	7,650

Figure 2.4-99 Grading Plan Intake Channel

Figure 2.4-100 Deleted by Amendment 83

Figure 2.4-101 Deleted by Amendment 33



LEGEND

- ♦ WELL
- ★ SPRING
- — ROADS
- ○ 2 MILE RADIUS OF PLANT SITE

AMENDMENT 83

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

WELL AND SPRING INVENTORY
WITHIN 2 MILE RADIUS OF WATTS BAR
NUCLEAR PLANT SITE
FSAR FIG 2.4-102

Figure 2.4-102 Wells And Spring Inventory Within 2-Mile Radius of Watts Bar Nuclear Plant Site

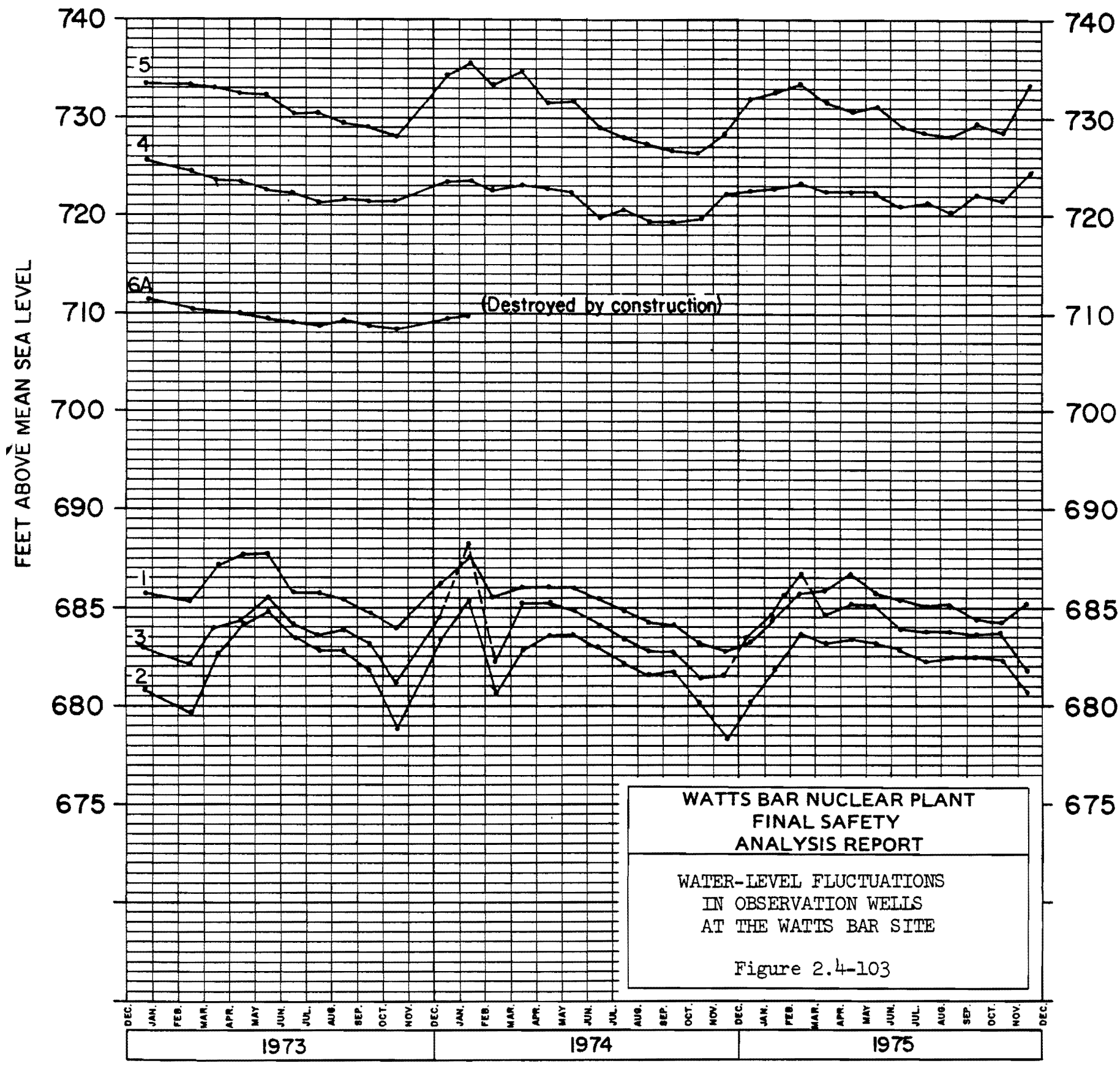
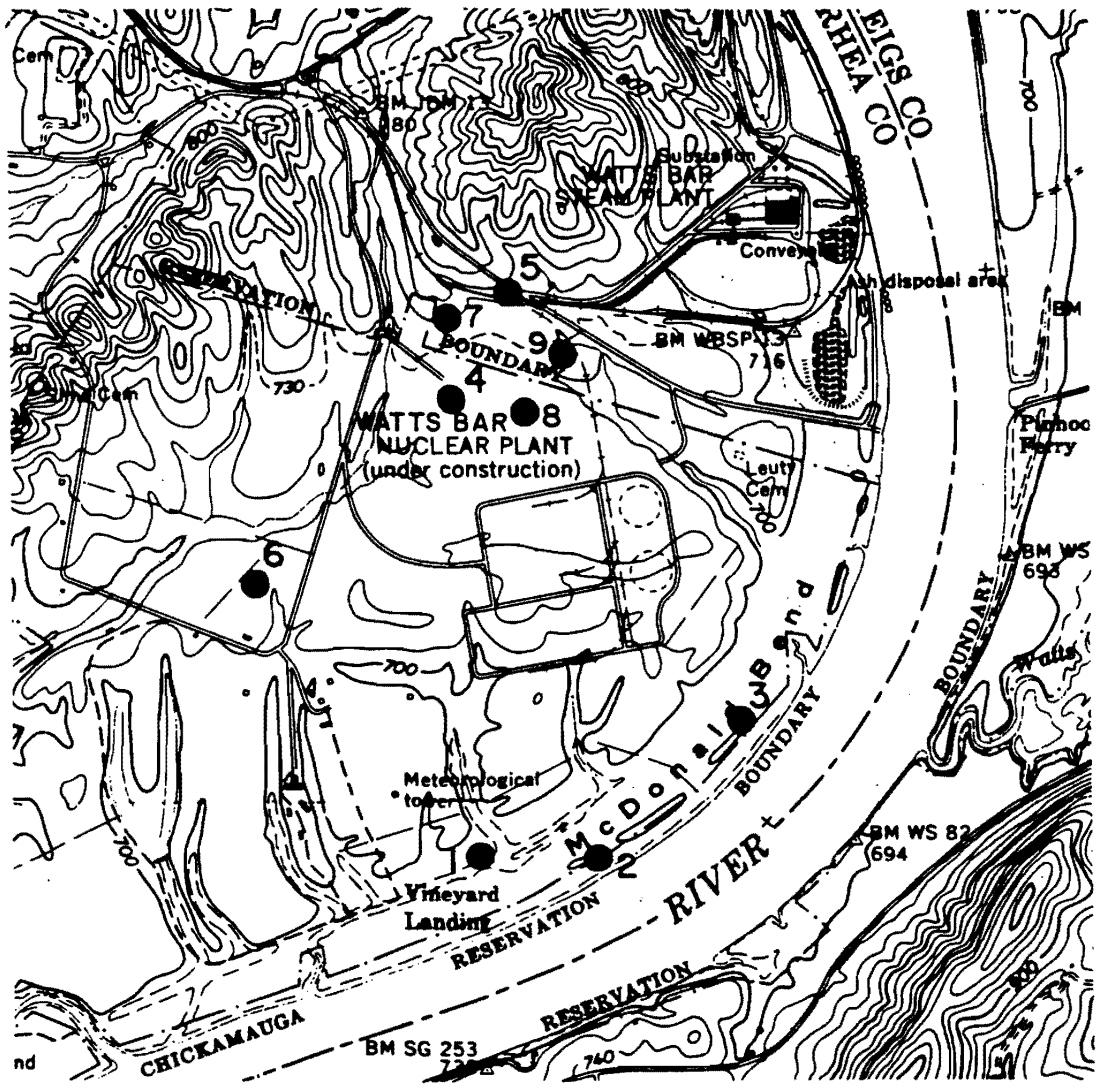


Figure 2.4-103 Water-Level Fluctuations In Observation Wells at The Watts Bar Site



NOTE:

Topographic base from U.S.G.S - T.V.A. 7.5 minute quadrangle, Decatur, Tenn., 118-SE, Contour interval 20 feet.

LEGEND:

●2 — Ground-water observation well showing number.

SCALE:

1000 0 1000 2000 Feet

Revised by Amendment 50

WATTS BAR NUCLEAR PLANT
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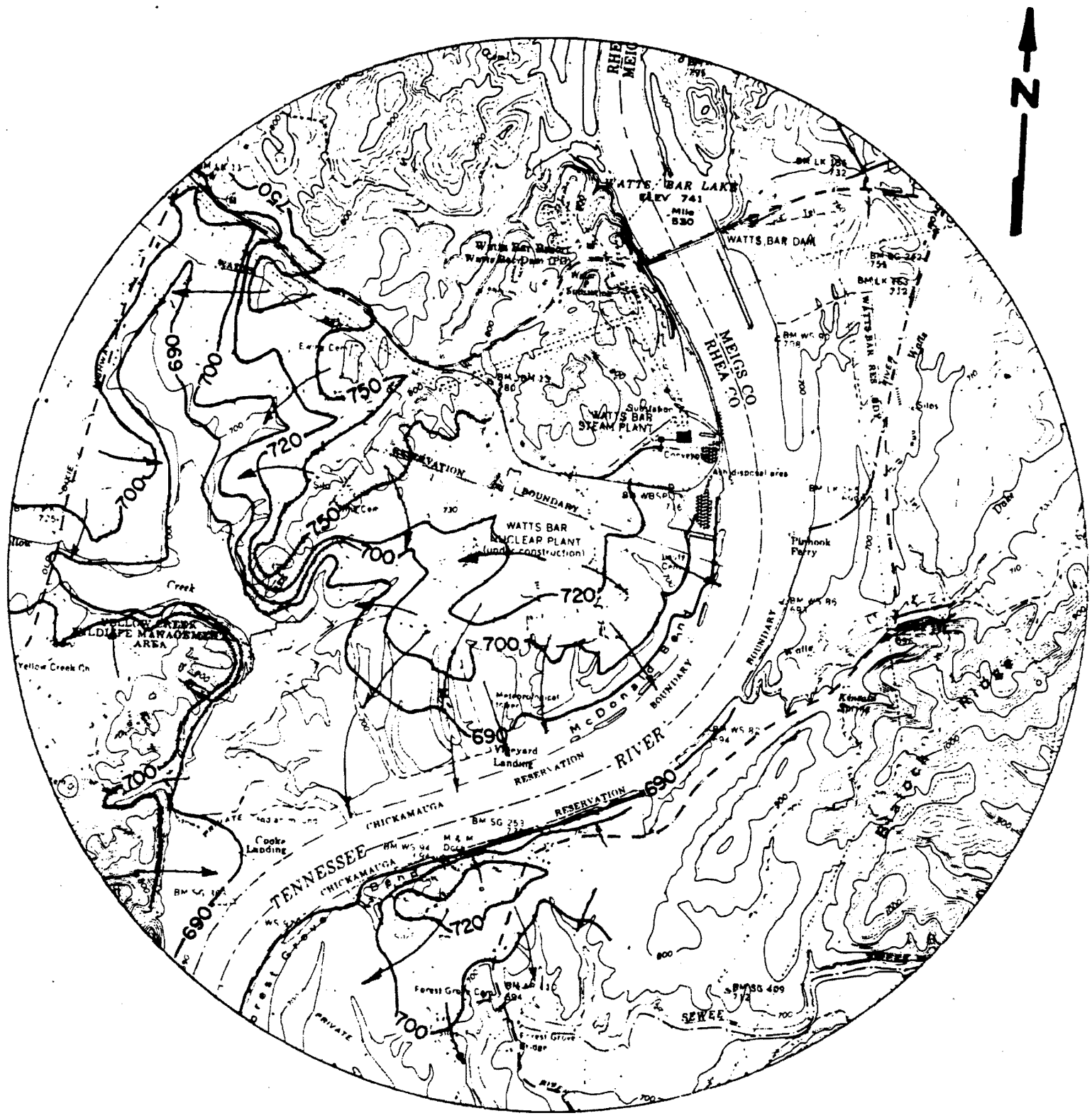
ANALYSIS REPORT

LOCATIONS OF
GROUND - WATER
OBSERVATION WELLS
FIGURE 2.4-104

Figure 2.4-104 Locations of Ground - Water Observation Wells



EXPLANATION:

700 — Water table contour, in feet above mean sea level.

— General direction of ground-water movement.

SCALE:

2000 0 4000 Feet

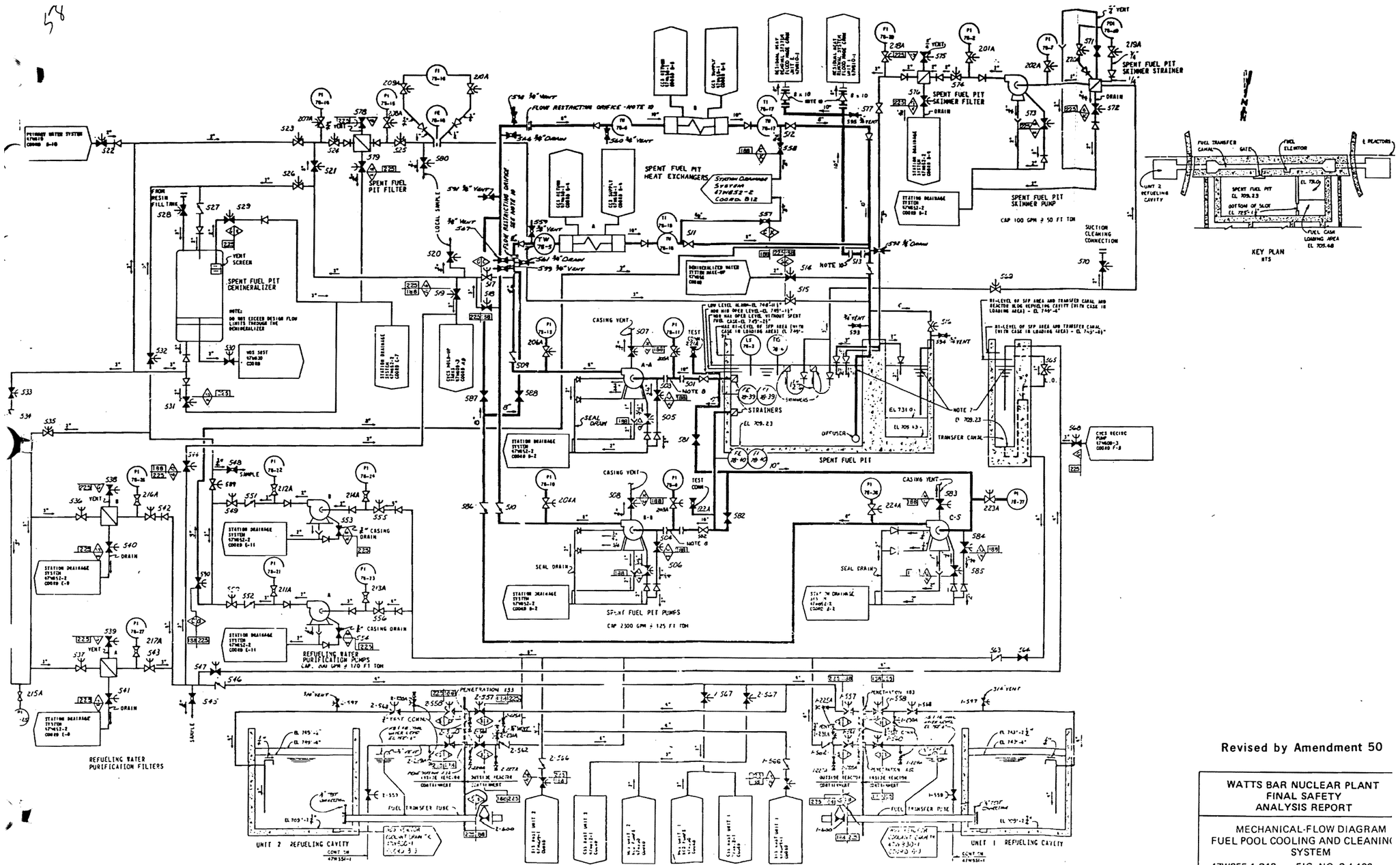
Revised by Amendment 50

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

GENERALIZED WATER-TABLE
CONTOUR MAP

Figure 2.4-105

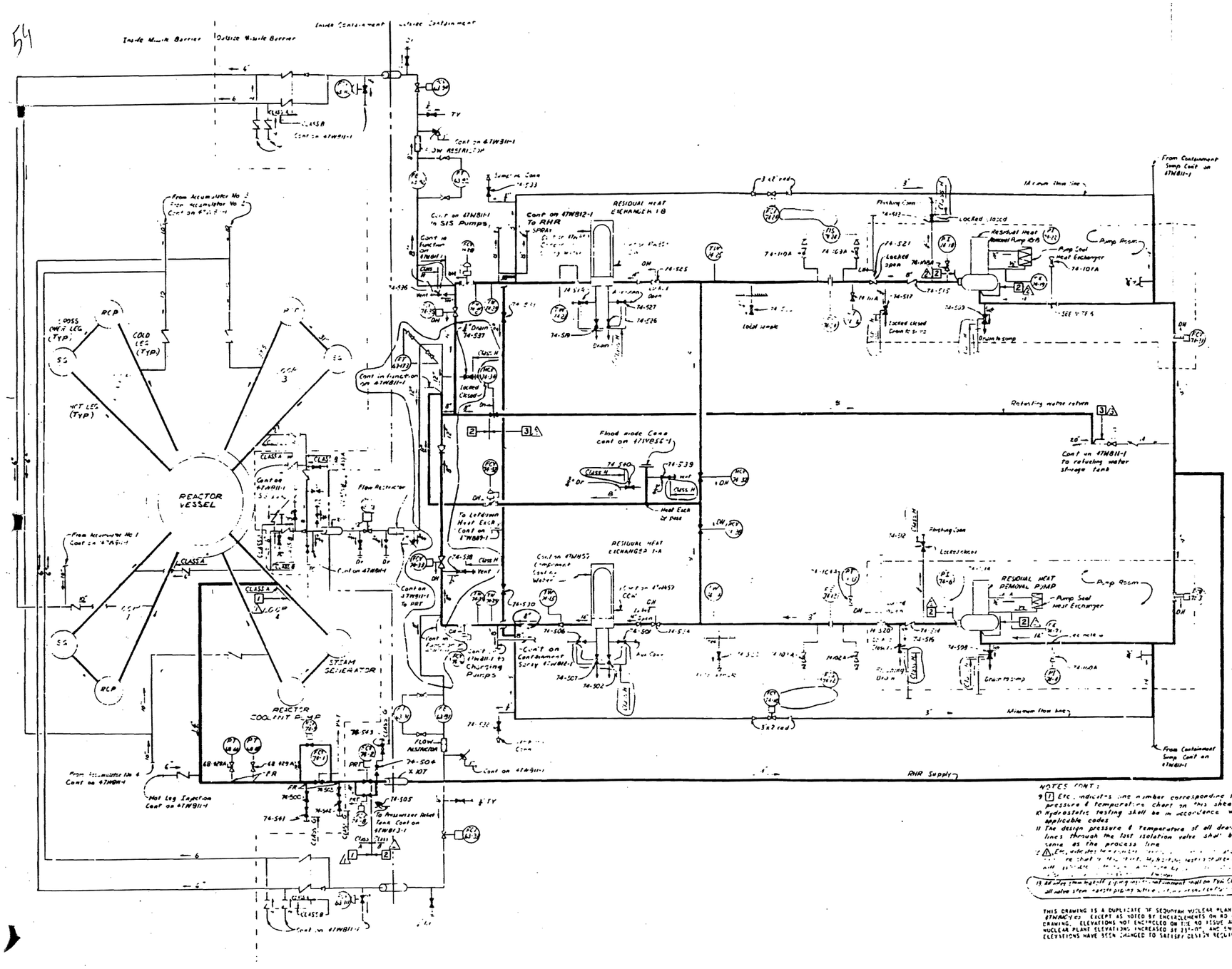
Figure 2.4-105 Generalized Water-Table Contour Map January 1972



Revised by Amendment 50

WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT	
MECHANICAL-FLOW DIAGRAM FUEL POOL COOLING AND CLEANING SYSTEM	
47W855-1 R13	FIG. NO. 2.4-106

Figure 2.4-106 Mechanical - Flow Diagram Fuel Pool Cooling and Cleaning System



DESIGN PRESSURE & TEMPERATURE DATA

Line No	Design Pressure (PSIG)	Design Temperature (°F)
1	2485	650
2	600	400
3	100	100

NOTES CONT:

9 [1] Etc. indicates line number corresponding to design pressure & temperature chart on this sheet.

10 Hydrostatic testing shall be in accordance with the applicable codes.

11 The design pressure & temperature of all drain & vent lines through the last isolation valve shall be the same as the process line.

REFERENCE DRAWINGS

47B601-74-SERIES-INSTRUMENT TABULATION

47W611-74 1-2 ---- LOGIC DIAGRAM

47B21-1 ------- PIPING SYSTEM CLASSIFICATION

47W432 SERIES - RHR SYSTEM PIPING

WESTINGHOUSE Specifications SO-WAT/WBT RESIDUAL HEAT REMOVAL SYSTEM.

Revised by Amendment

WATTS BAR NUCLEAR PLA
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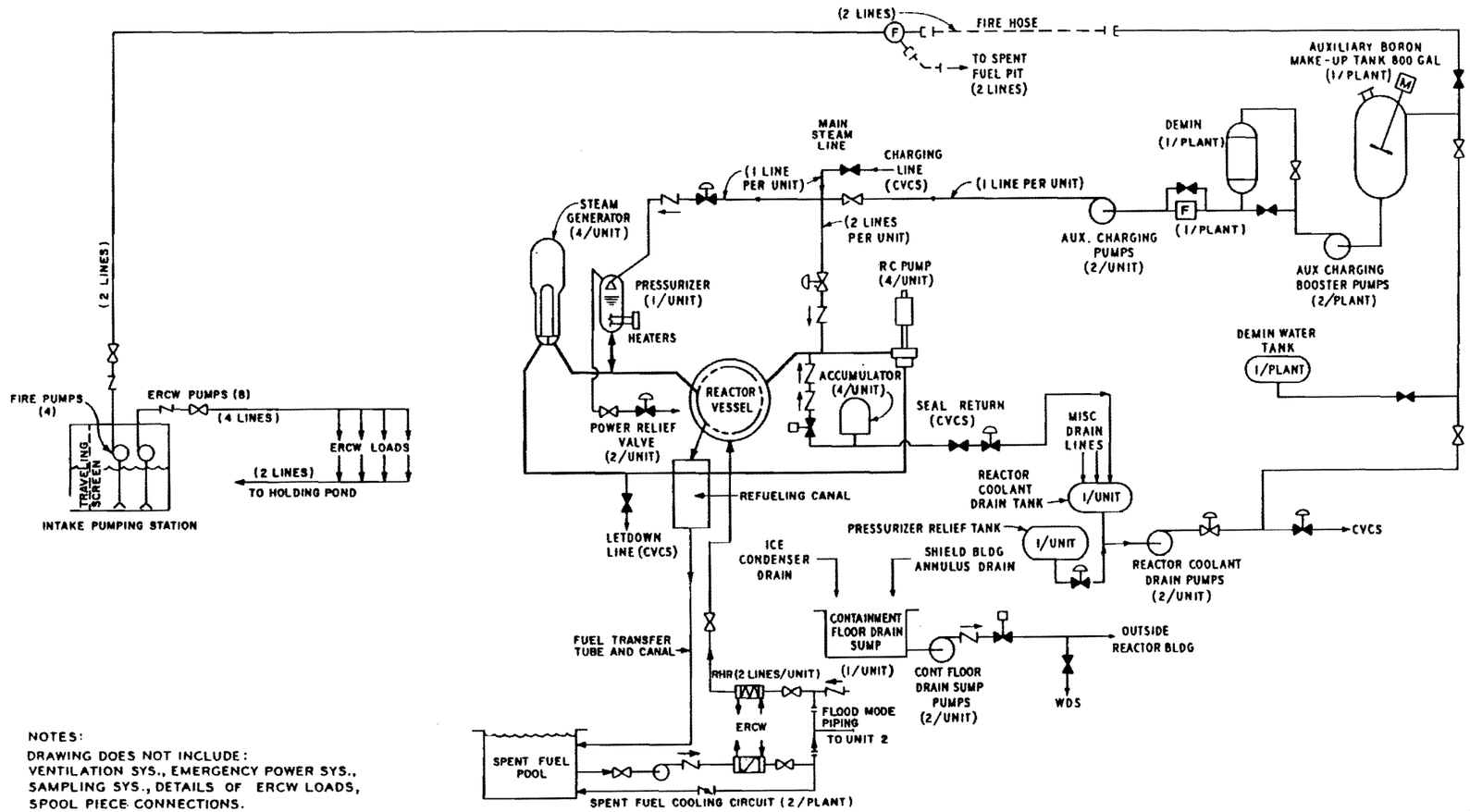
POWERHOUSE UNITS 1 & 2
FLOW DIAGRAM
RESIDUAL HEAT REMOVAL
SYSTEM
TVA DWG NO. 47W810-1 R16
FIGURE 2.4-107

Figure 2.4-107 Powerhouse Units 1 & 2 Flow Diagram - Residual Heat Removal System



NOTES:
DRAWING DOES NOT INCLUDE:
VENTILATION SYS., EMERGENCY POWER SYS., SAMPLING SYS., DETAILS OF ERCW LOADS, SPOOL PIECE CONNECTIONS.

WATTS BAR NUCLEAR PLANT
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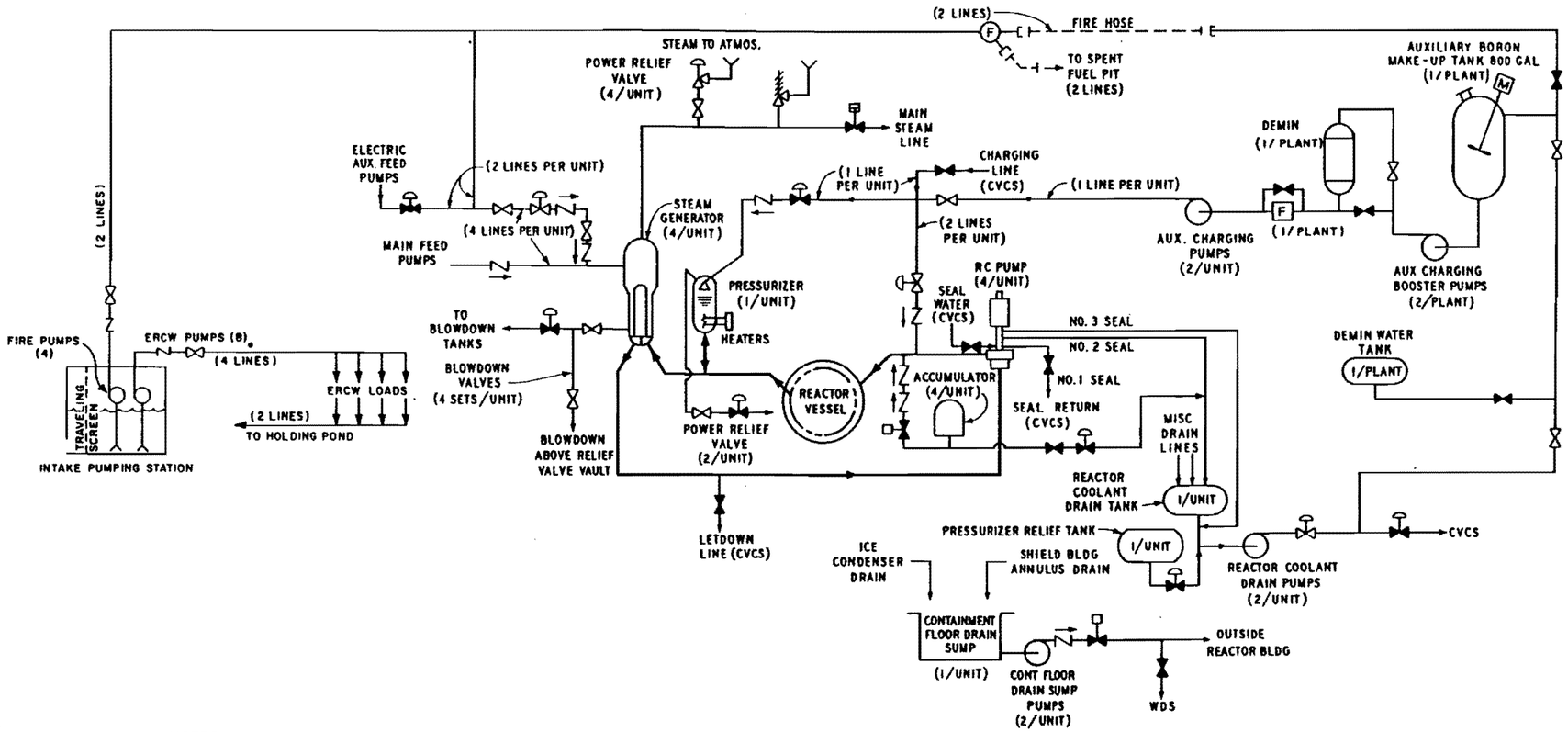
SCHEMATIC FLOW DIAGRAM
FLOOD PROTECTION PROVISIONS
OPEN REACTOR COOLING
(unit 1 shown, unit 2 similar)
Figure 2.4-108

Figure 2.4-108 Schematic Flow Diagram Flood Protection Provisions Open Reactor Cooling (Unit 1 Shown, Unit 2 Similar)

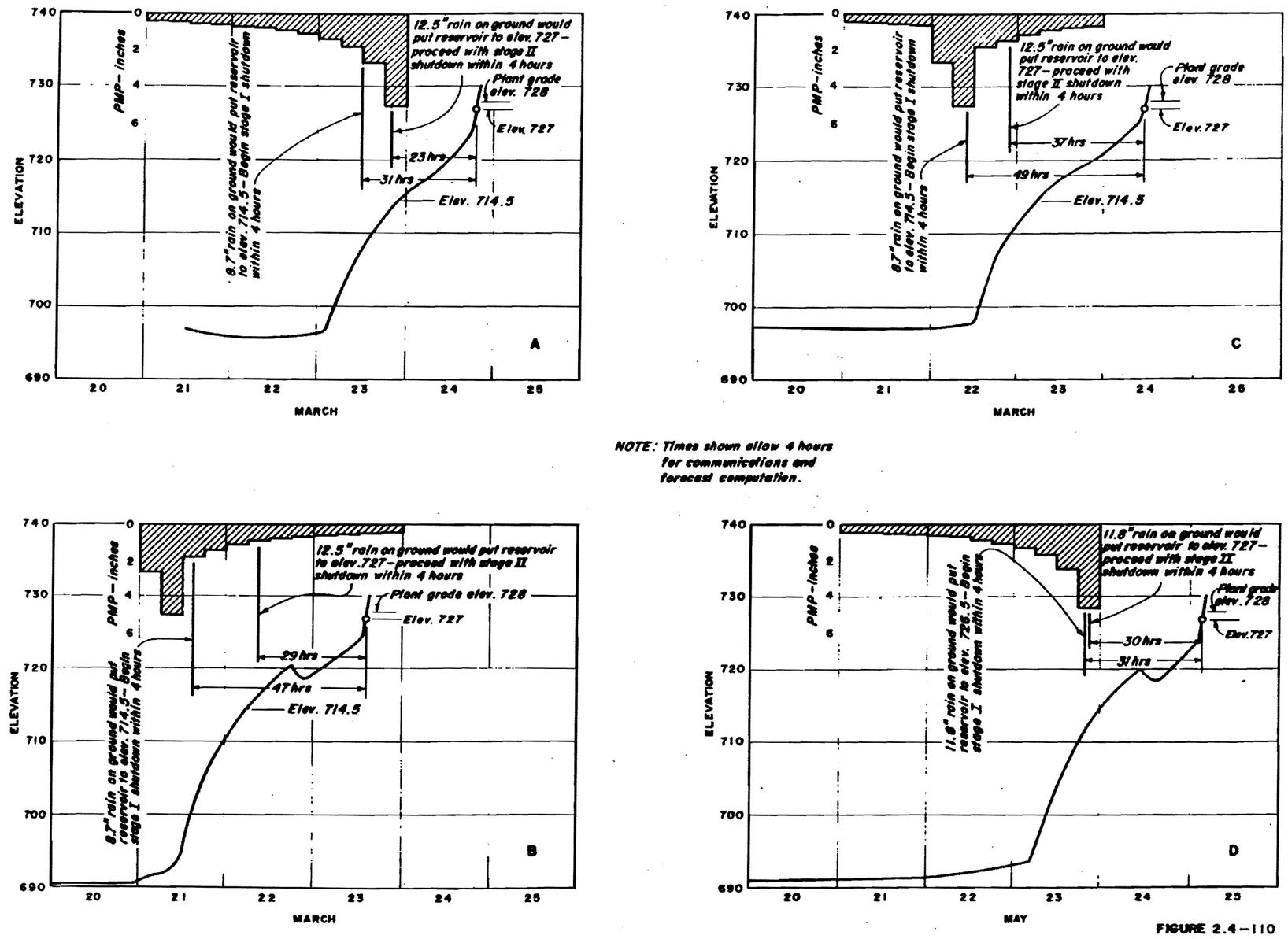


NOTES:
DRAWING DOES NOT INCLUDE:
VENTILATION SYS., EMERGENCY POWER SYS., SAMPLING SYS., DETAILS OF ERCW LOADS, SPOOL PIECE CONNECTIONS.

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

SCHEMATIC FLOW DIAGRAM
FLOOD PROTECTION PROVISIONS
NATURAL CONVECTION COOLING
(unit 1 shown, unit 2 similar)
Figure 2.4-109

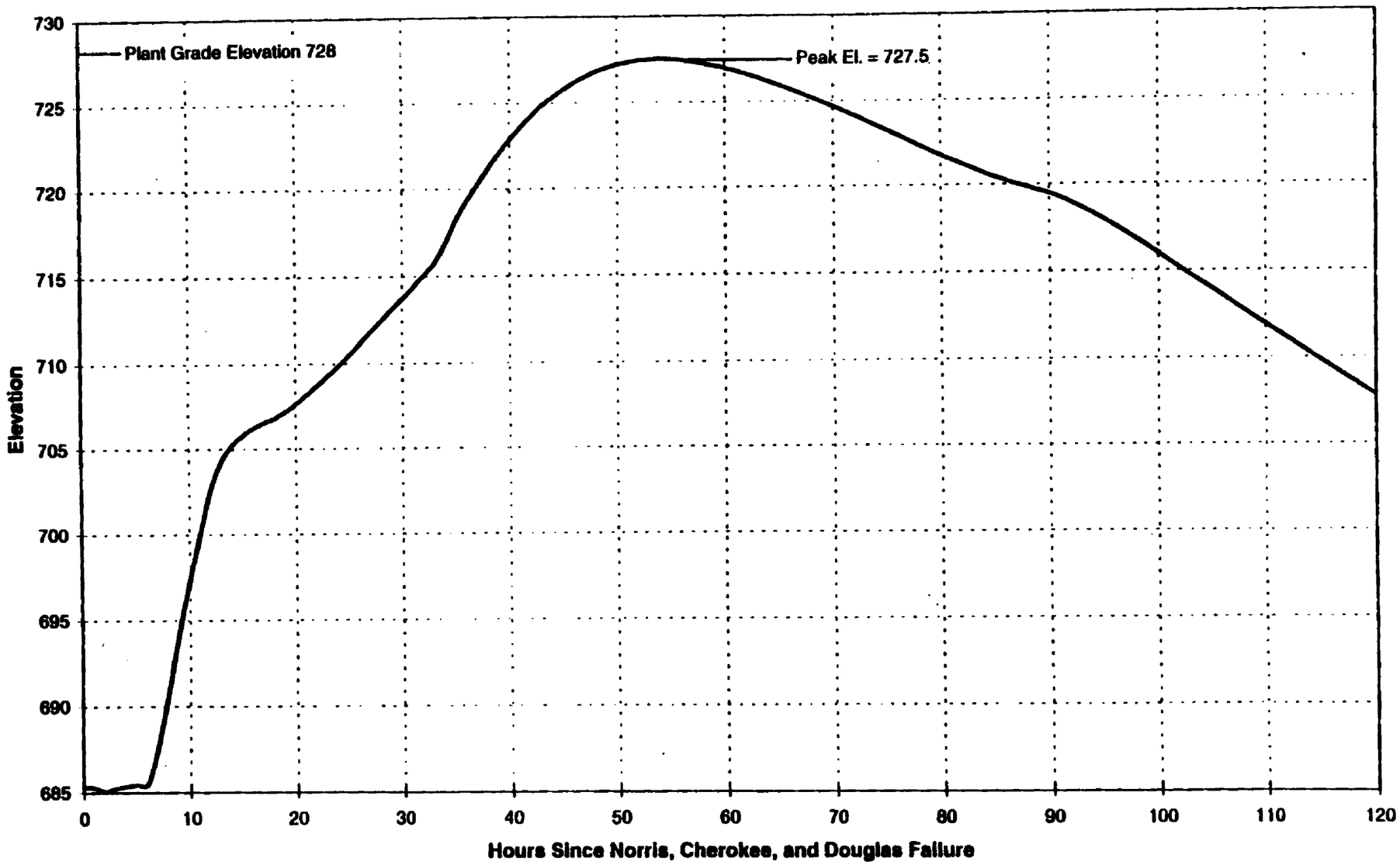
Figure 2.4-109 Schematic Flow Diagram Flood Protection Provisions Natural Convection Cooling (Unit 1 Shown, Unit 2 Similar)



WATTS BAR NUCLEAR PLANT RAINFALL FLOOD PROTECTION PLAN
BASIS FOR SAFE SHUTDOWN FOR PLANT FLOODING

Added by Amendment 32

Figure 2.4-110 Watts Bar Nuclear Plant Rainfall Flood Protection Plan Basis For Safe Shutdown For Plant Flooding



Seismic Flood Analysis - Norris, Cherokee and Douglas SSE with 25-year Flood
Watts Bar Plant
Figure 2.4 - 111

FSAR - Amendment 92

Figure 2.4-111 Douglas PMF Failure Wave at Watts Bar Plant