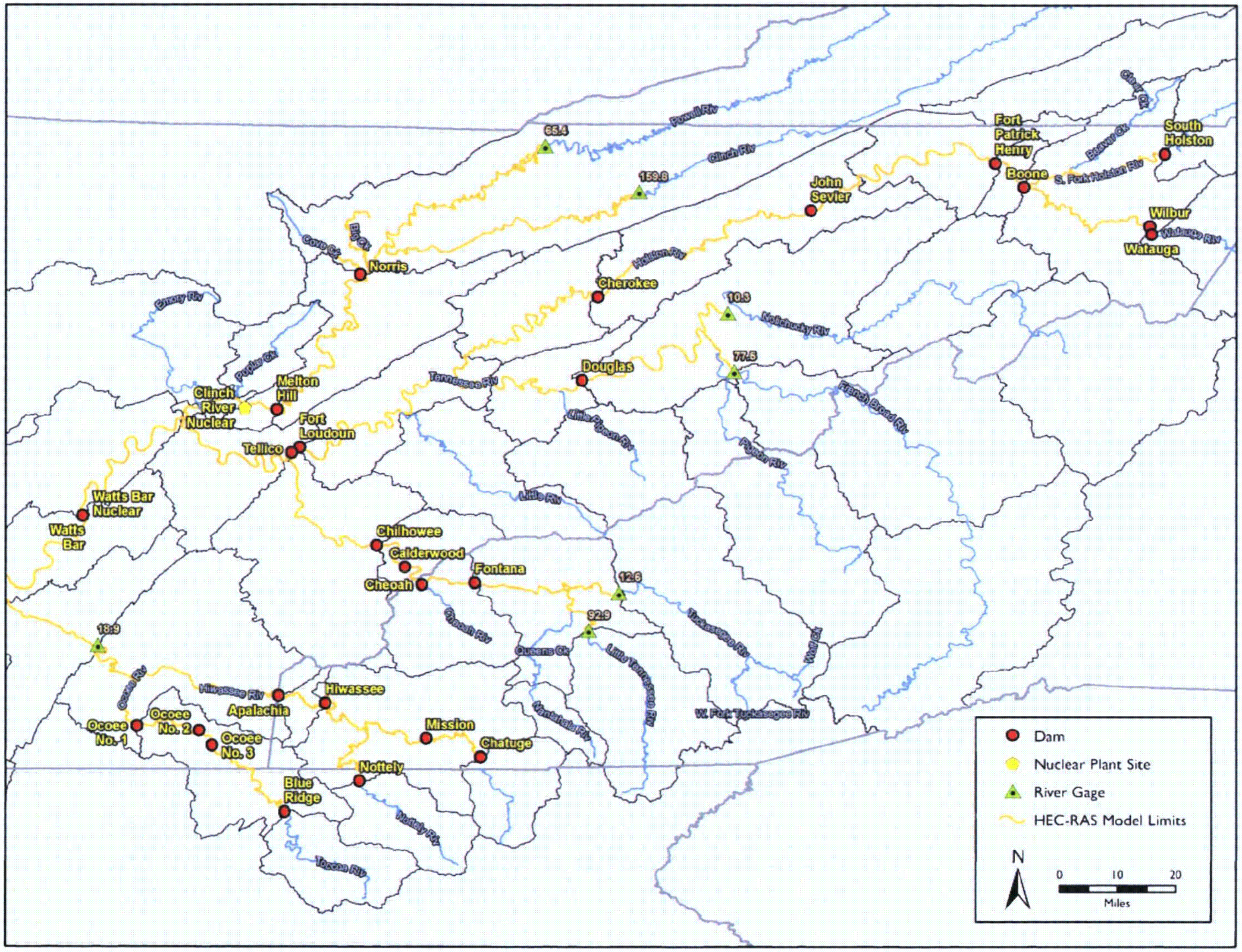


Figure 2.4-1a Extent of HEC-RAS Modeling (Sheet 1 of 2)

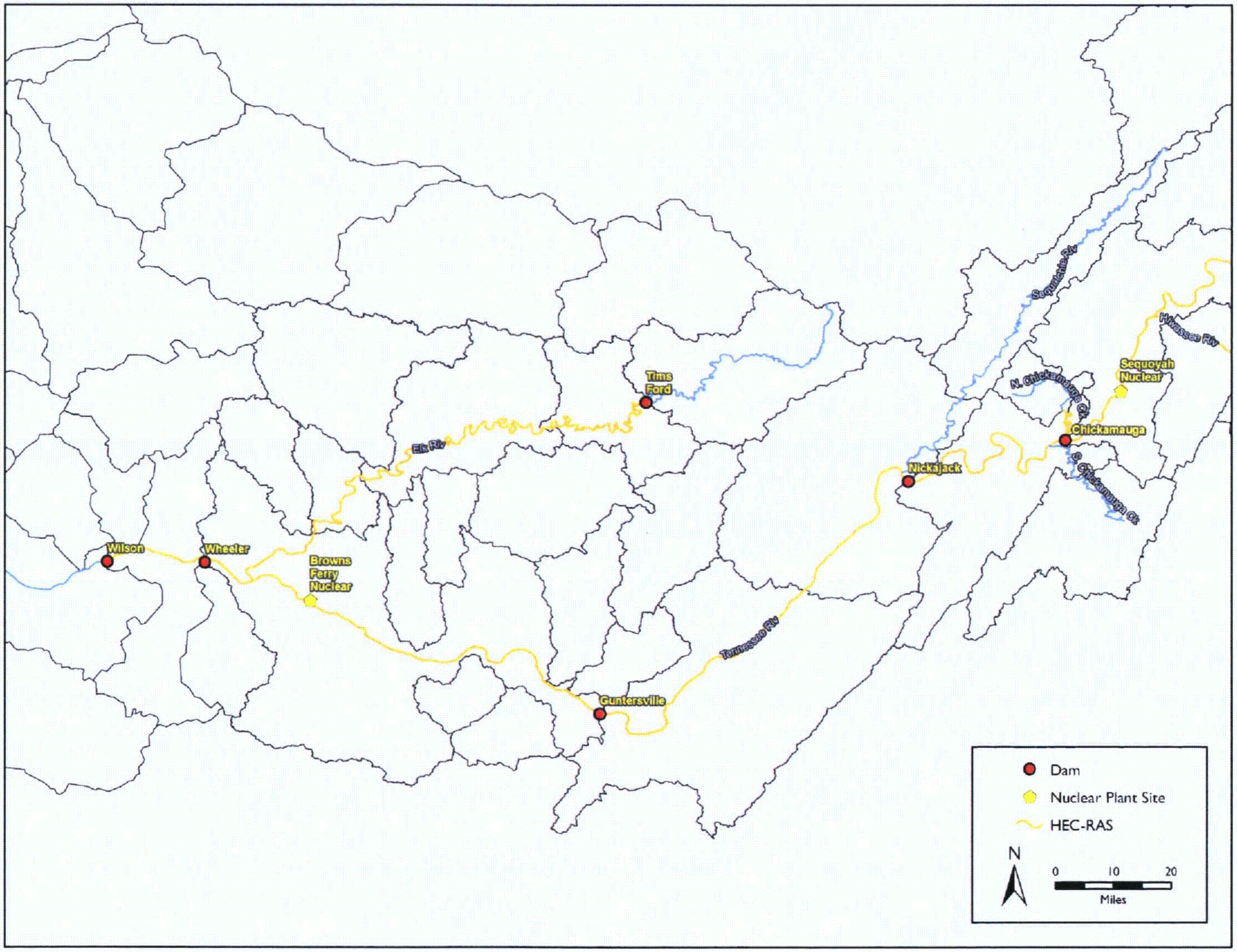
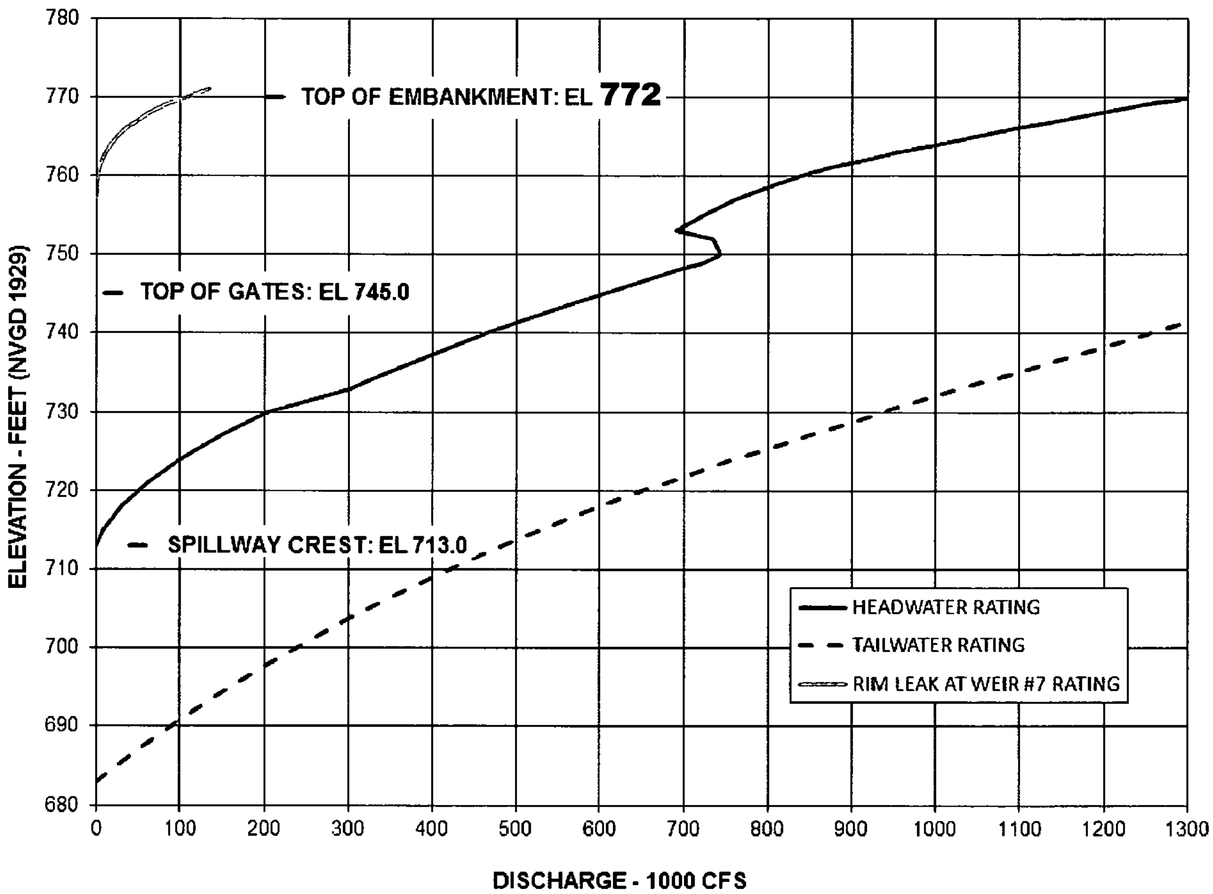
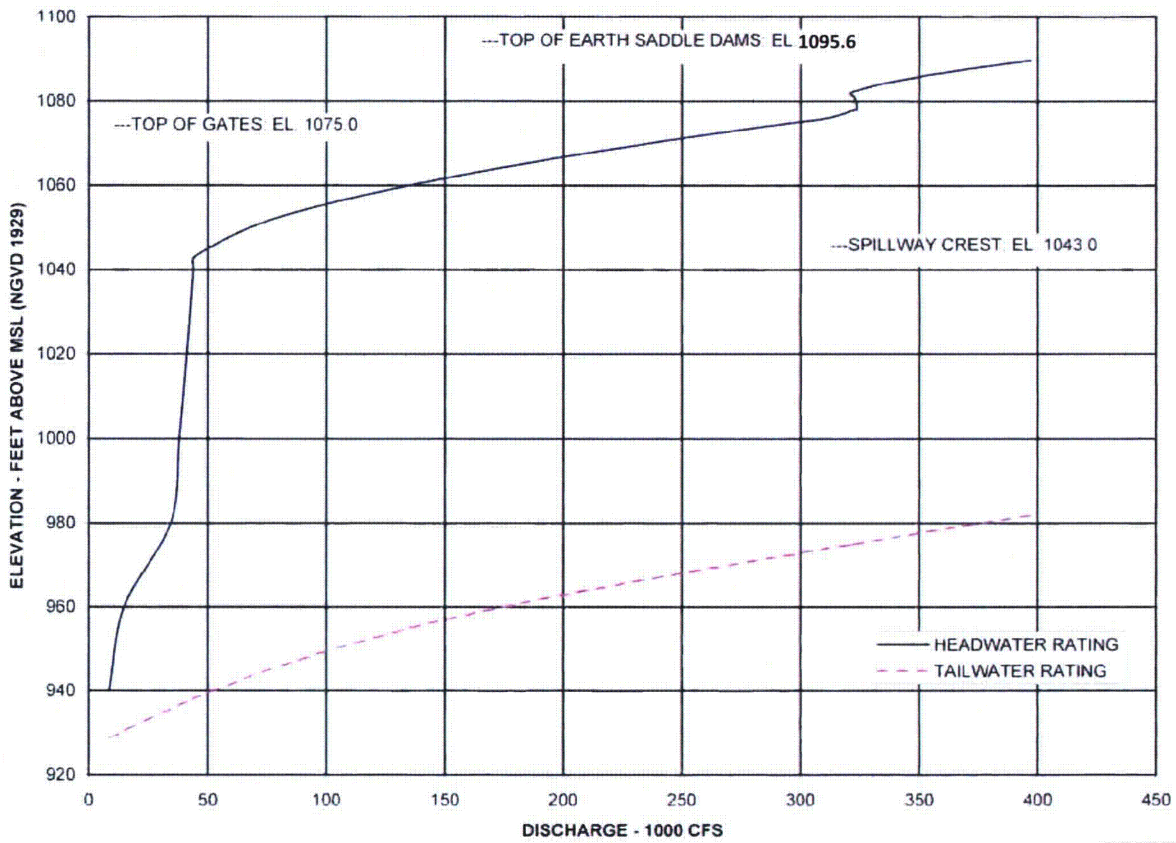


Figure 2.4-1a Extent of HEC-RAS Modeling (Sheet 2 of 2)



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

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



WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

Discharge Rating Curve, Cherokee Dam

Figure 2.4-11 (Sheet 6 of 13)

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

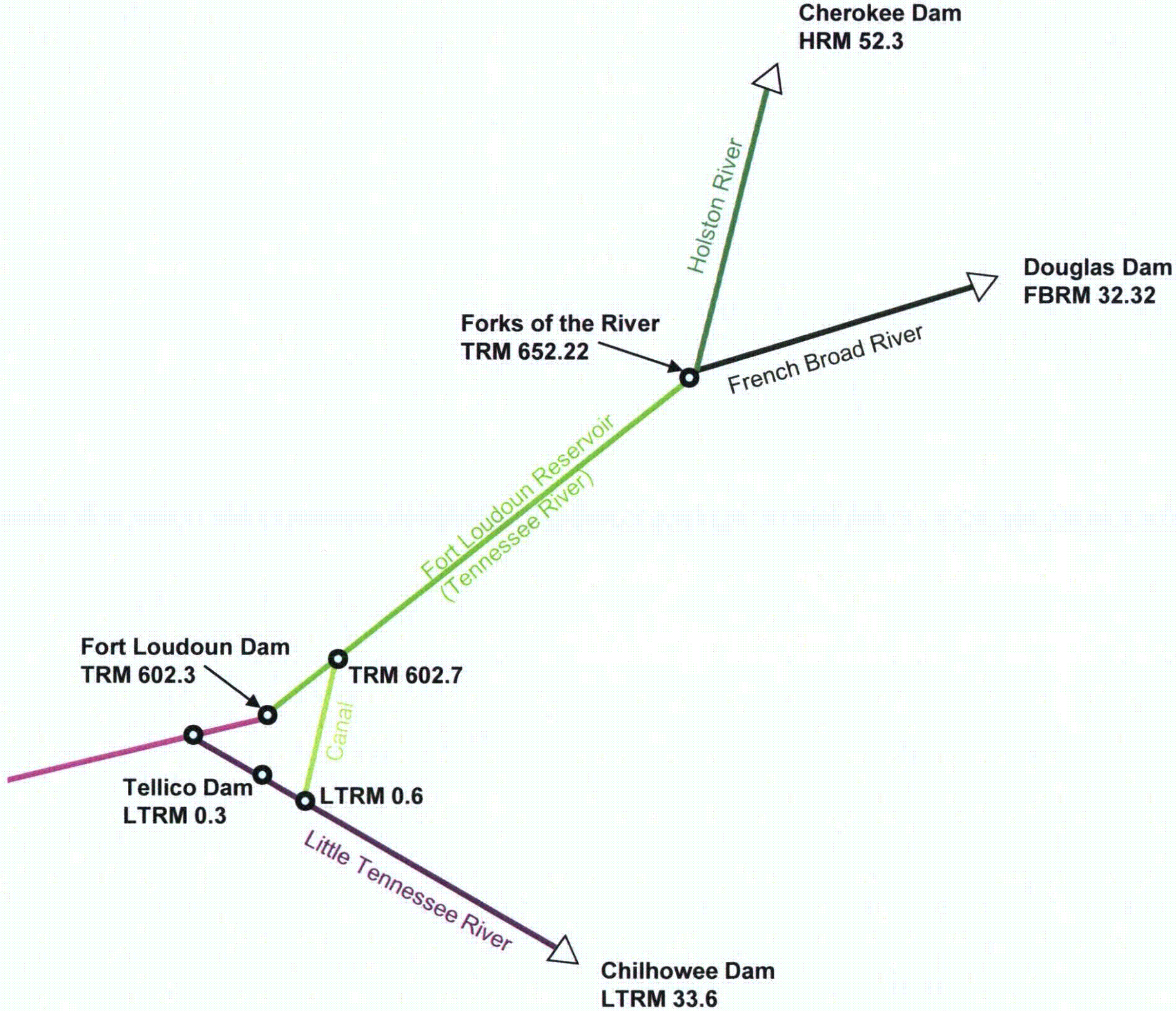


Figure 2.4-12 Fort Loudoun & Tellico HEC-RAS Schematic

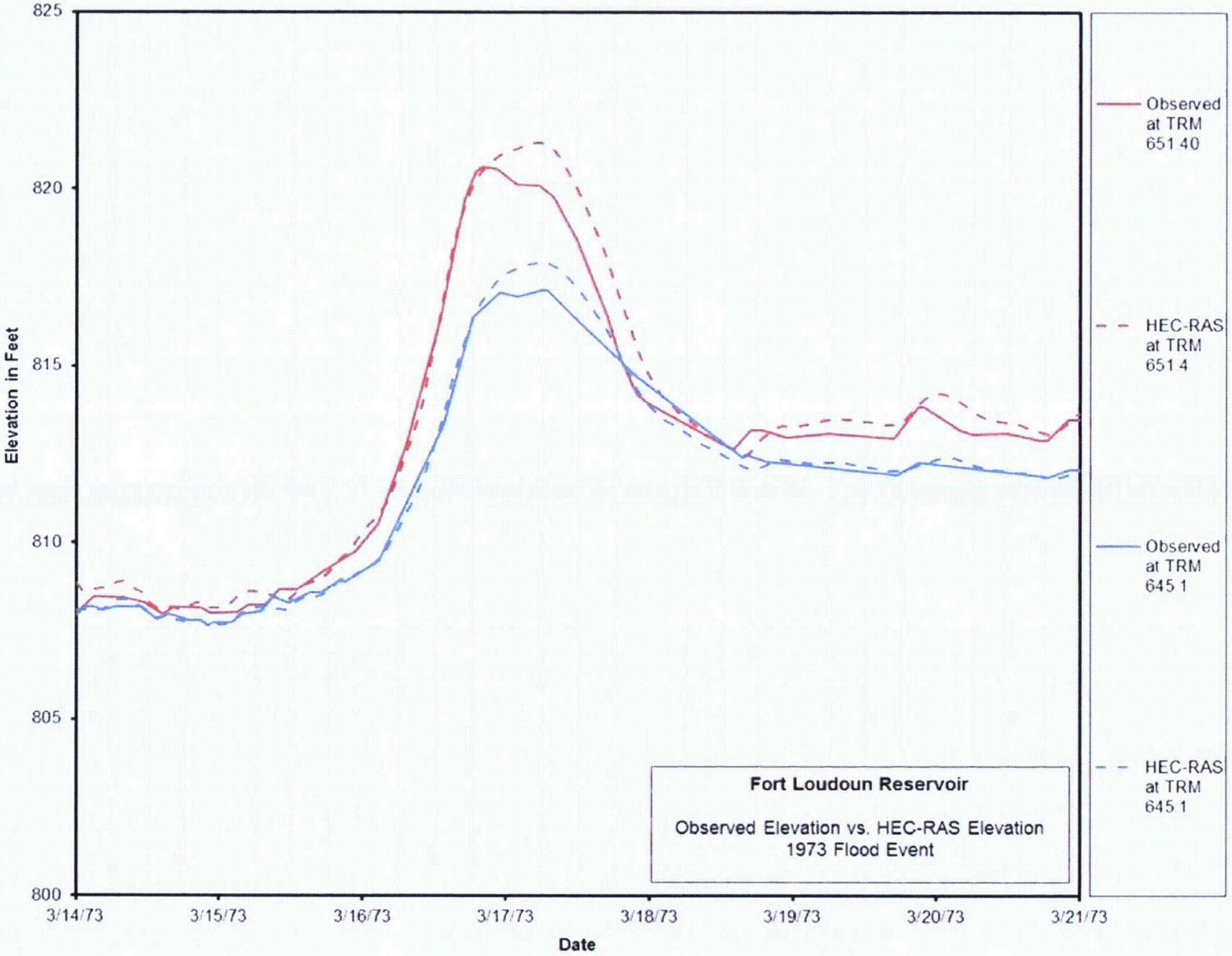


Figure 2.4-13 Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood (Sheet 1 of 4)

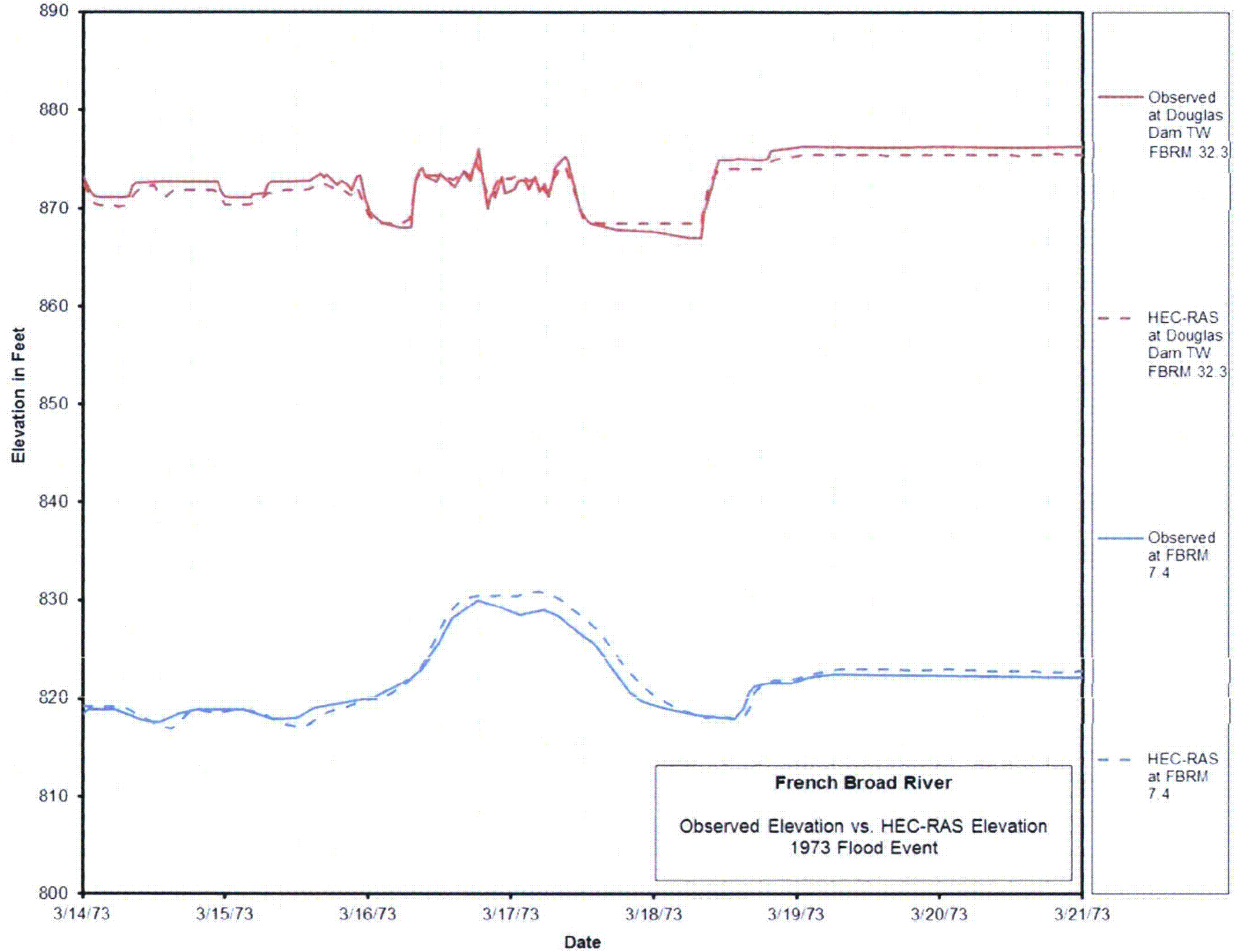


Figure 2.4-13 Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood (Sheet 2 of 4)

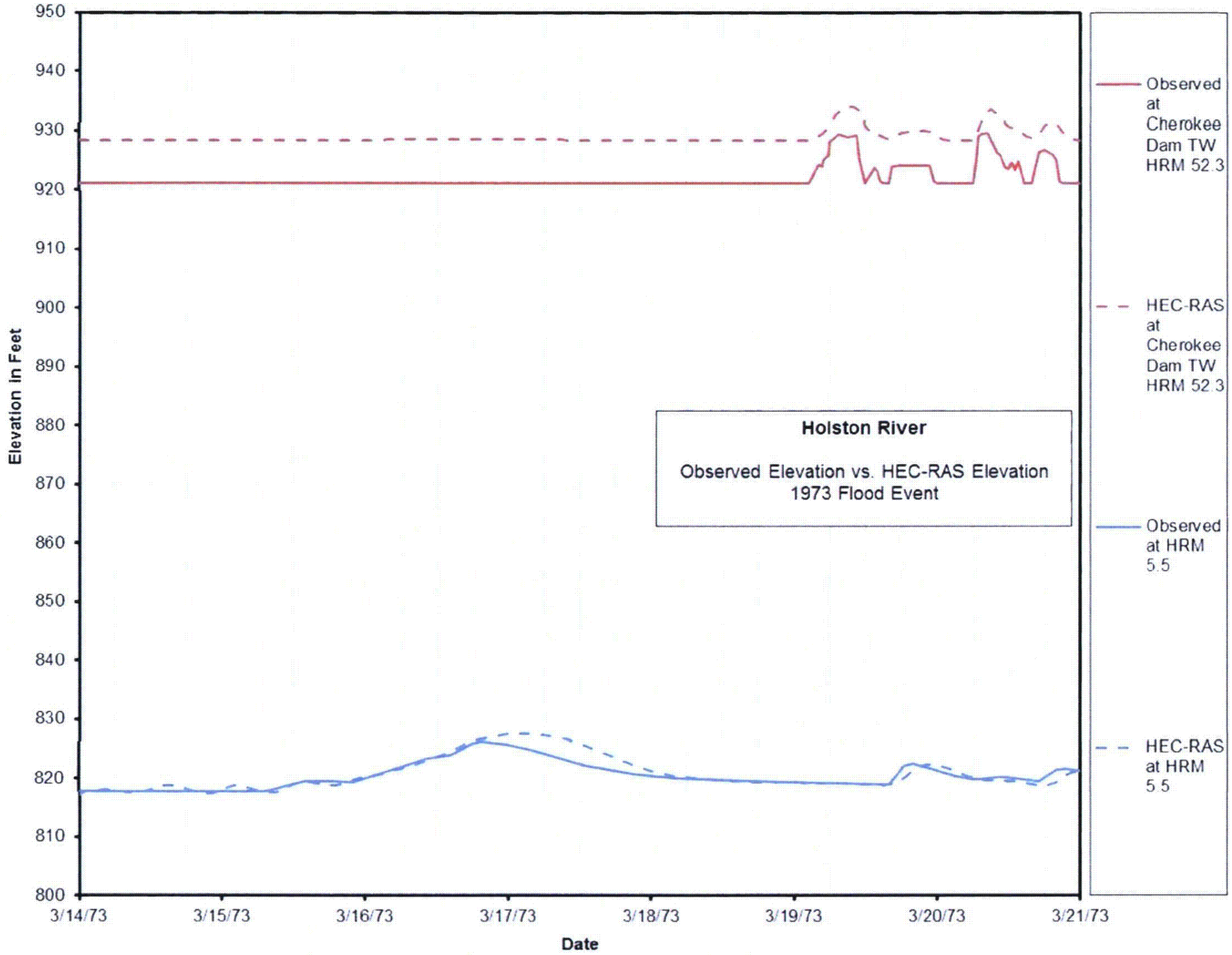


Figure 2.4-13 Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood (Sheet 3 of 4)

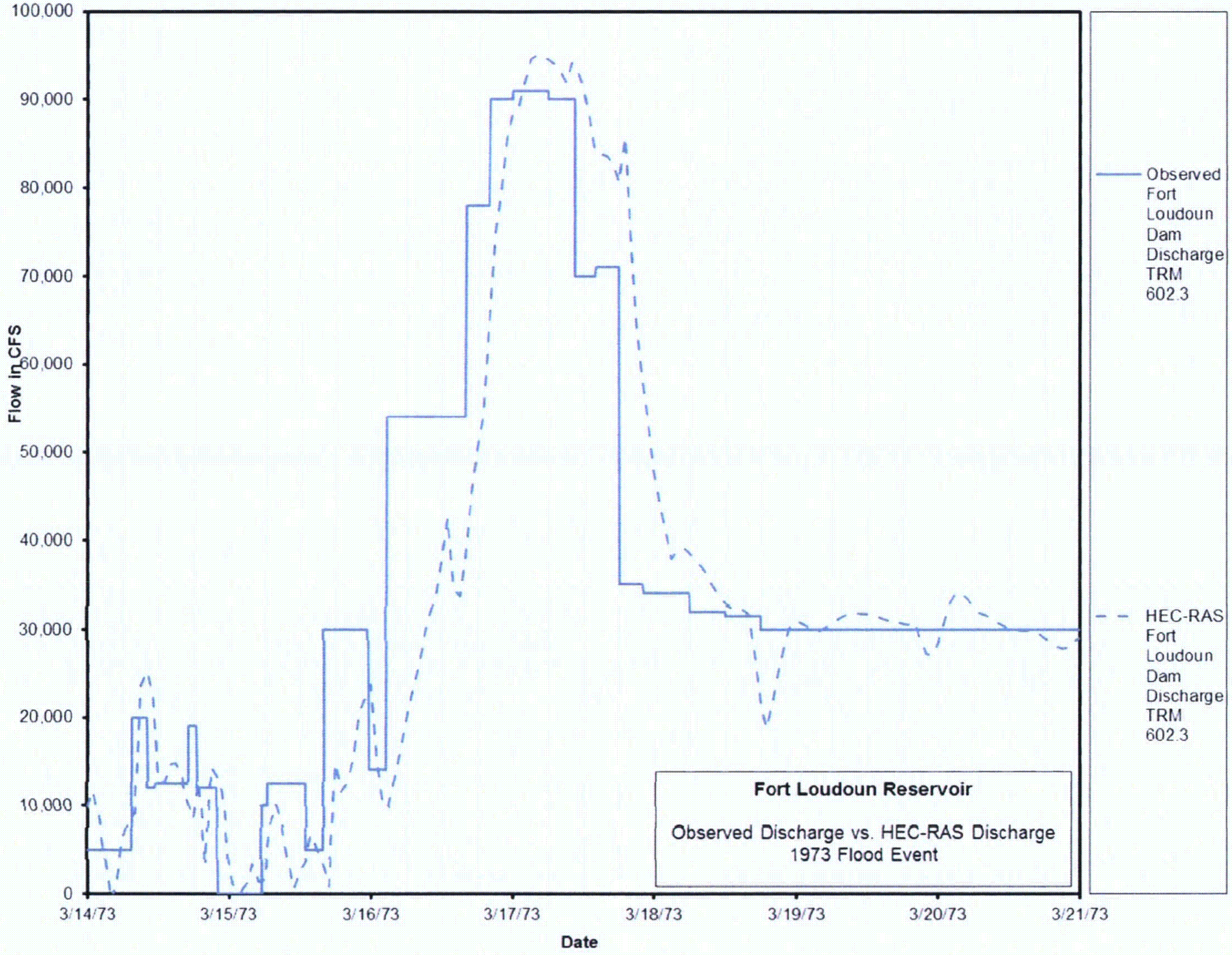


Figure 2.4-13 Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood (Sheet 4 of 4)

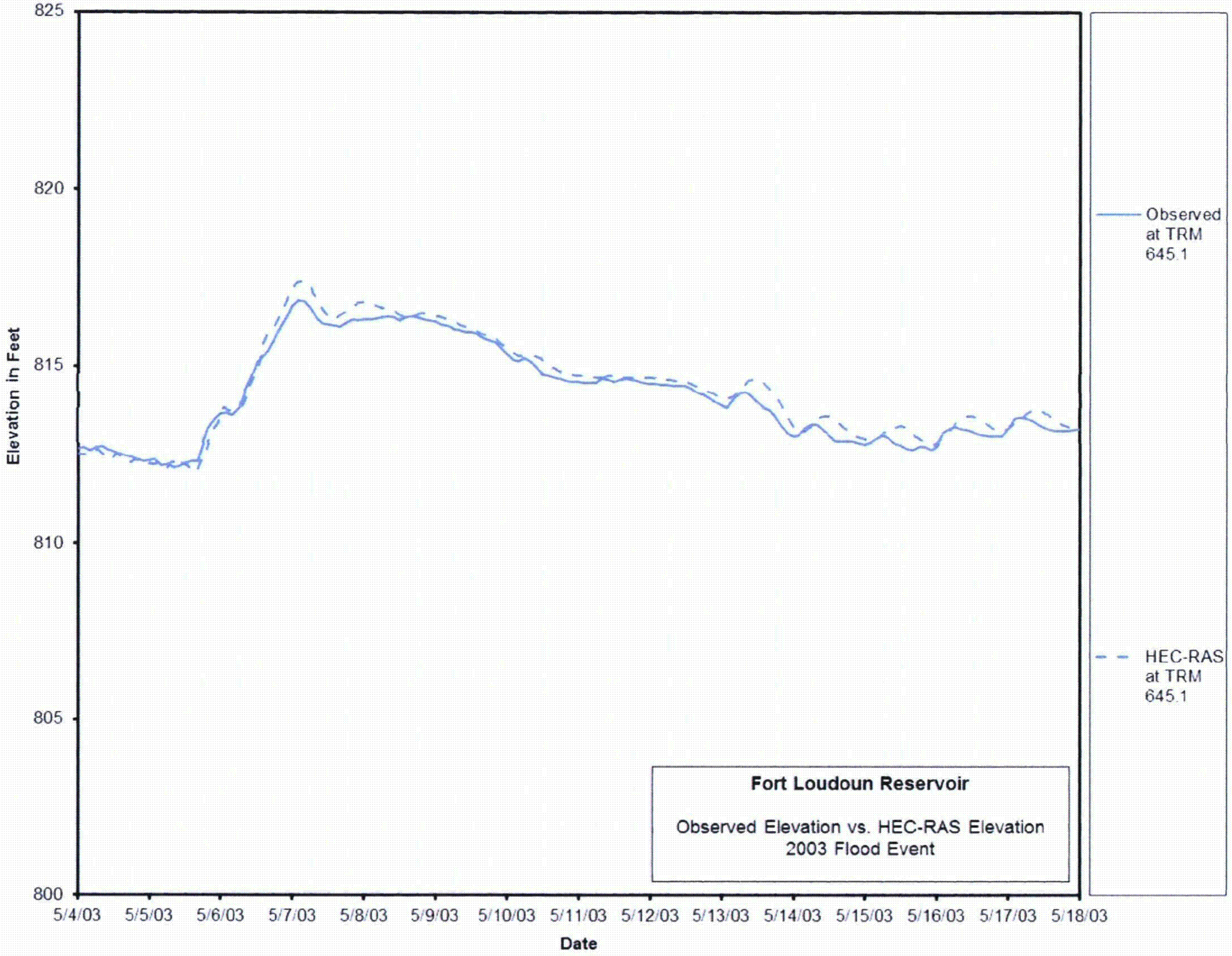
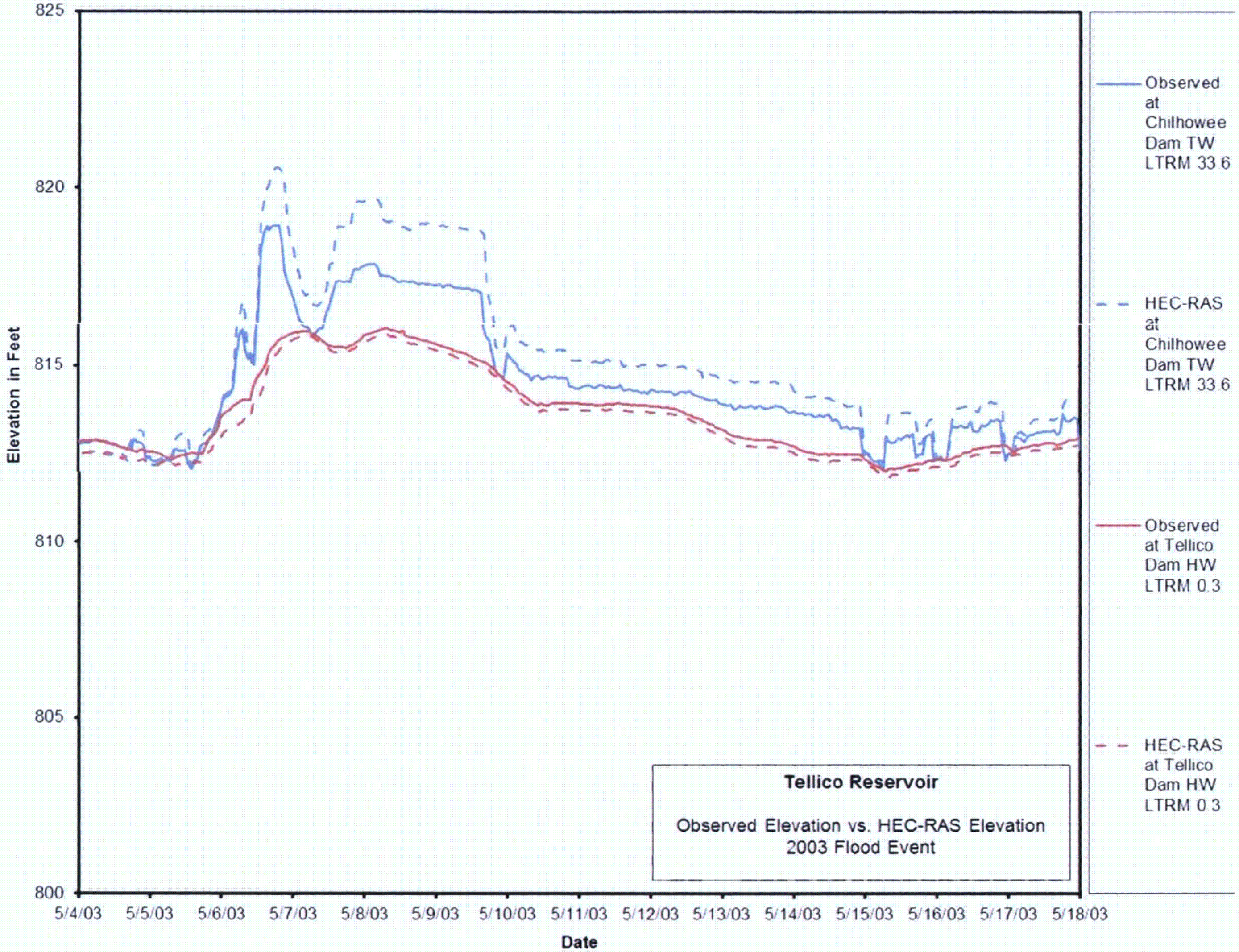


Figure 2.4-14 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 1 of 5)



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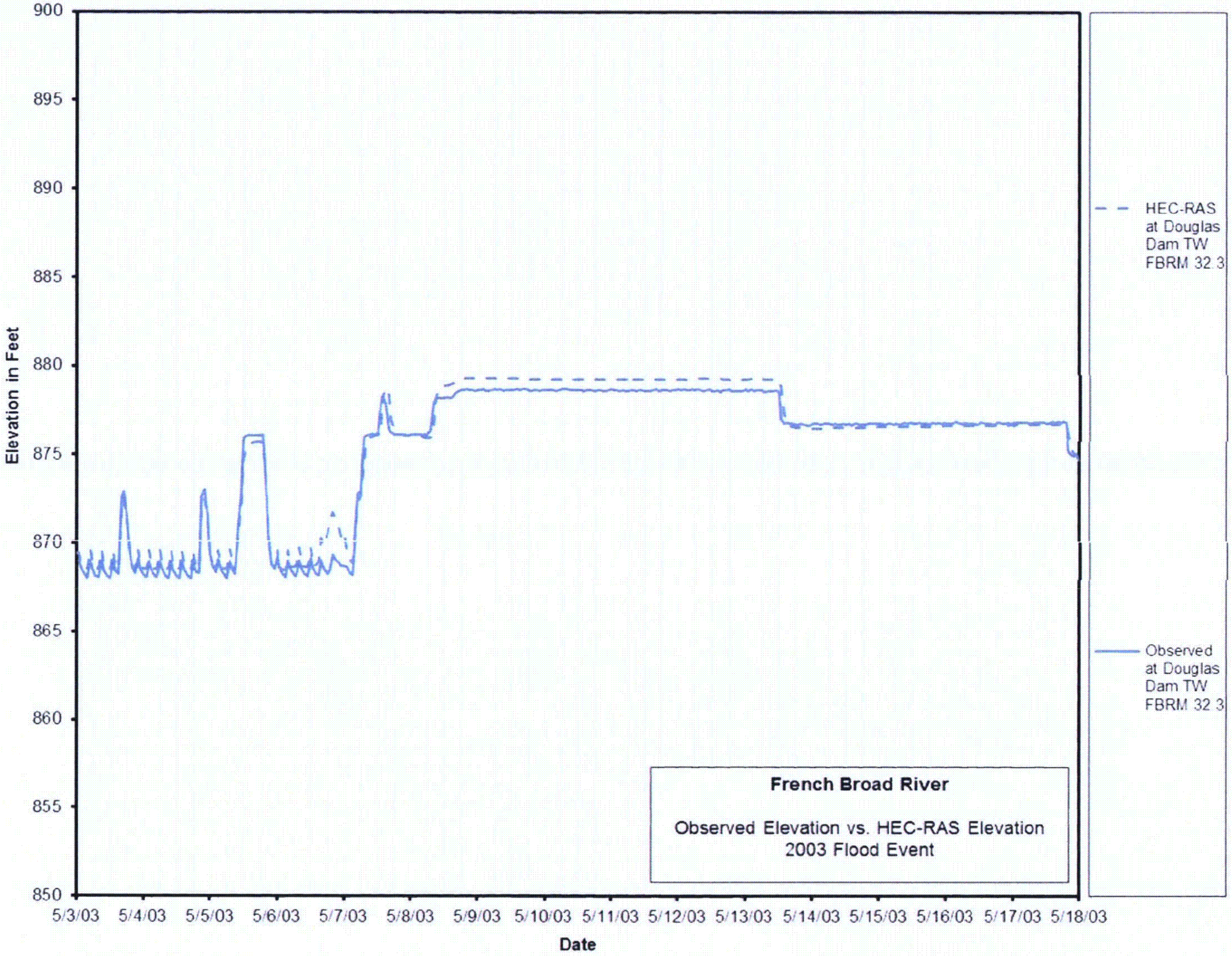


Figure 2.4-14 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 3 of 5)

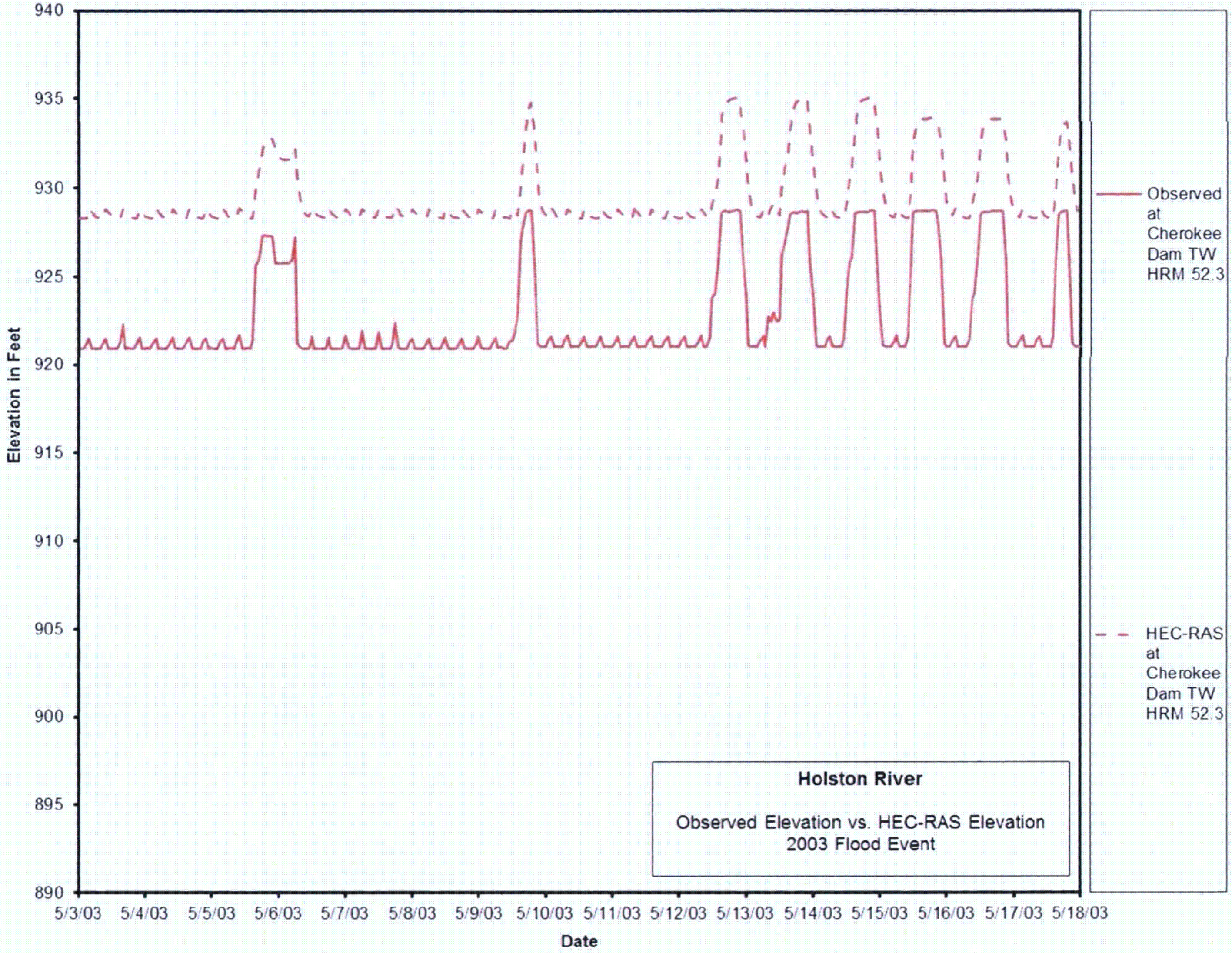


Figure 2.4-14 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 4 of 5)

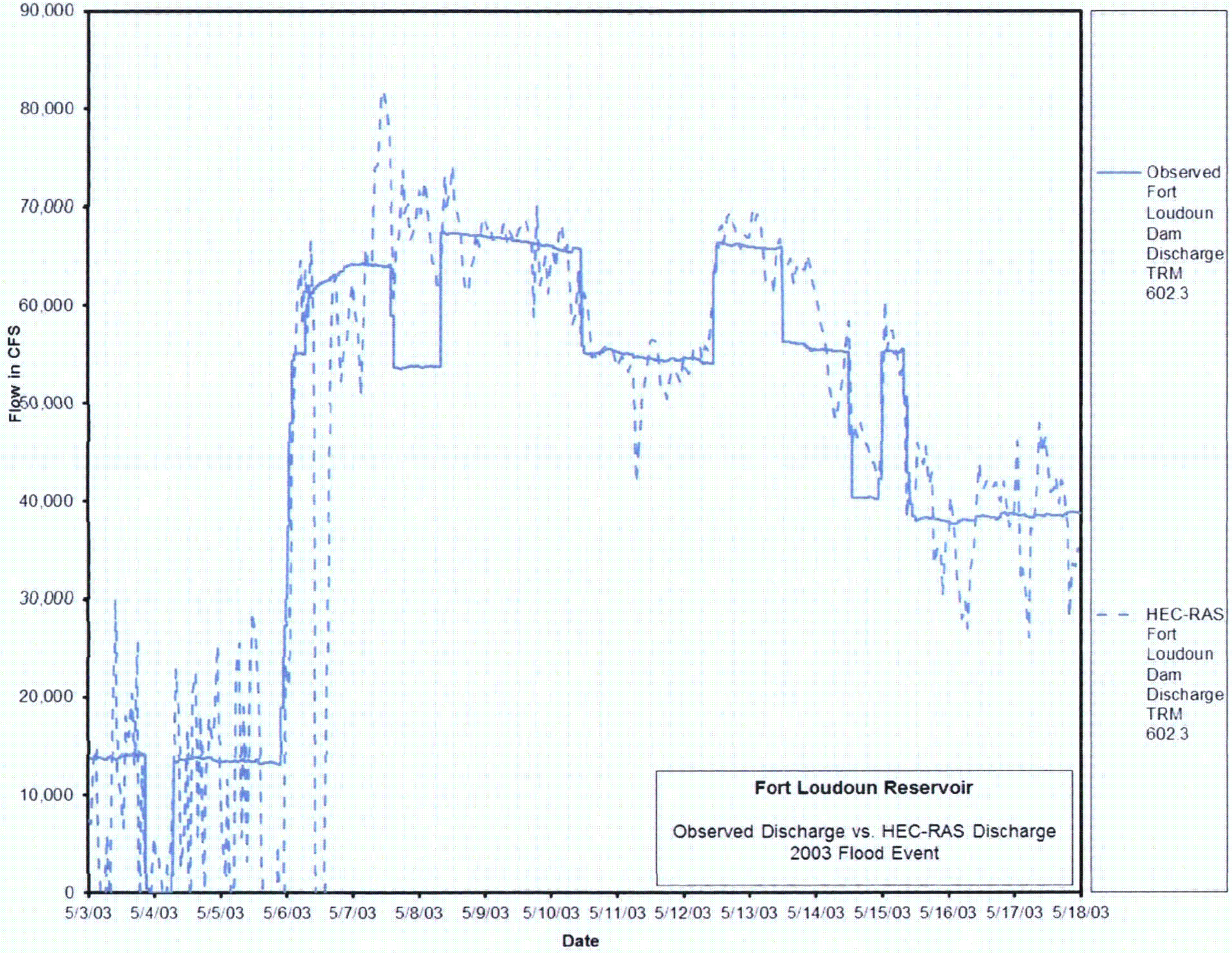


Figure 2.4-14 Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood (Sheet 5 of 5)

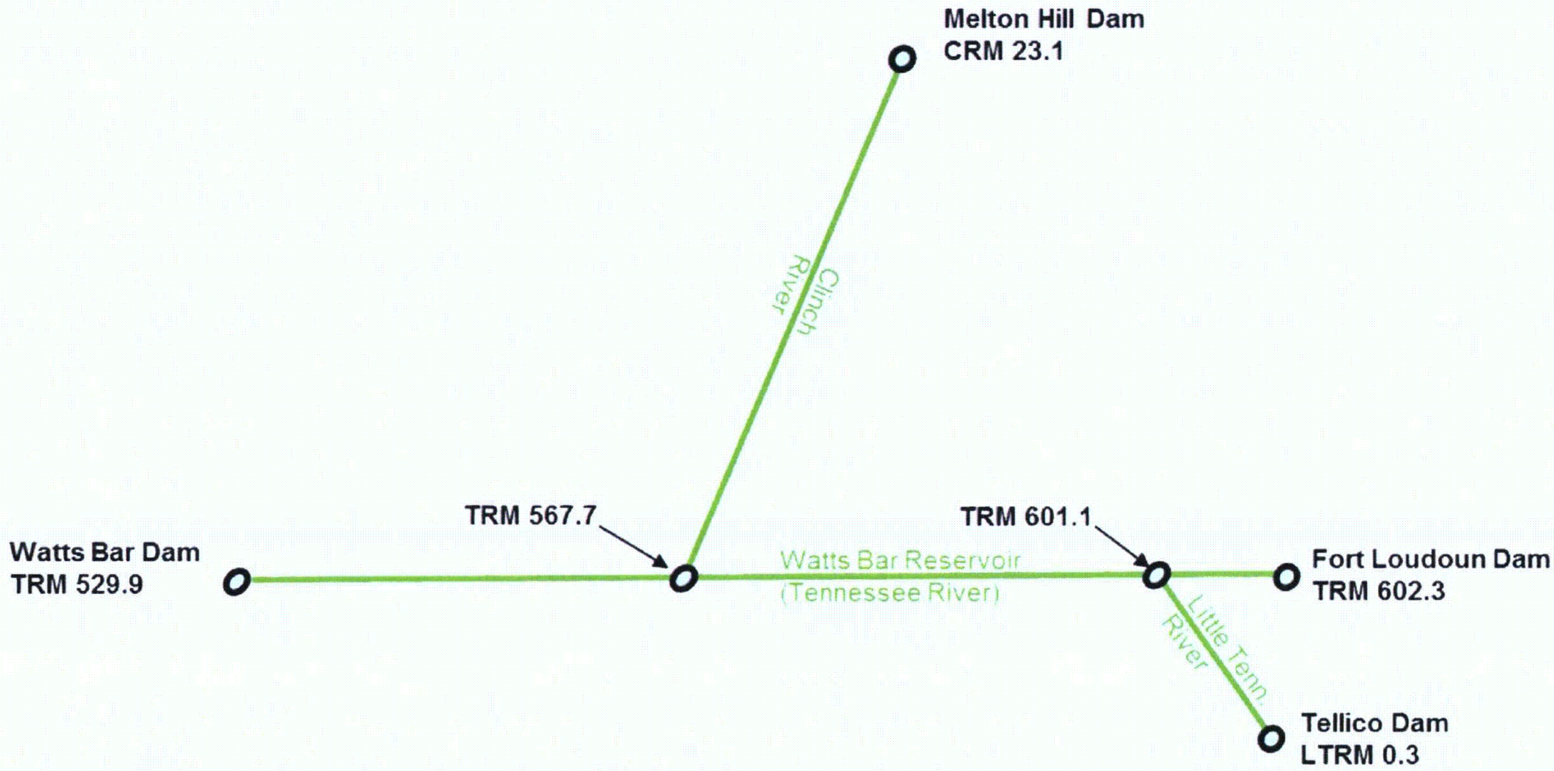


Figure 2.4-15 Watts Bar HEC-RAS Unsteady Flow Model Schematic

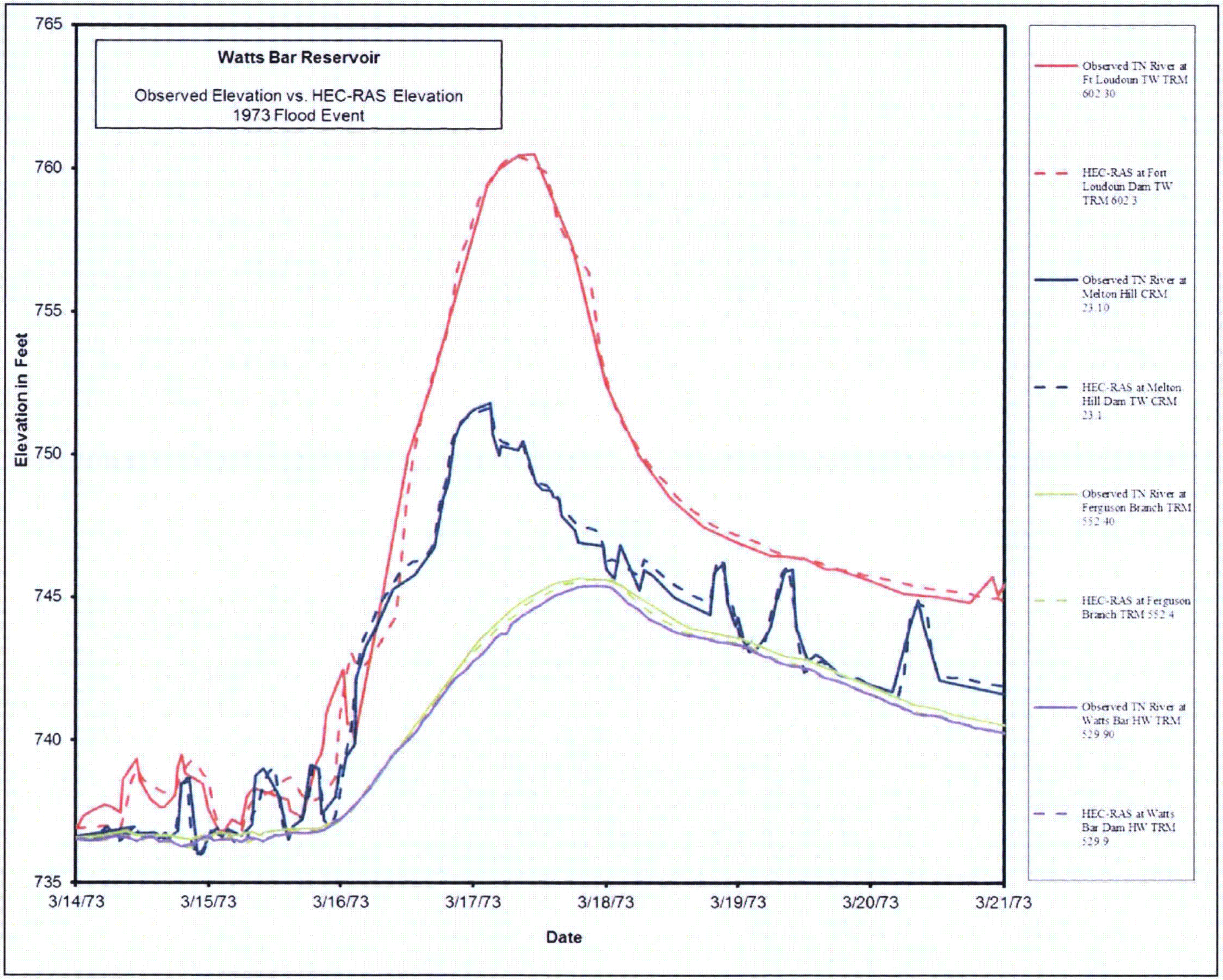


Figure 2.4-16 Unsteady Flow Model Watts Bar Reservoir March 1973 Flood (Sheet 1 of 2)

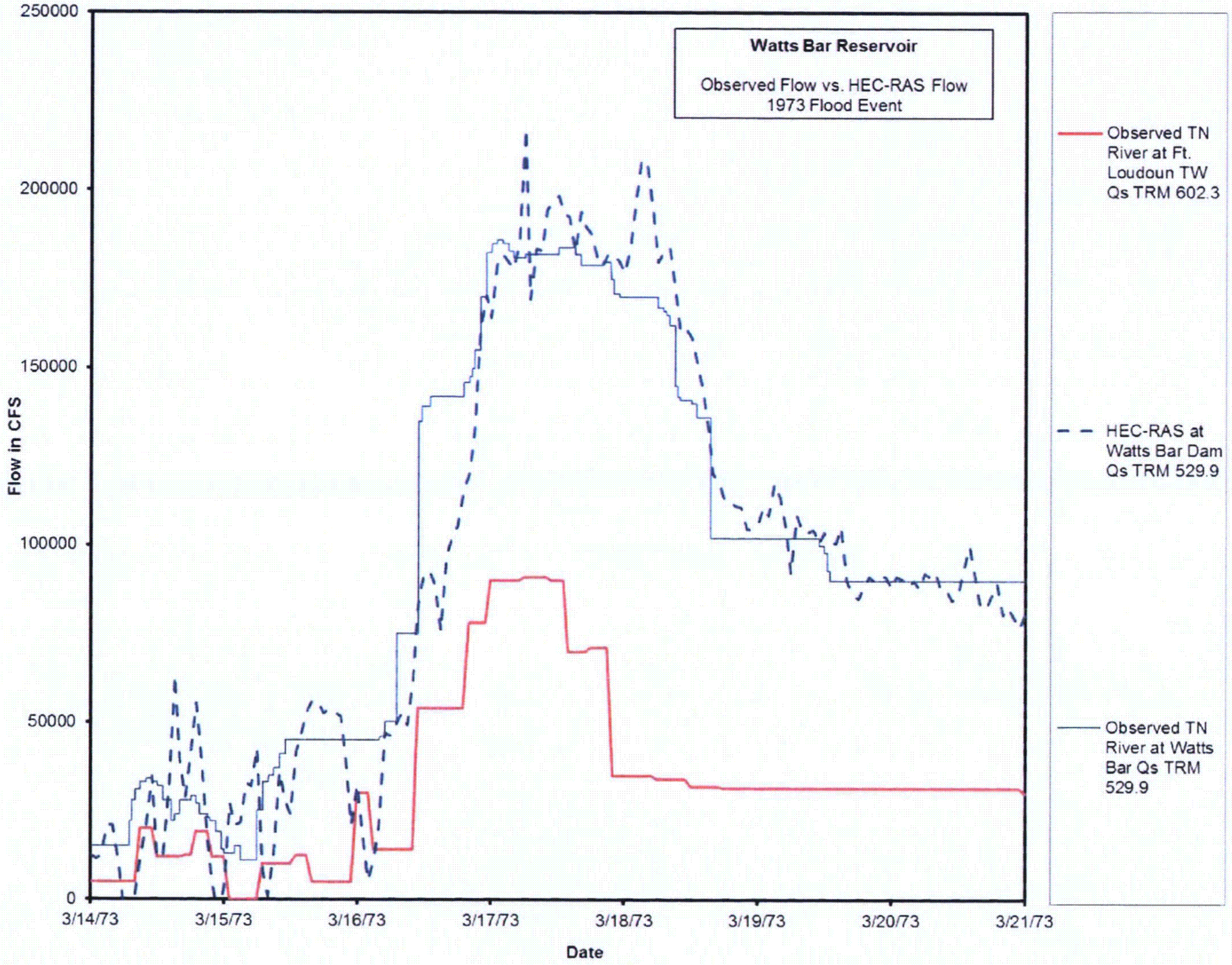


Figure 2.4-16 Unsteady Flow Model Watts Bar Reservoir March 1973 Flood (Sheet 2 of 2)

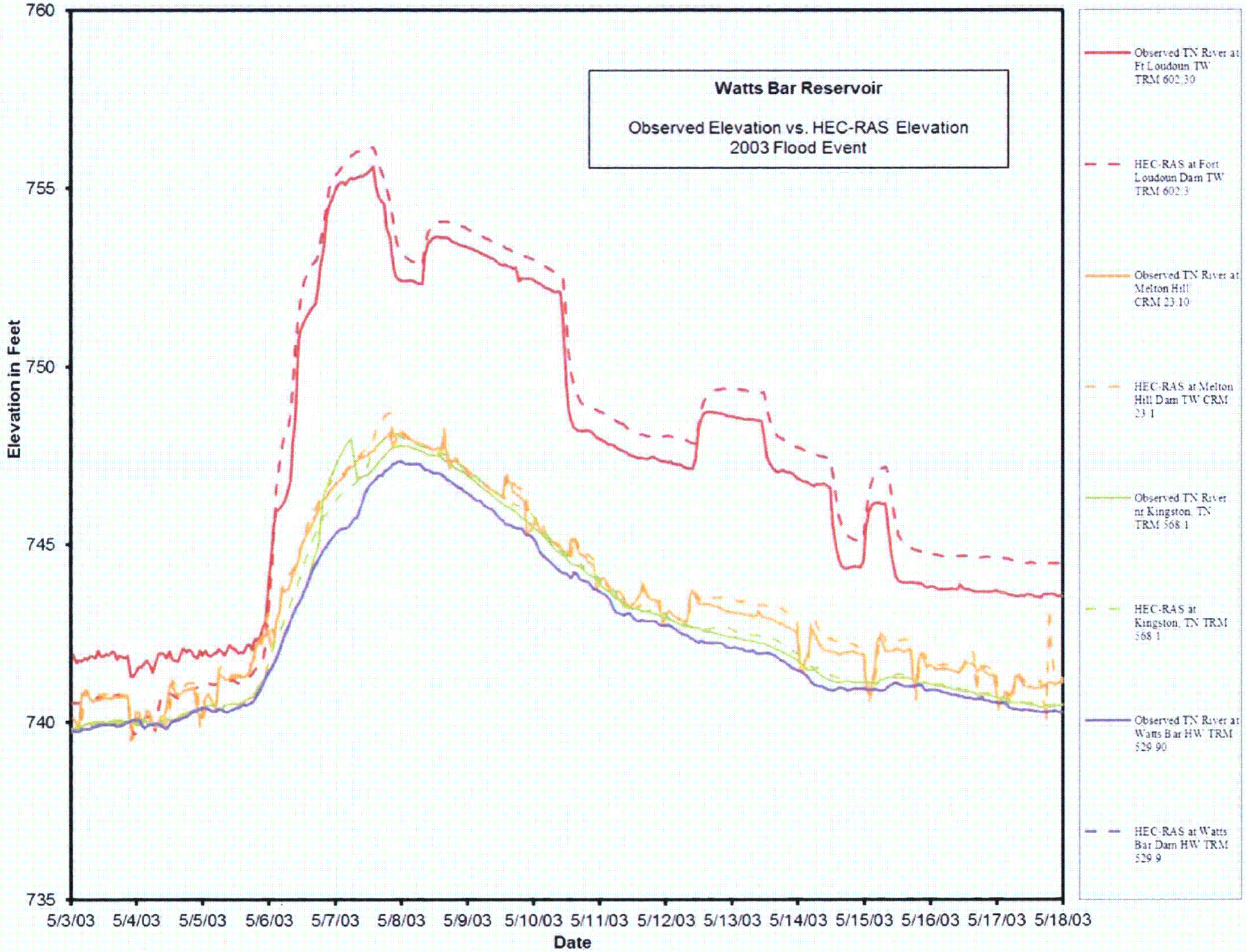


Figure 2.4-17 Unsteady Flow Model Watts Bar Reservoir May 2003 Flood (Sheet 1 of 2)

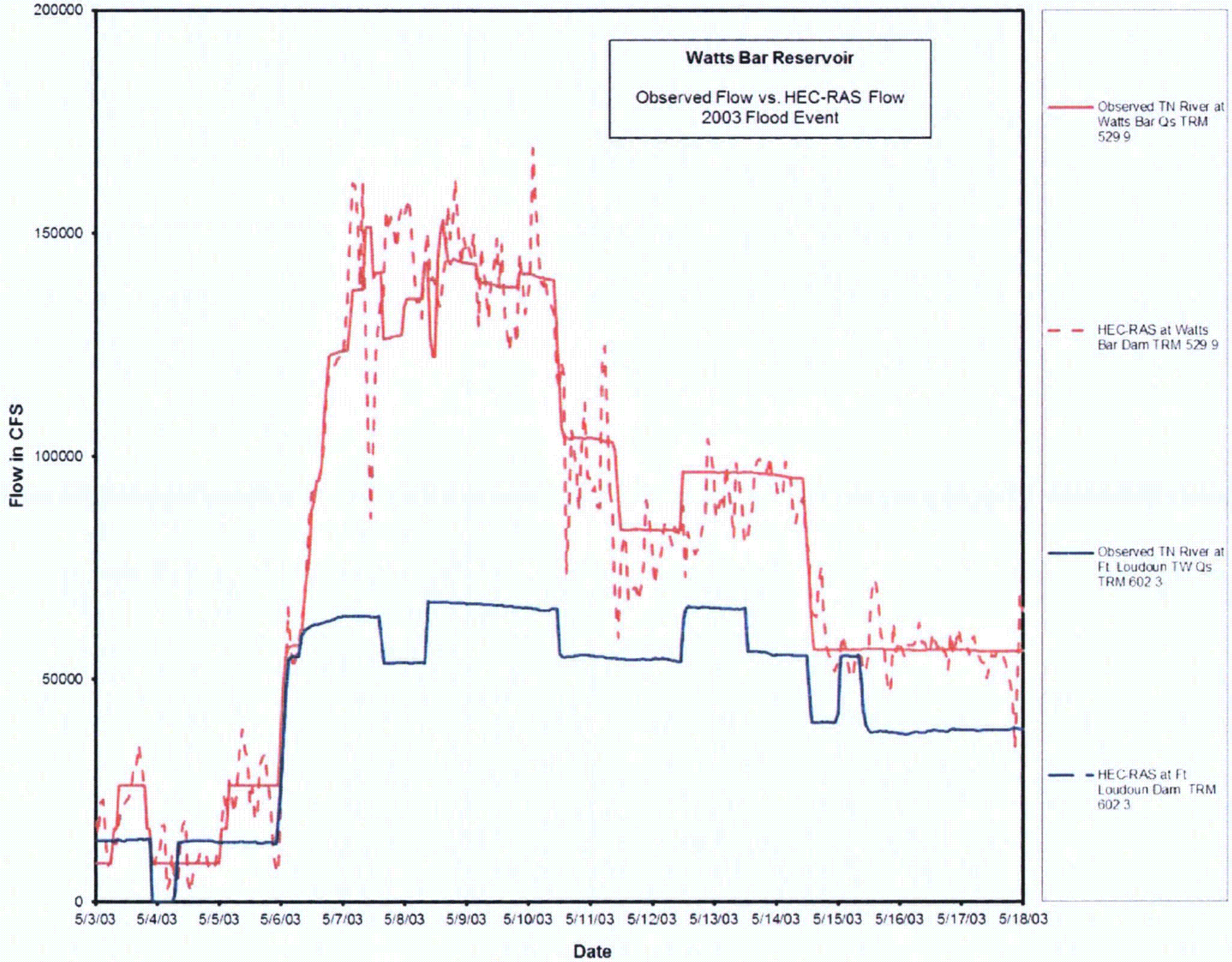


Figure 2.4-17 Unsteady Flow Model Watts Bar Reservoir May 2003 Flood (Sheet 2 of 2)

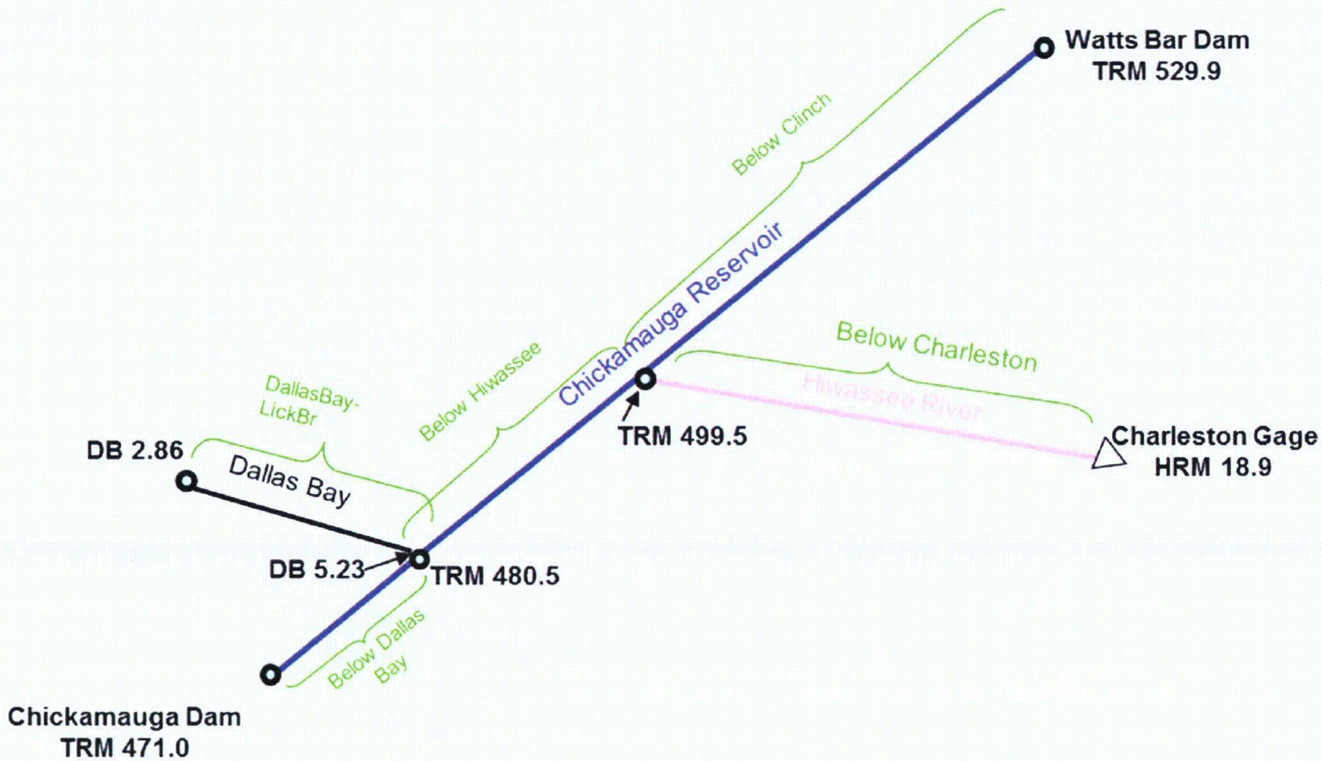


Figure 2.4-18 Chickamauga HEC-RAS Unsteady Flow Model Schematic

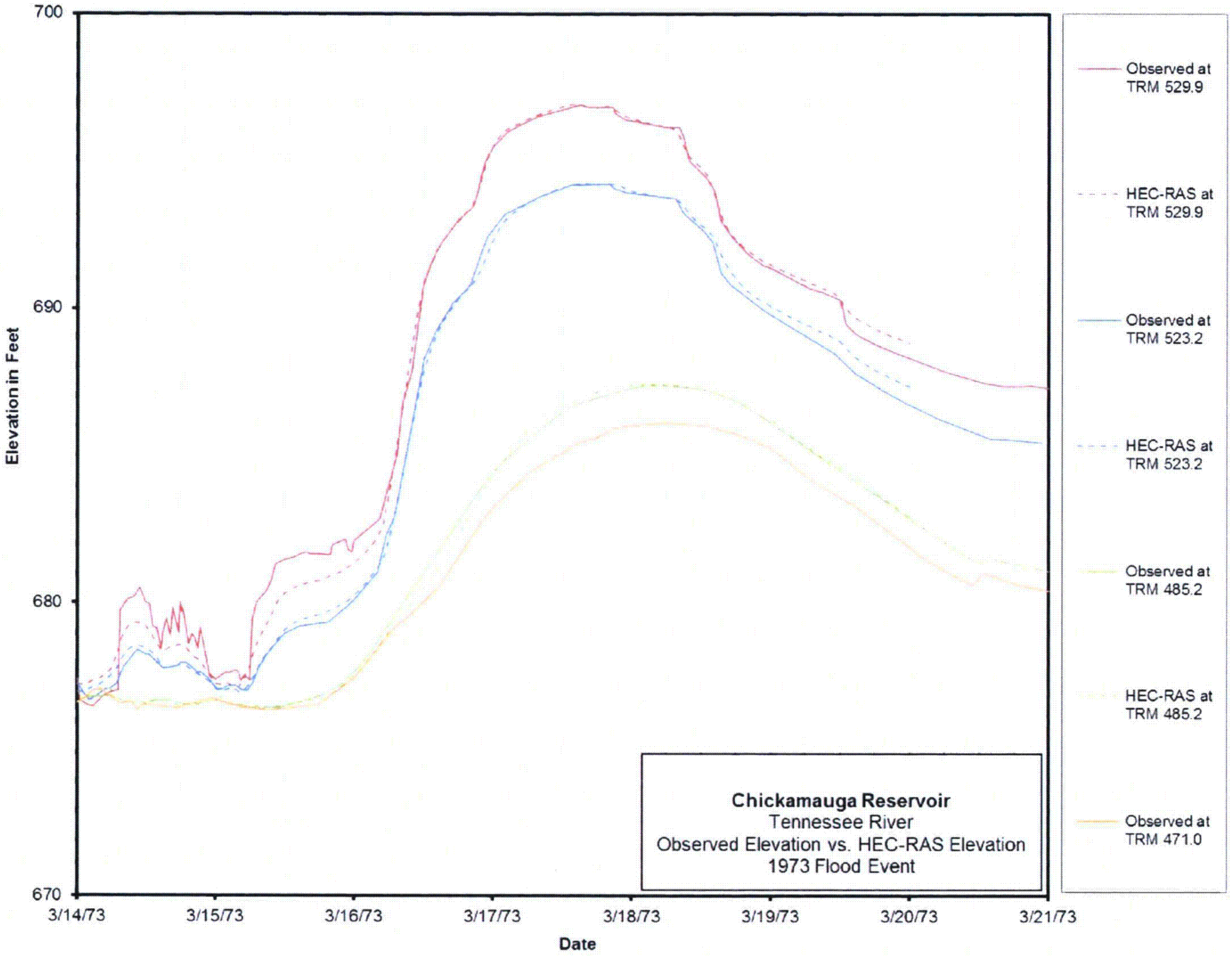


Figure 2.4-19 Unsteady Flow Model Chickamauga Reservoir March 1973 Flood (Sheet 1 of 3)

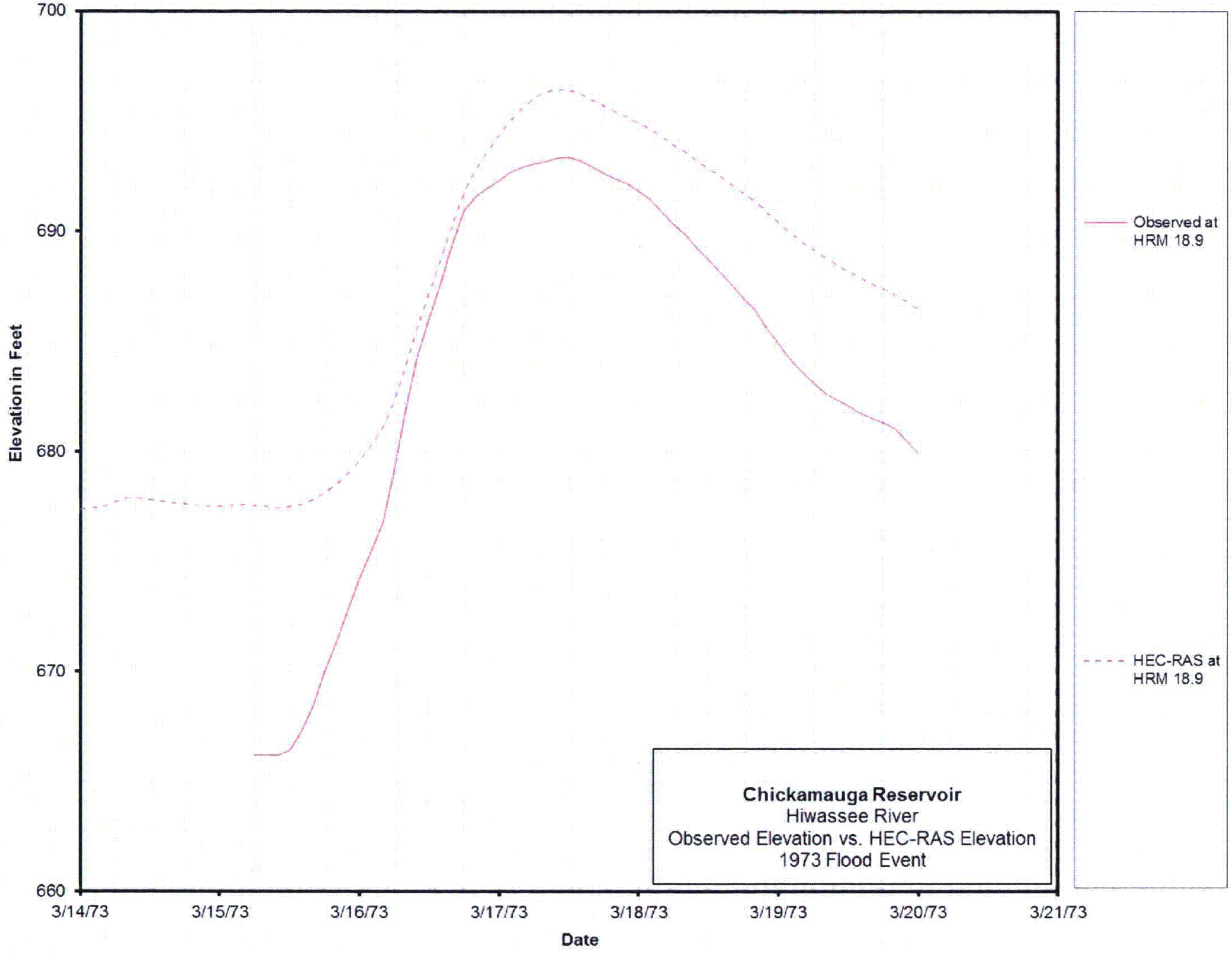


Figure 2.4-19 Unsteady Flow Model Chickamauga Reservoir March 1973 Flood (Sheet 2 of 3)

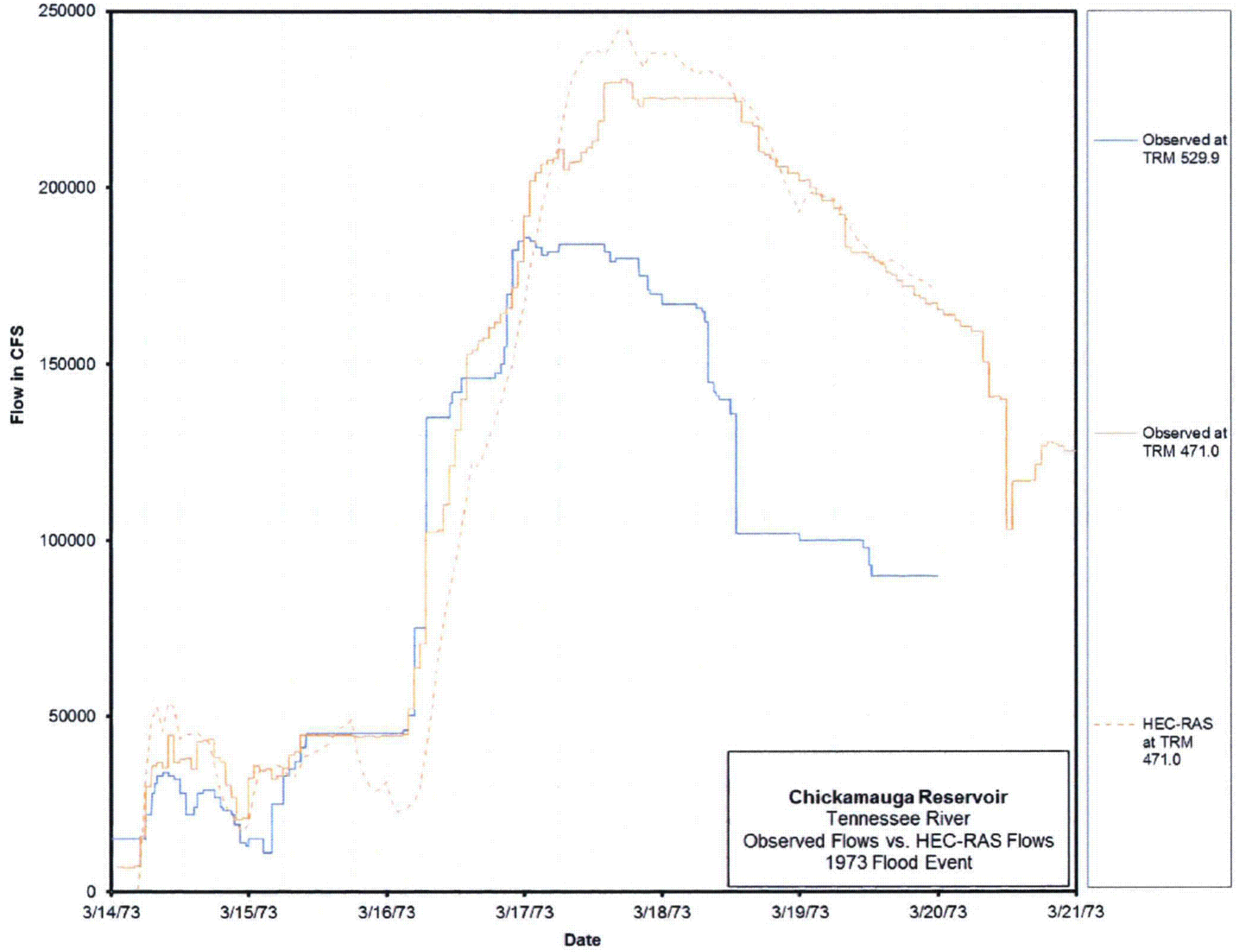


Figure 2.4-19 Unsteady Flow Model Chickamauga Reservoir March 1973 Flood (Sheet 3 of 3)

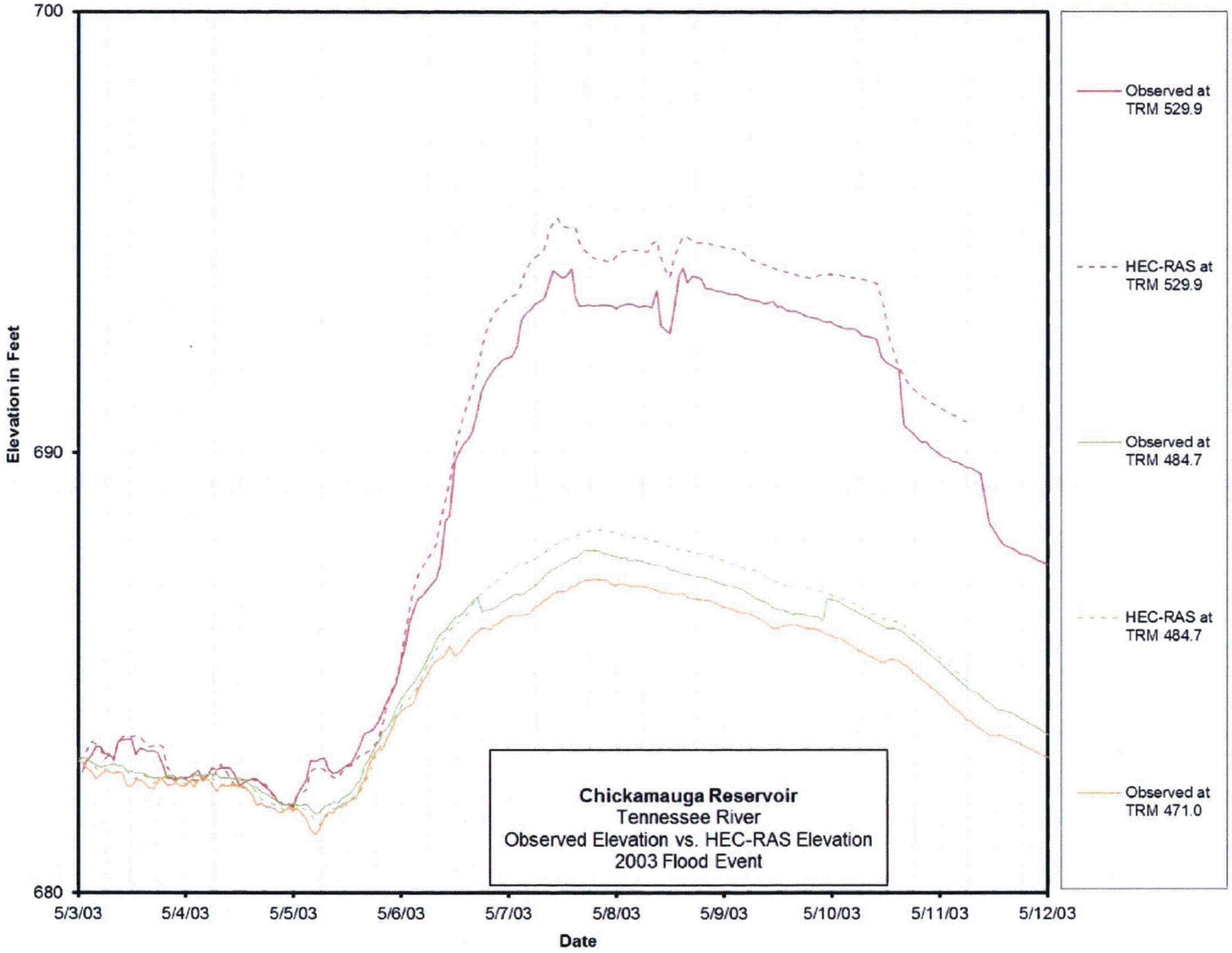


Figure 2.4-20 Unsteady Flow Model Chickamauga Reservoir May 2003 Flood (Sheet 1 of 3)

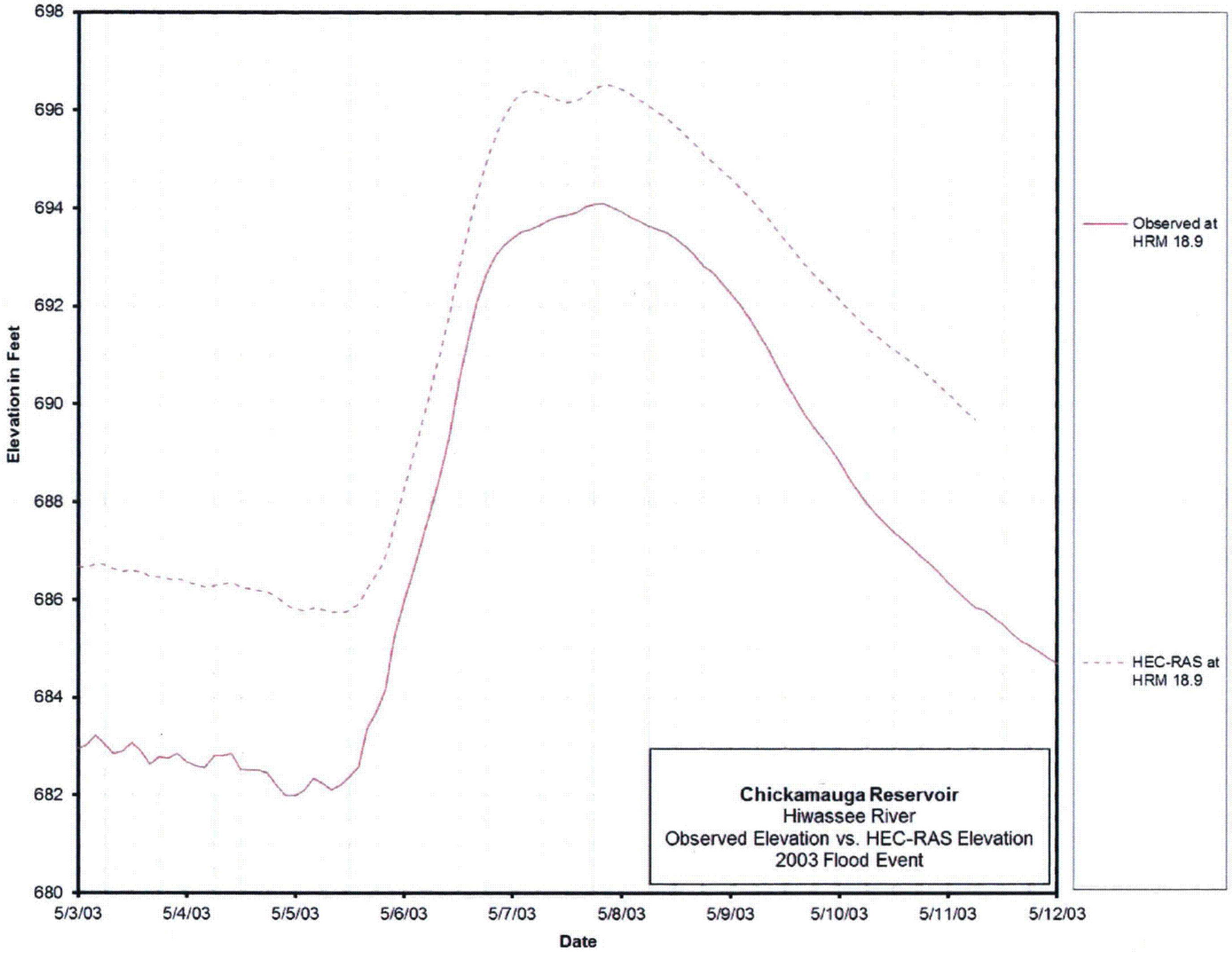


Figure 2.4-20 Unsteady Flow Model Chickamauga Reservoir May 2003 Flood (Sheet 2 of 3)

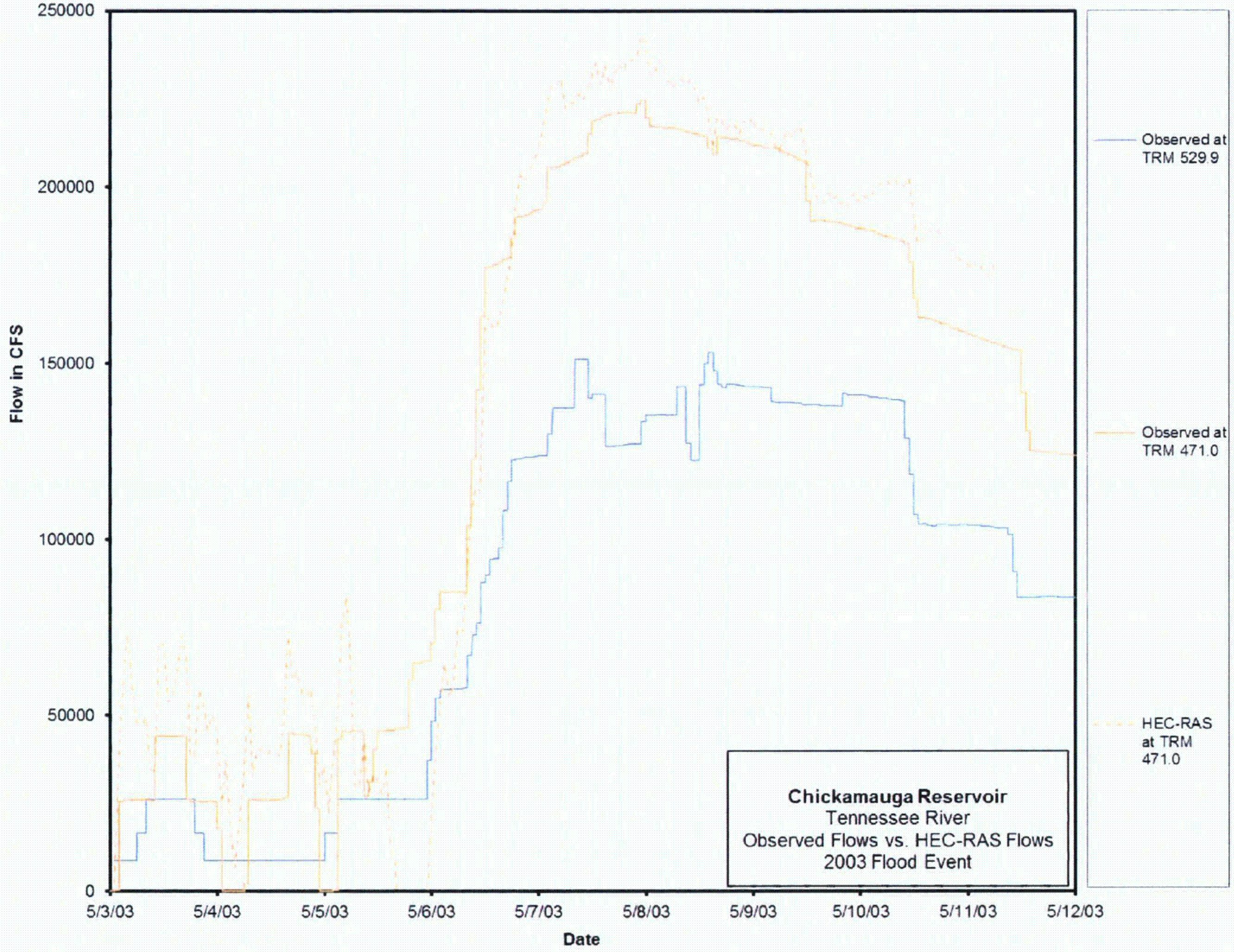


Figure 2.4-20 Unsteady Flow Model Chickamauga Reservoir May 2003 Flood (Sheet 3 of 3)

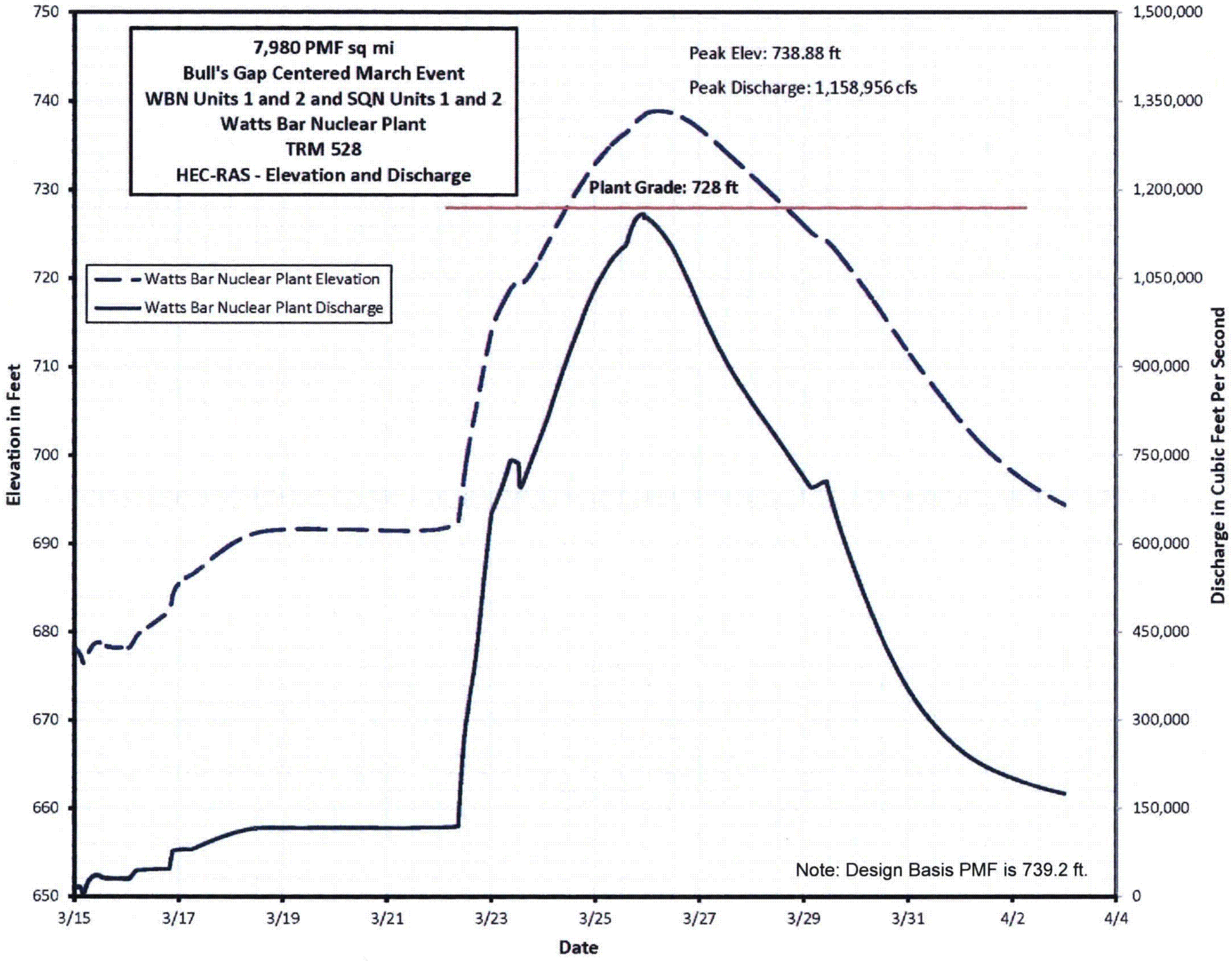


Figure 2.4-23 PMF Discharge and Elevation Hydrograph at Watts Bar Nuclear Plant (Sheet 1 of 2)

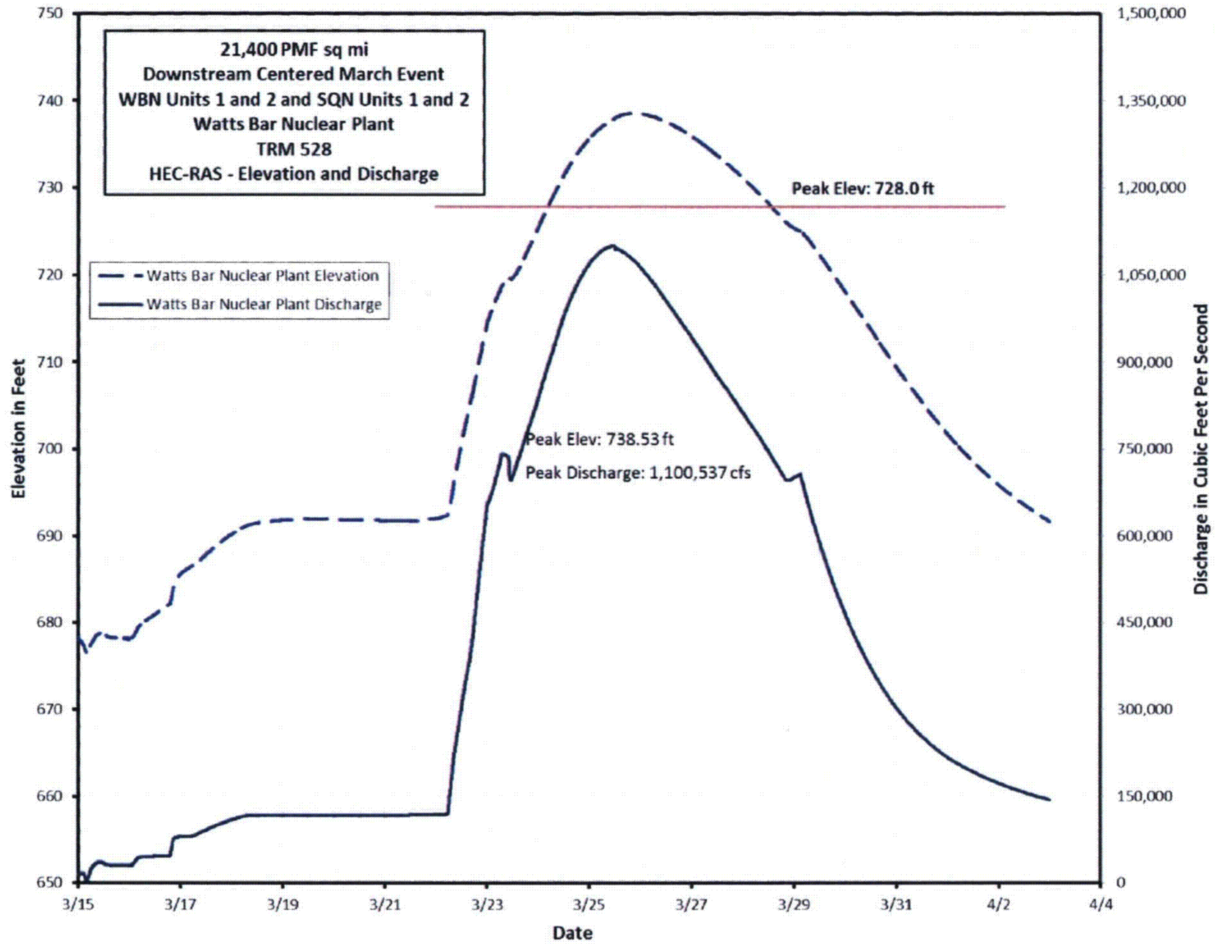


Figure 2.4-23 PMF Discharge and Elevation Hydrograph at Watts Bar Nuclear Plant (Sheet 2 of 2)

Figure 2.4-24 Is Not Used

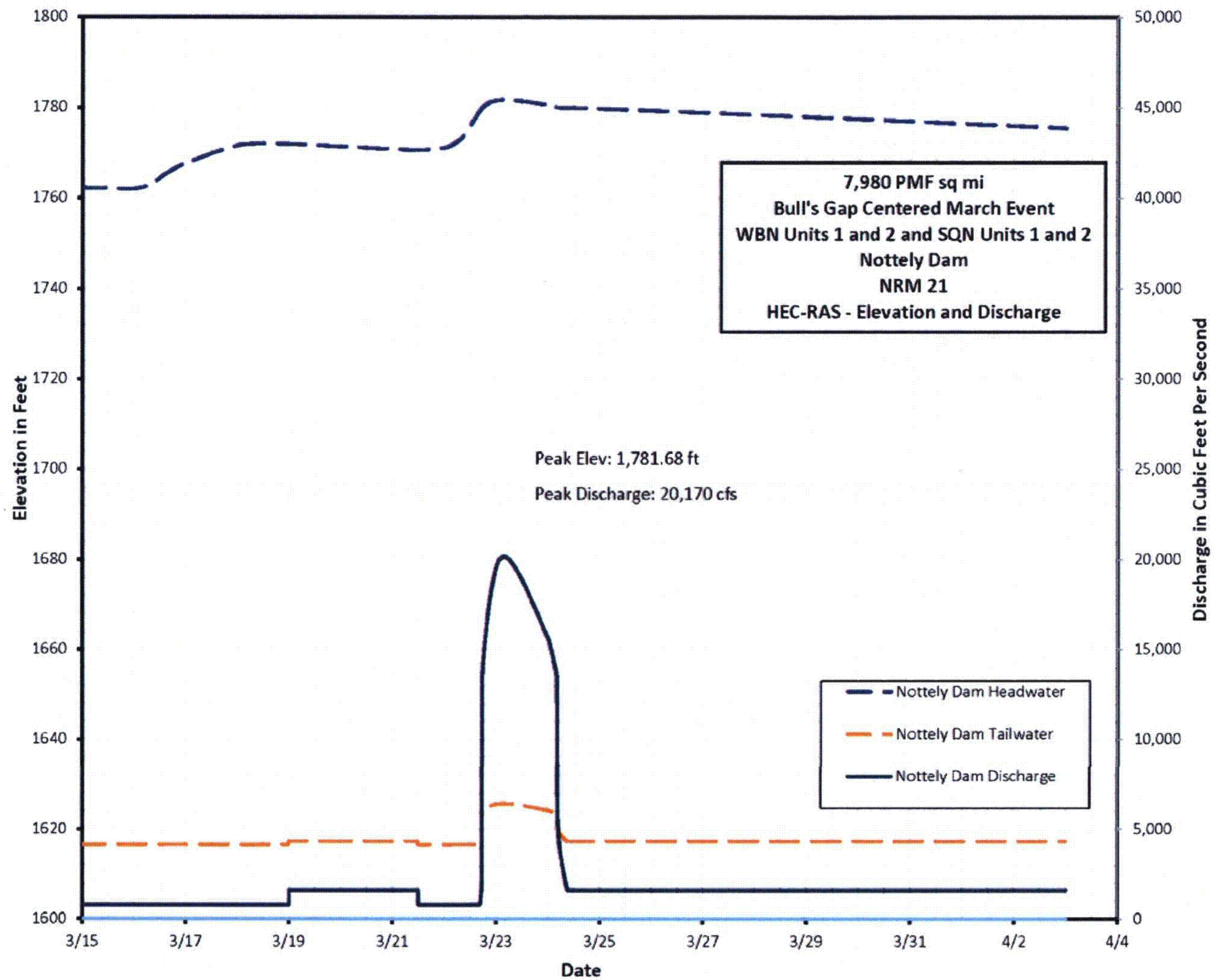


Figure 2.4- 25 Nottely Dam Hydrograph (Sheet 1 of 27)

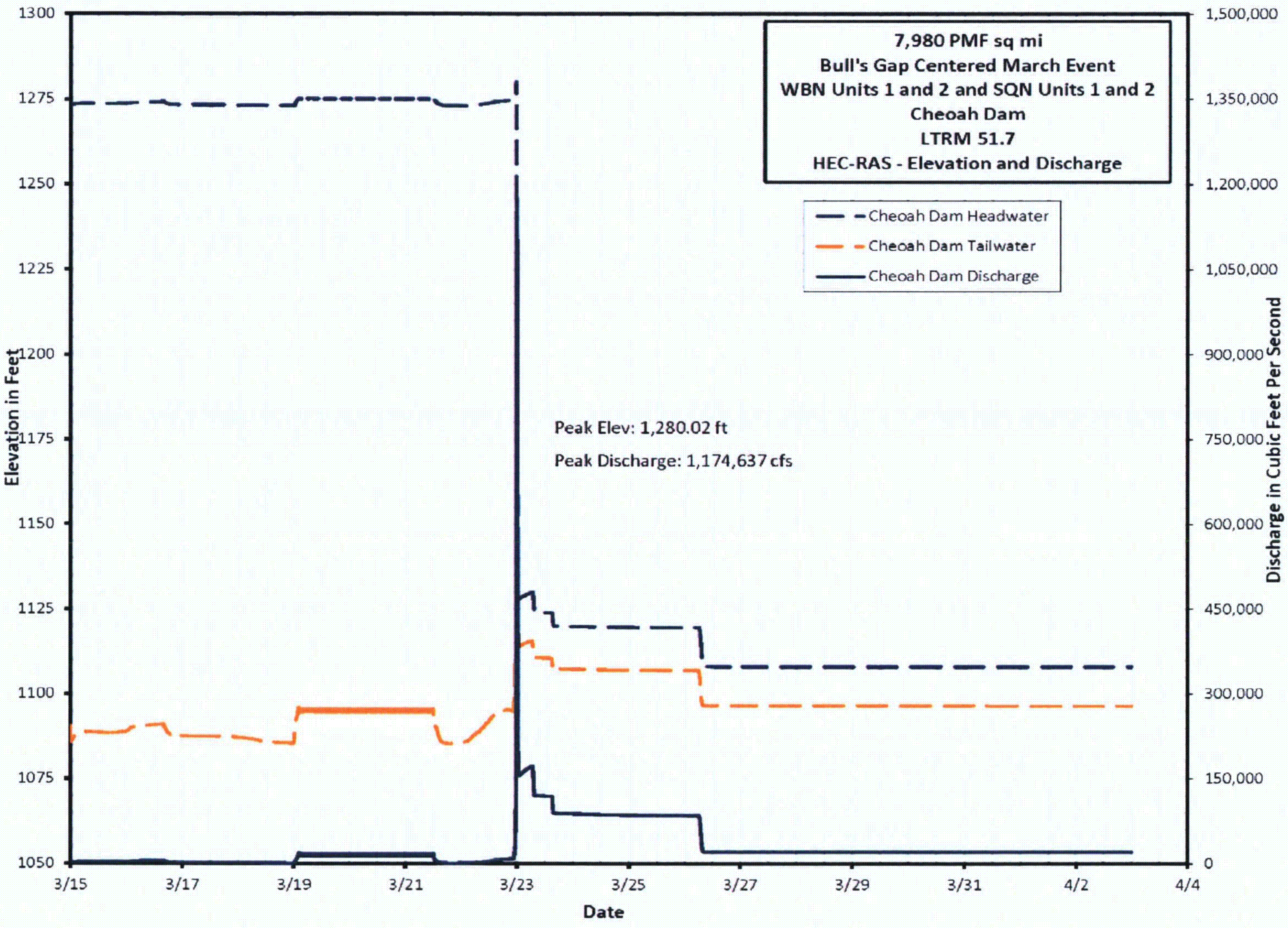


Figure 2.4- 25 Cheoah Dam Hydrograph (Sheet 2 of 27)

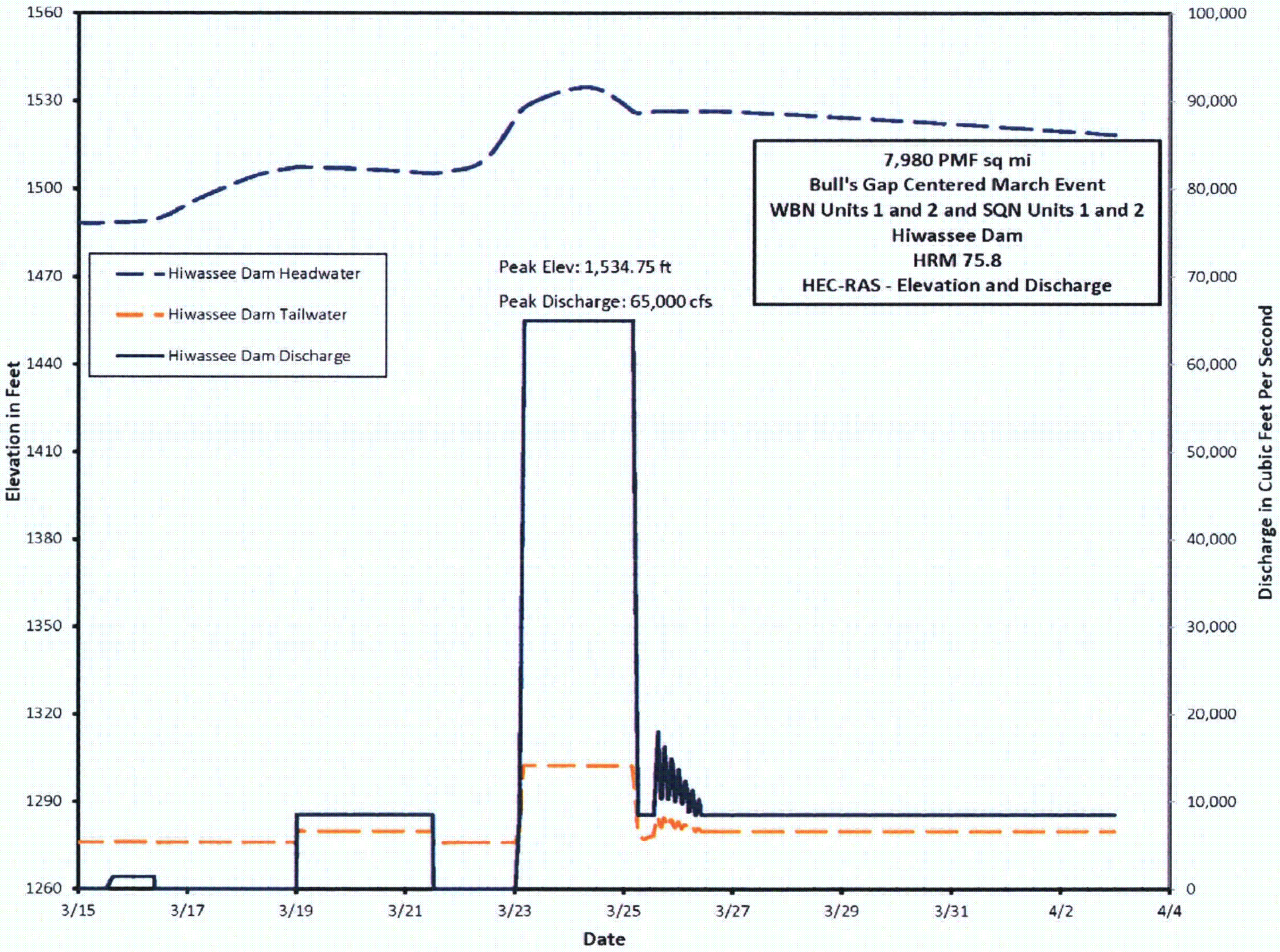


Figure 2.4- 25 Hiwassee Dam Hydrograph (Sheet 3 of 27)

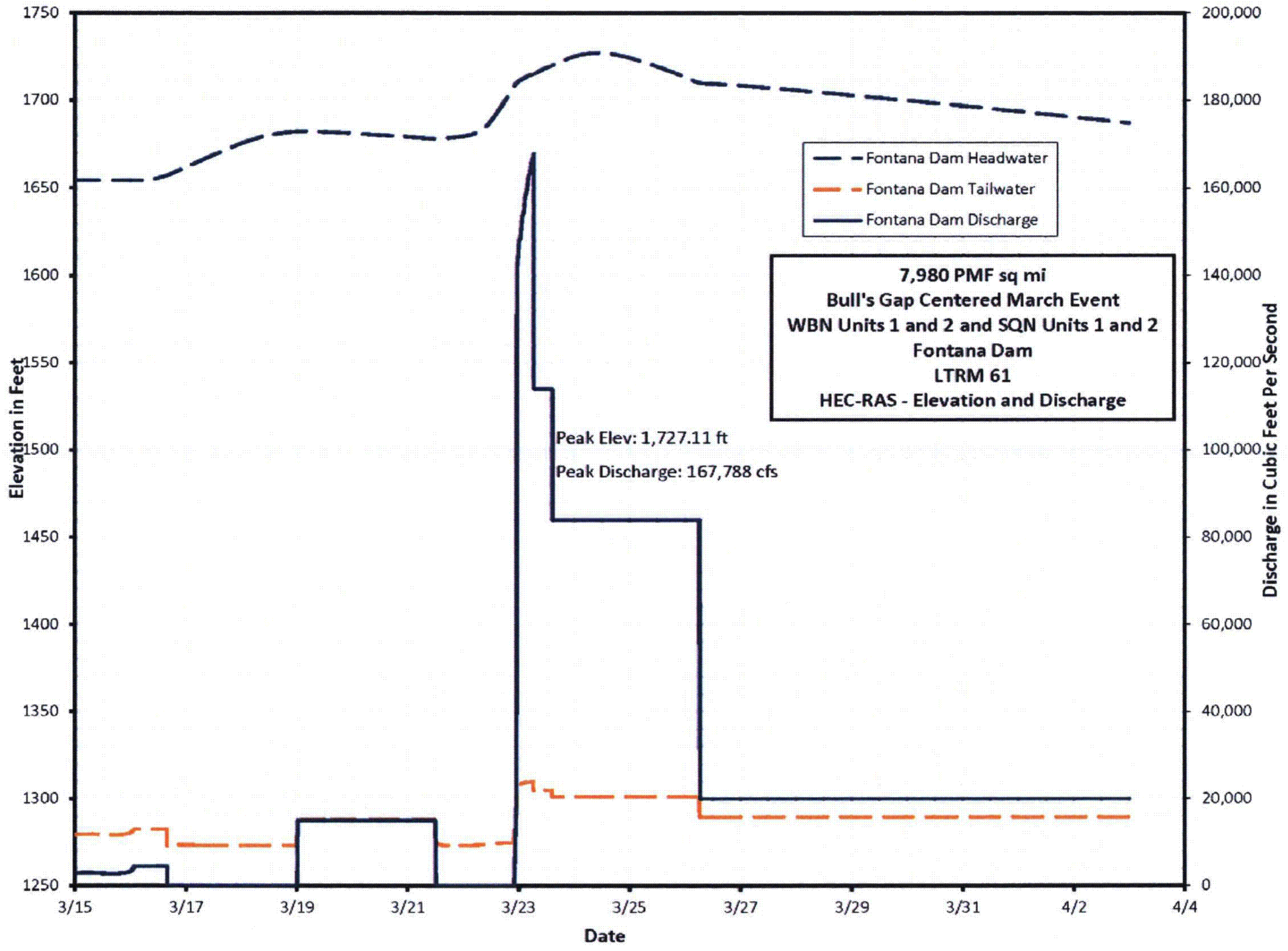


Figure 2.4- 25 Fontana Dam Hydrograph (Sheet 4 of 27)

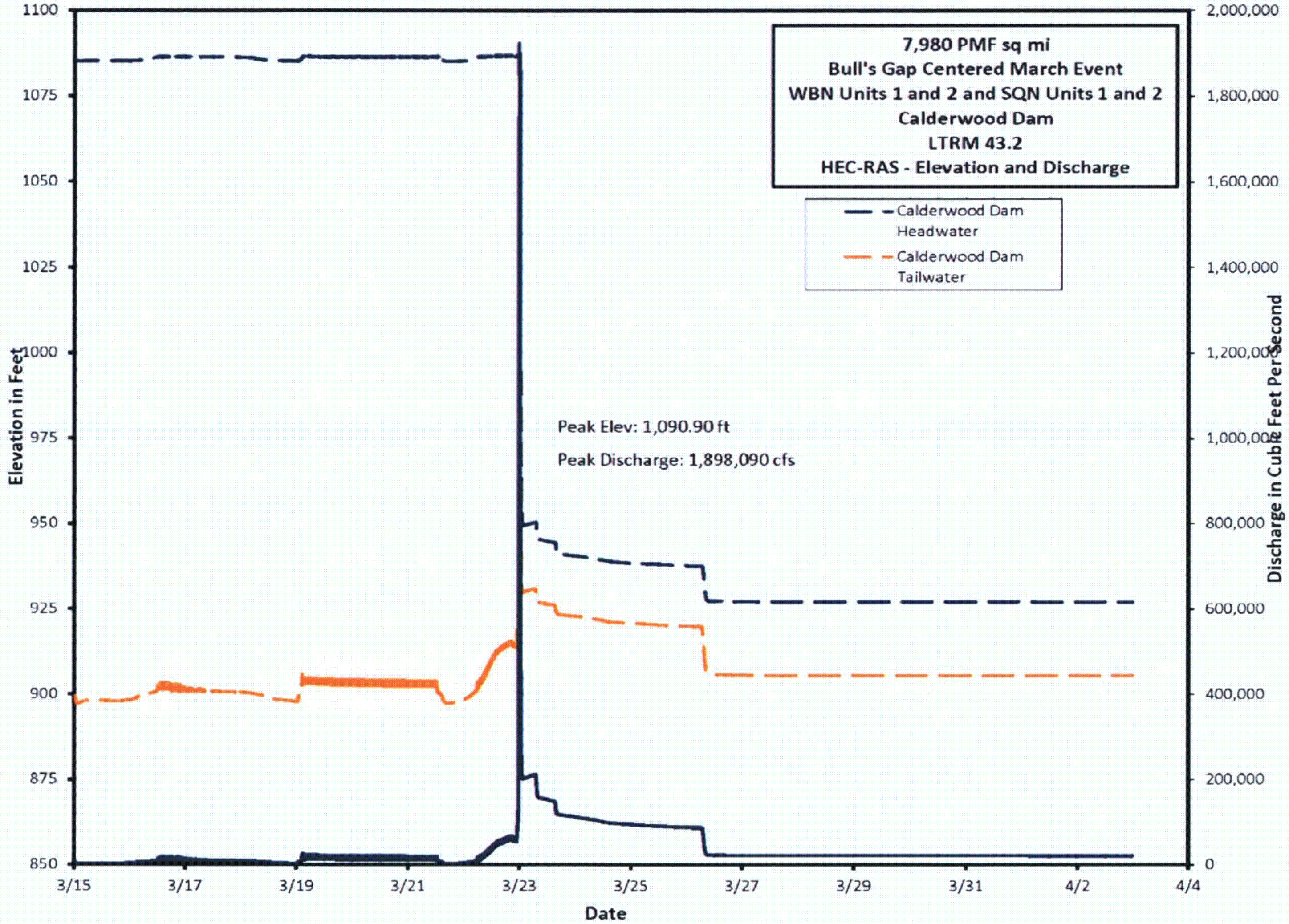


Figure 2.4- 25 Calderwood Dam Hydrograph (Sheet 5 of 27)

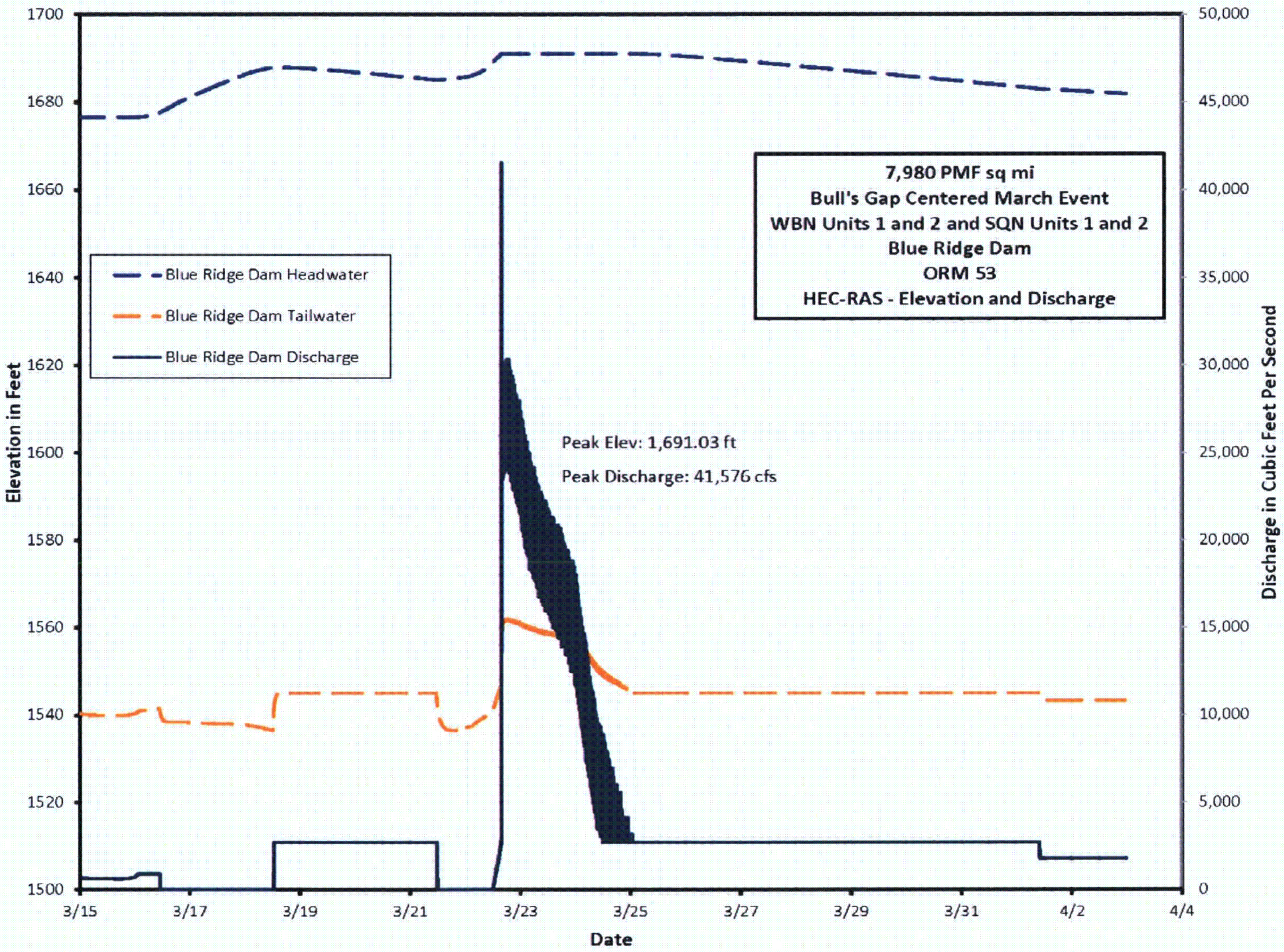


Figure 2.4- 25 Blue Ridge Dam Hydrograph (Sheet 6 of 27)

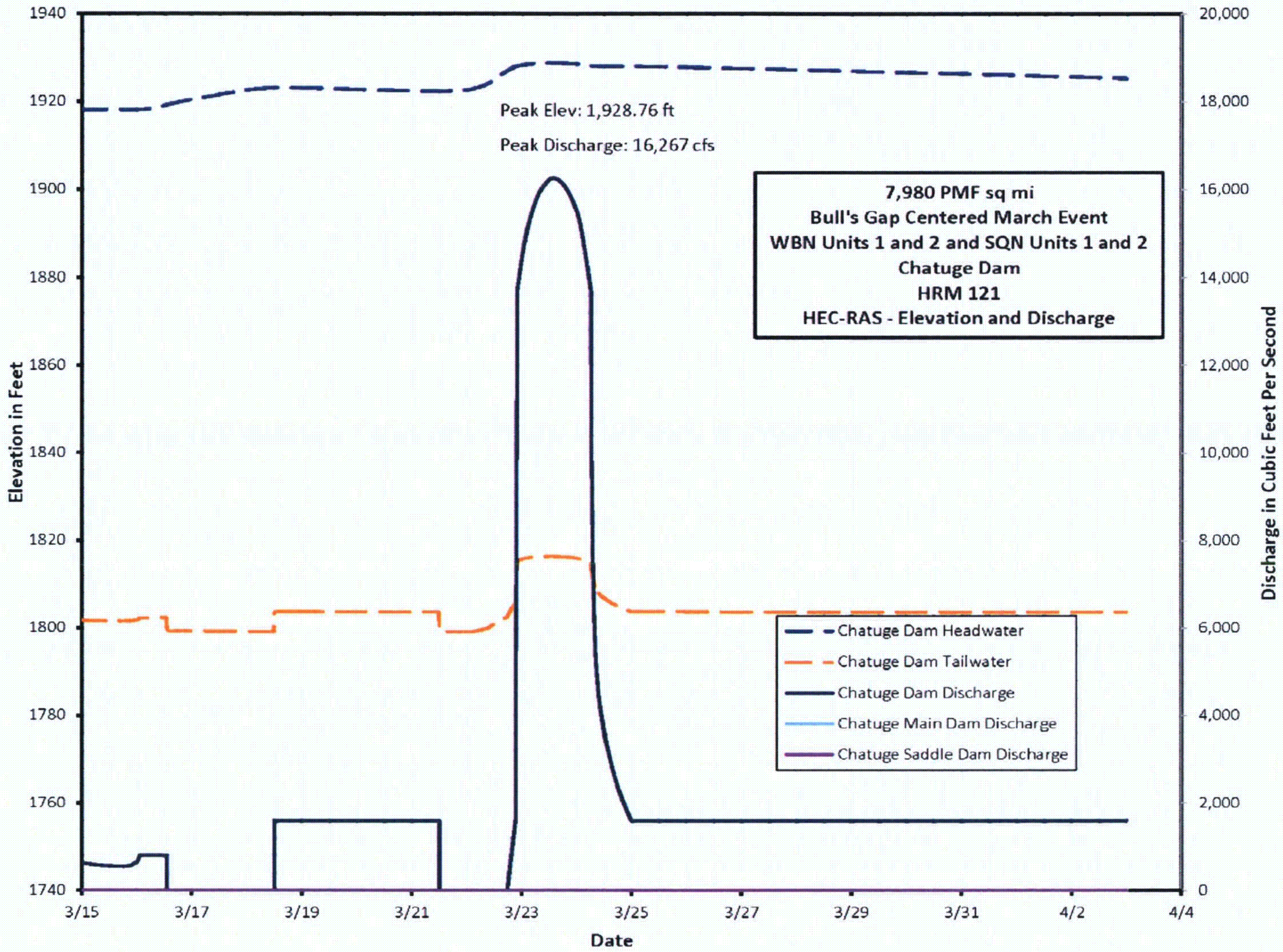


Figure 2.4- 25 Chatuge Dam Hydrograph (Sheet 7 of 27)

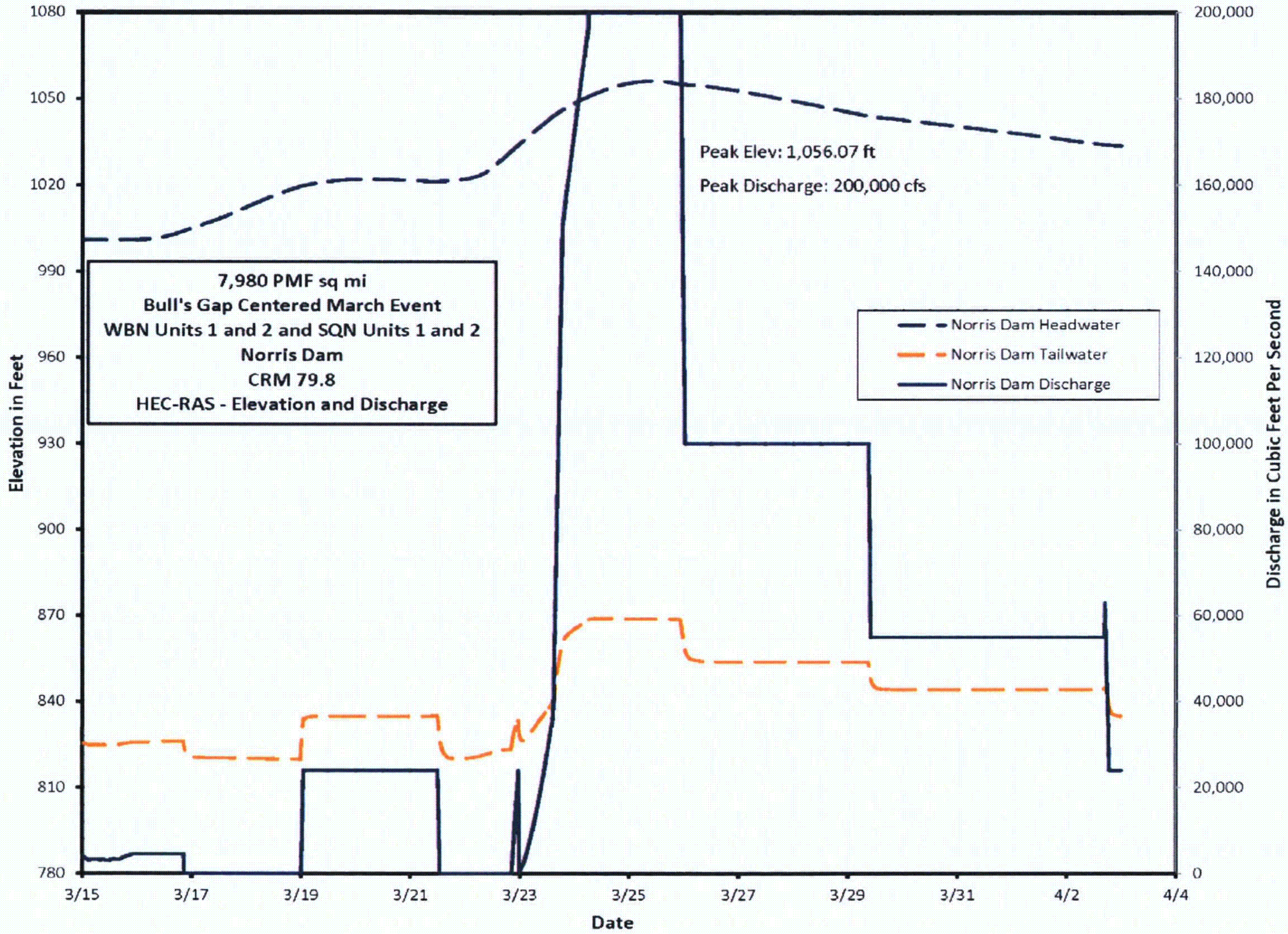


Figure 2.4- 25 Norris Dam Hydrograph (Sheet 8 of 27)

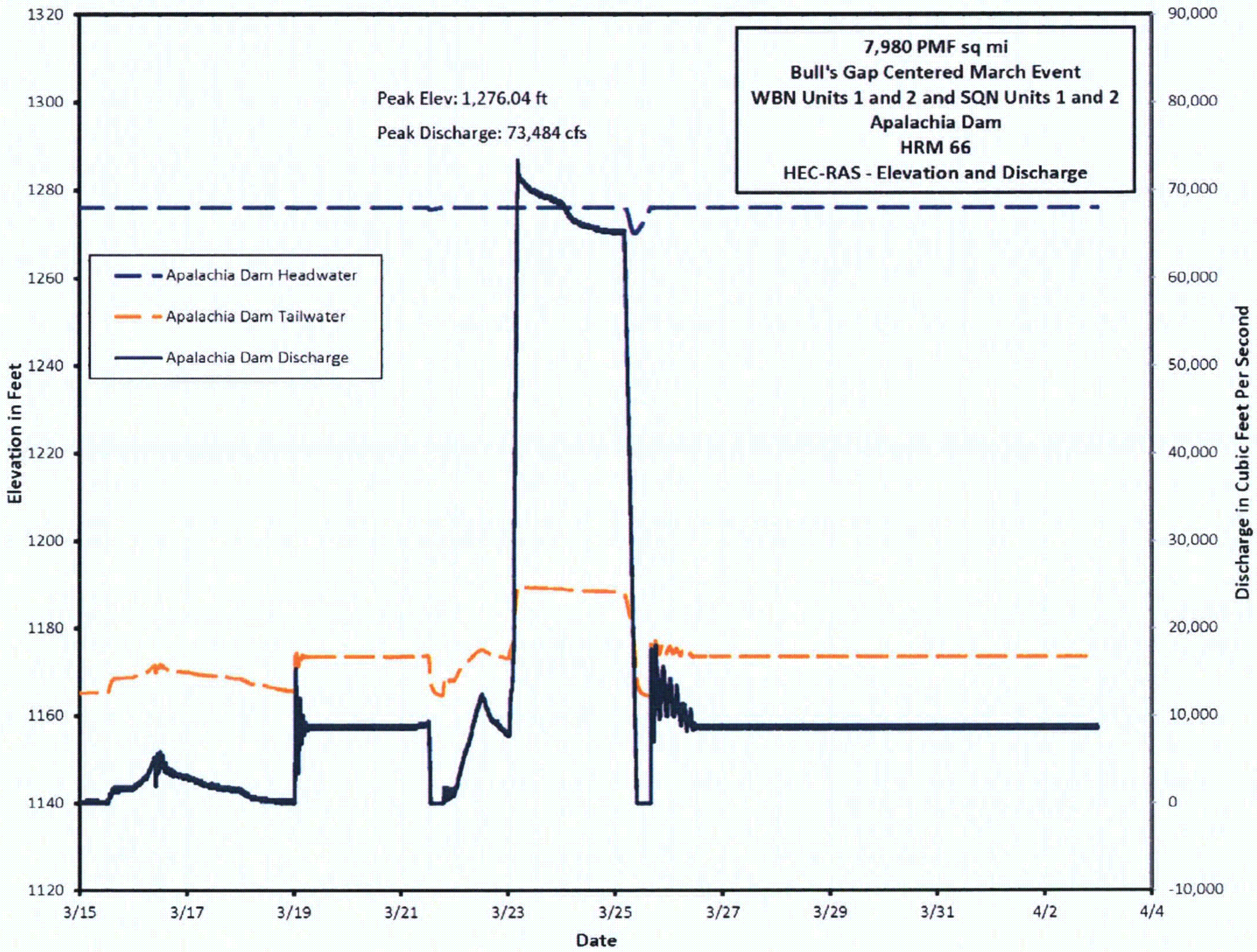


Figure 2.4- 25 Apalachia Dam Hydrograph (Sheet 9 of 27)

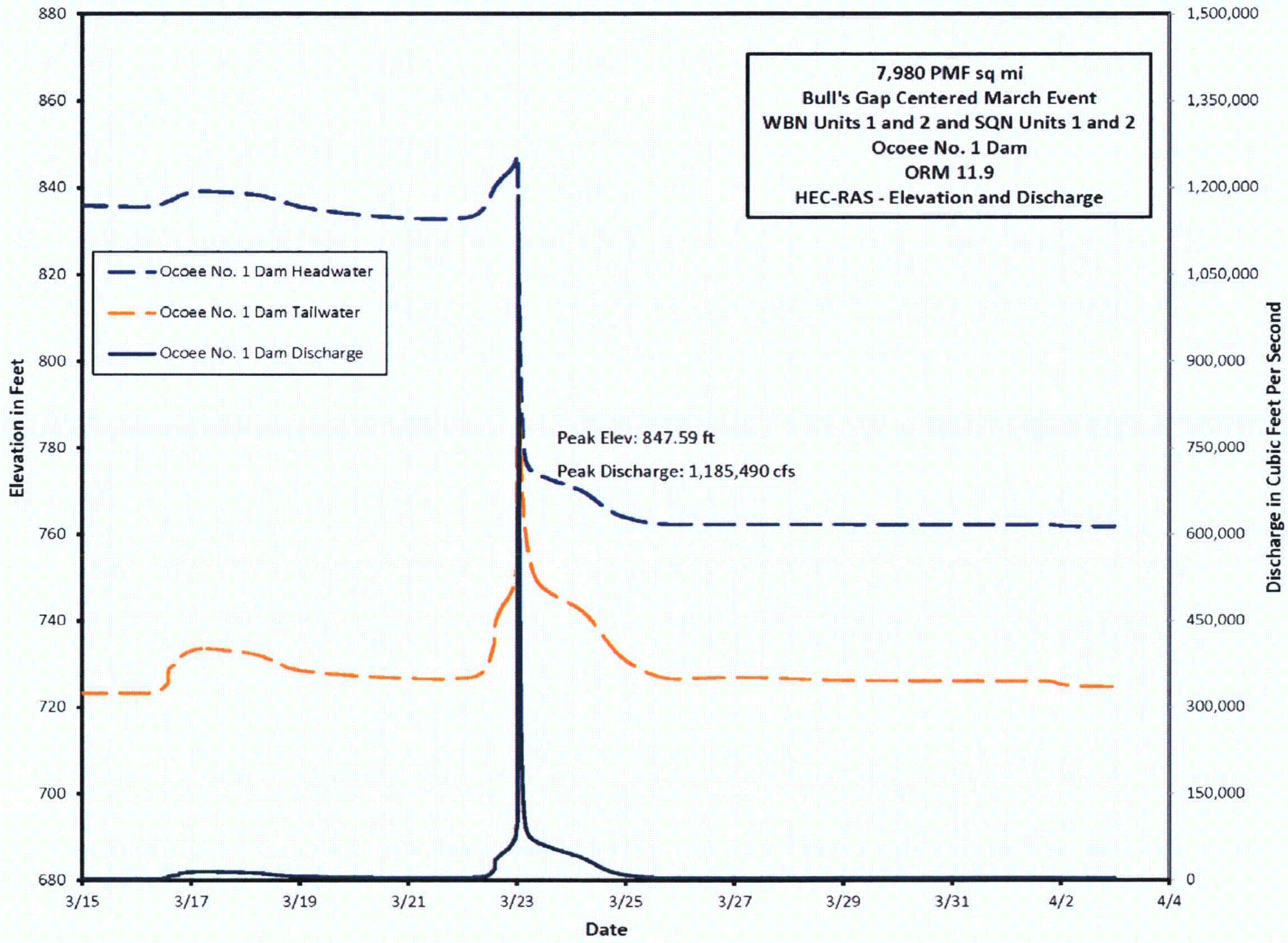


Figure 2.4- 25 Ocoee No. 1 Dam Hydrograph (Sheet 10 of 27)

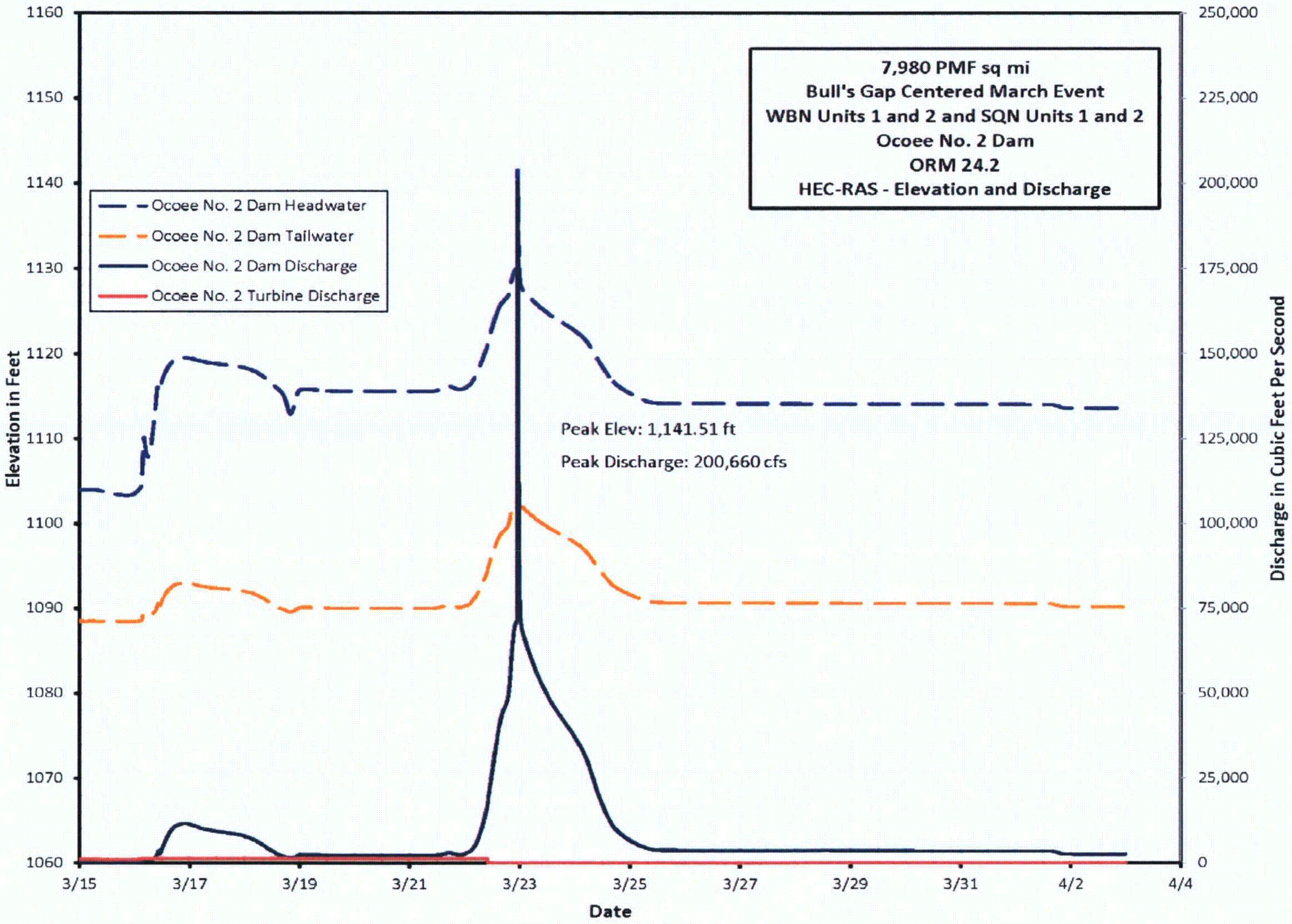


Figure 2.4- 25 Ocoee No. 2 Dam Hydrograph (Sheet 11 of 27)

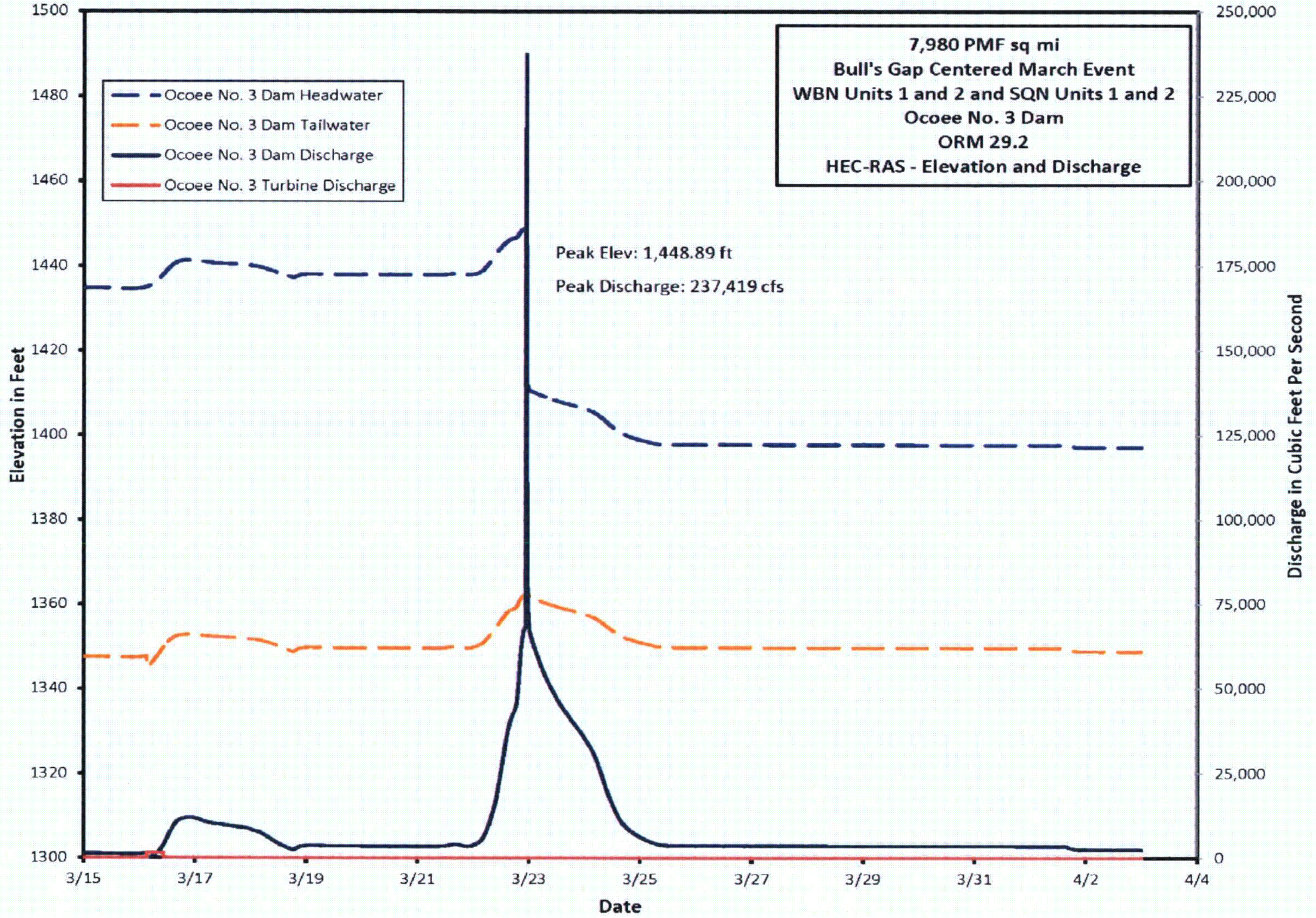


Figure 2.4- 25 Ocoee No. 3 Dam Hydrograph (Sheet 12 of 27)

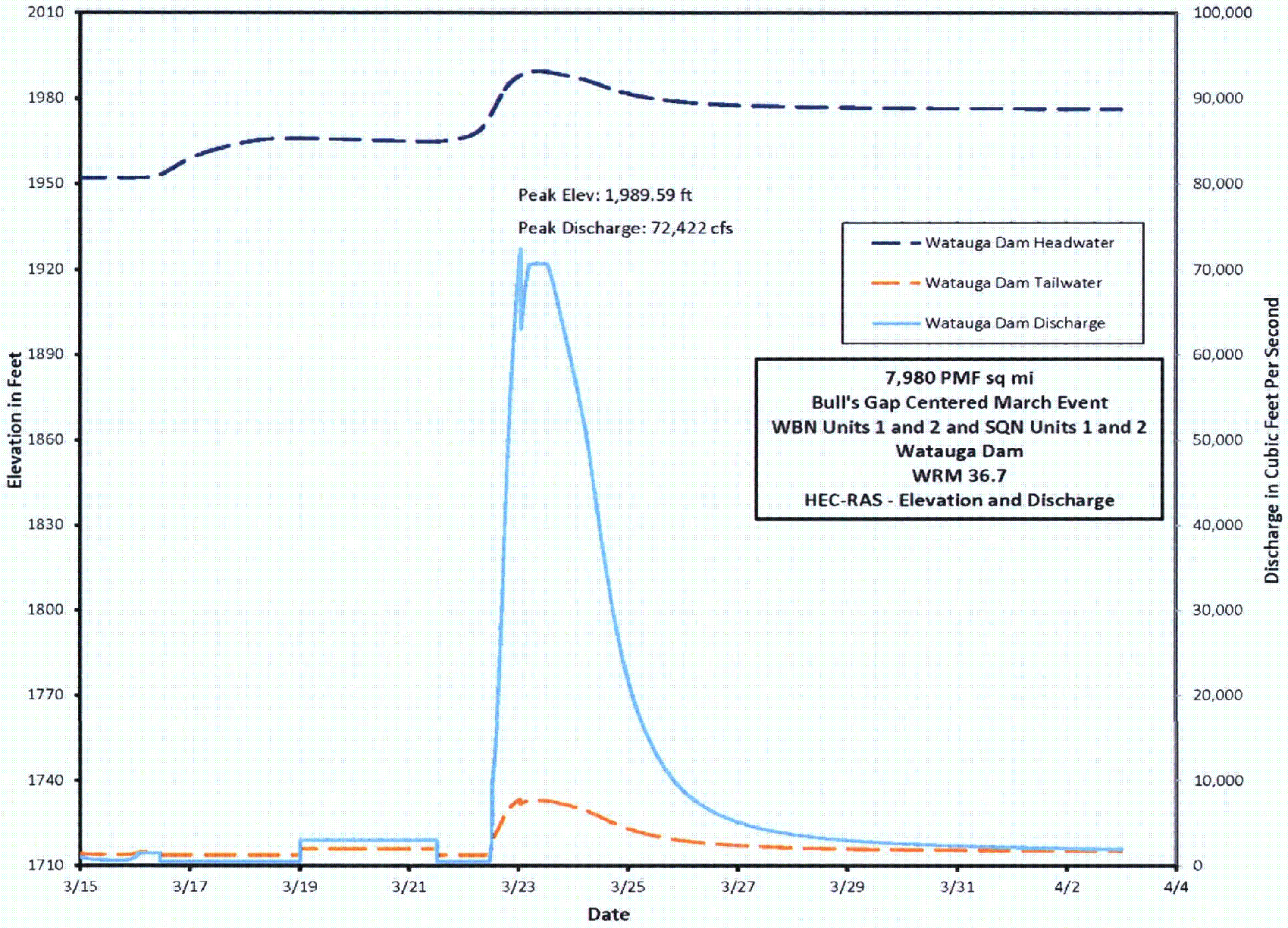


Figure 2.4- 25 Watauga Dam Hydrograph (Sheet 13 of 27)

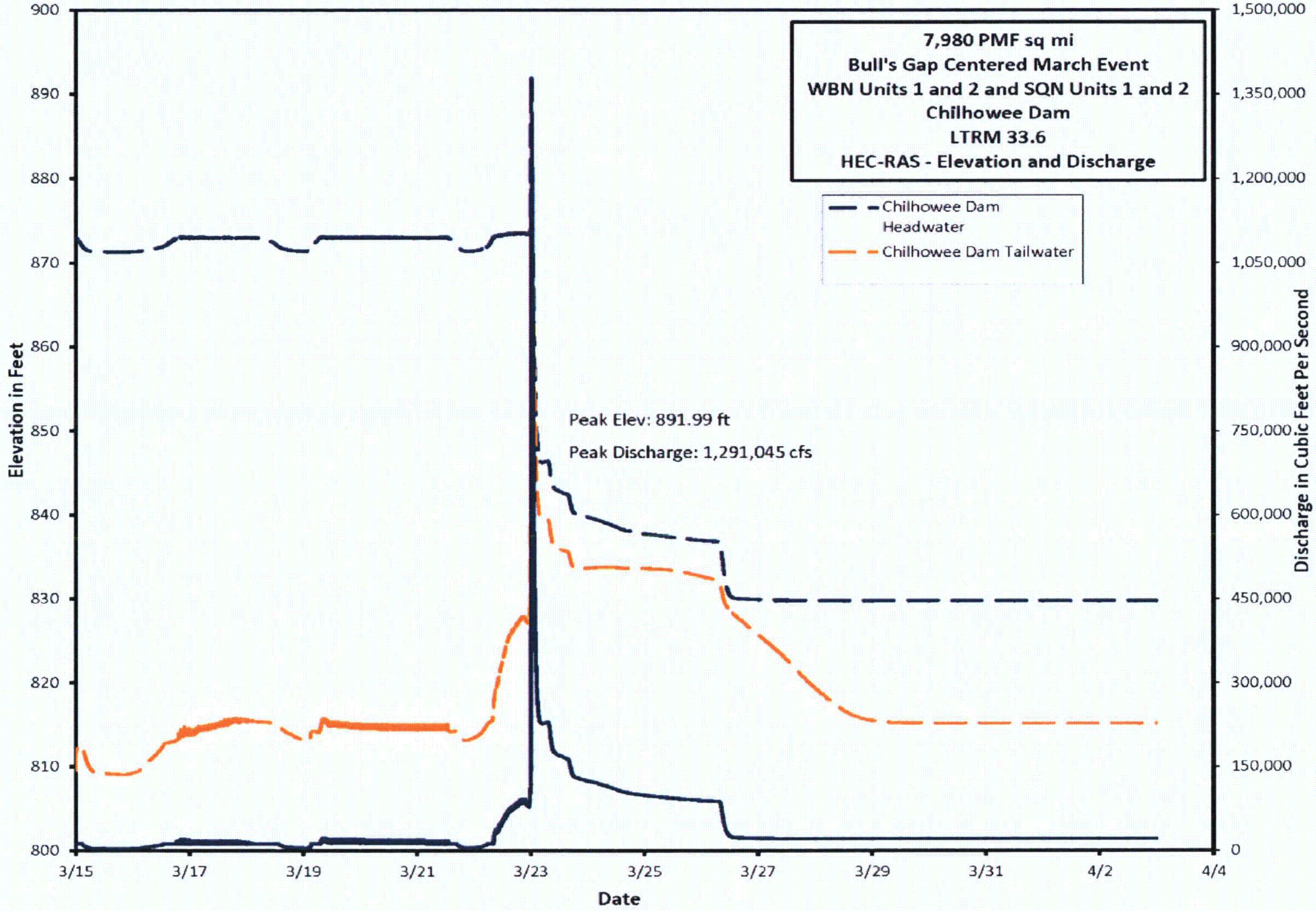


Figure 2.4- 25 Chilowee Dam Hydrograph (Sheet 14 of 27)

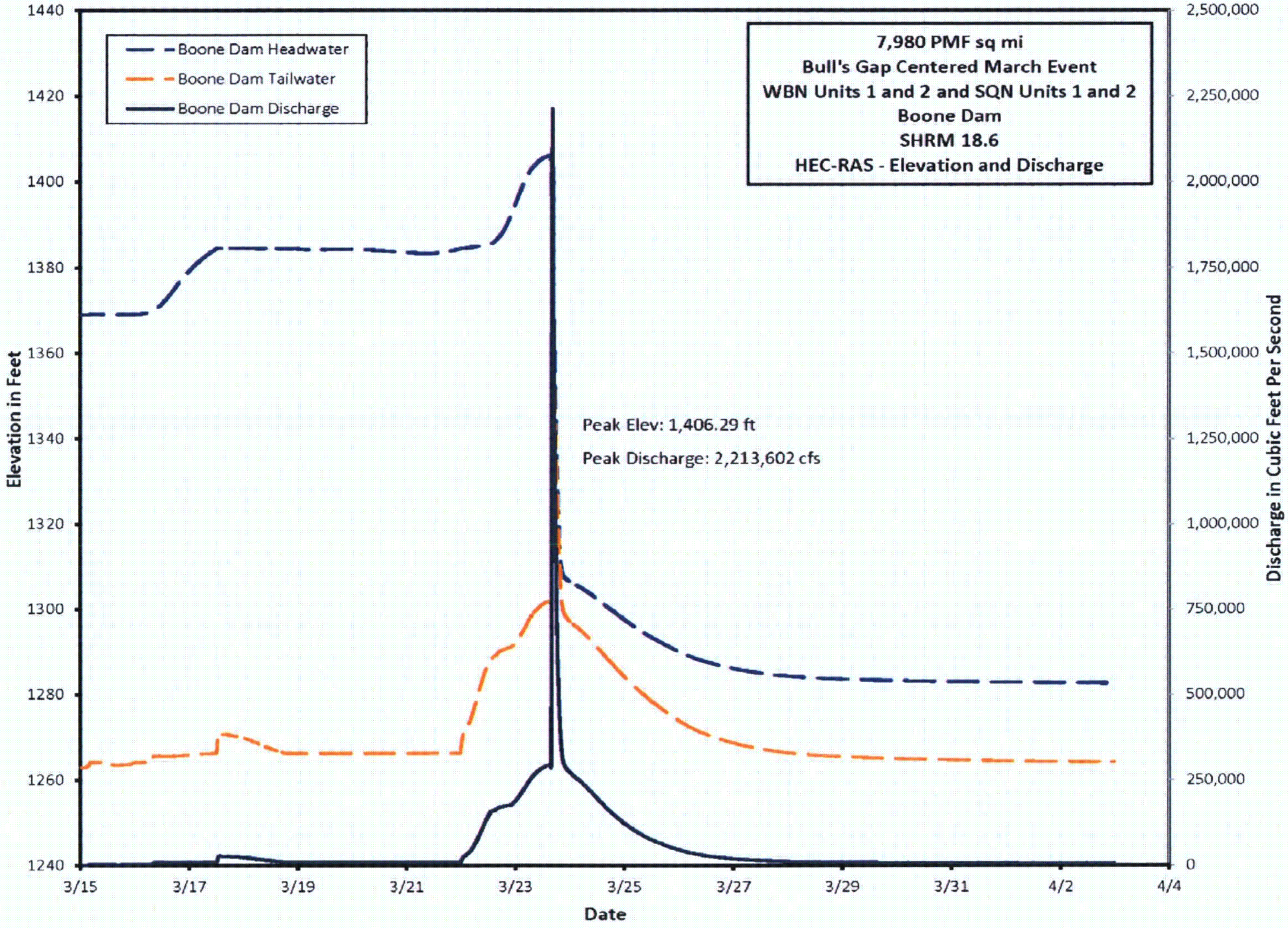


Figure 2.4- 25 Boone Dam Hydrograph (Sheet 15 of 27)

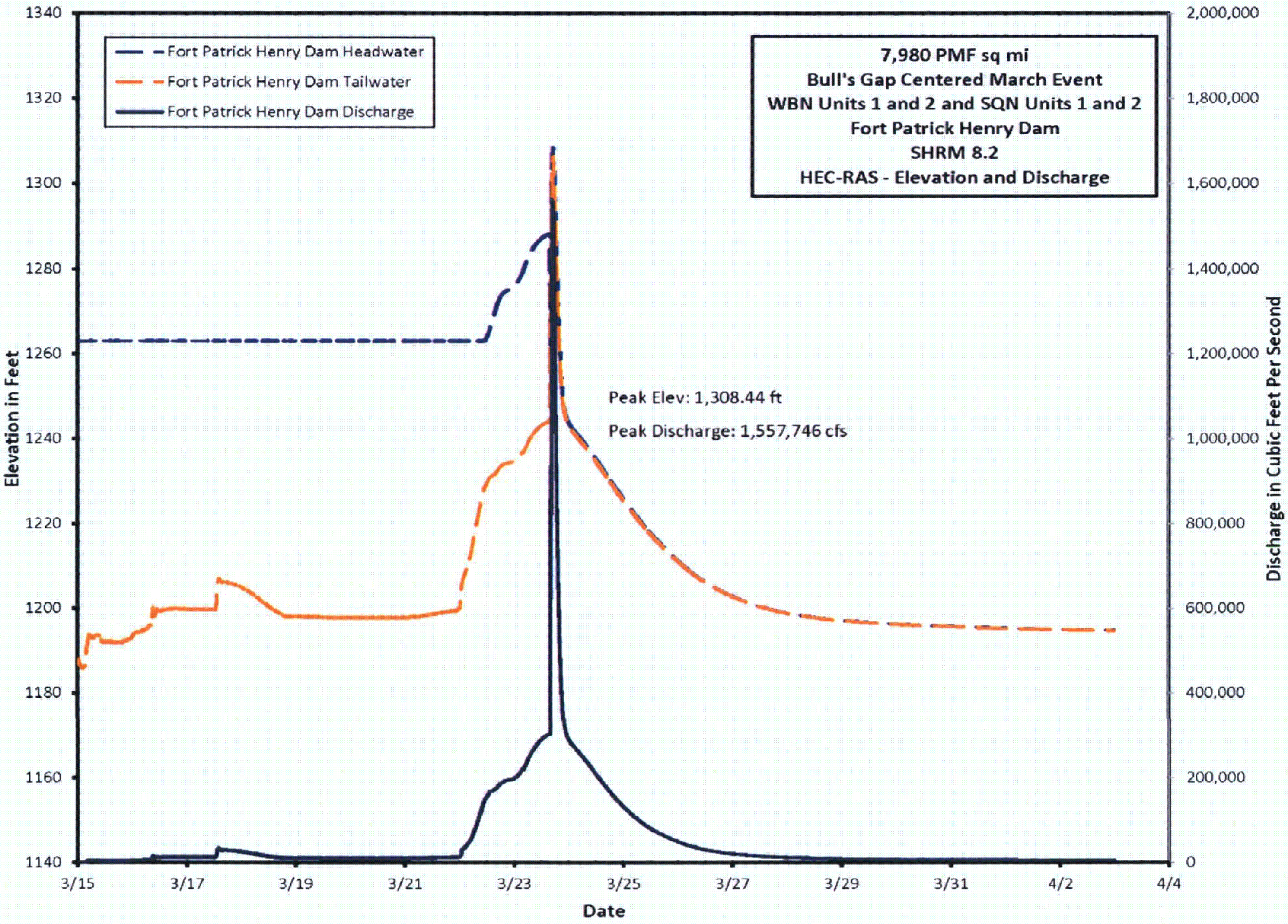


Figure 2.4- 25 Fort Patrick Henry Dam Hydrograph (Sheet 16 of 27)

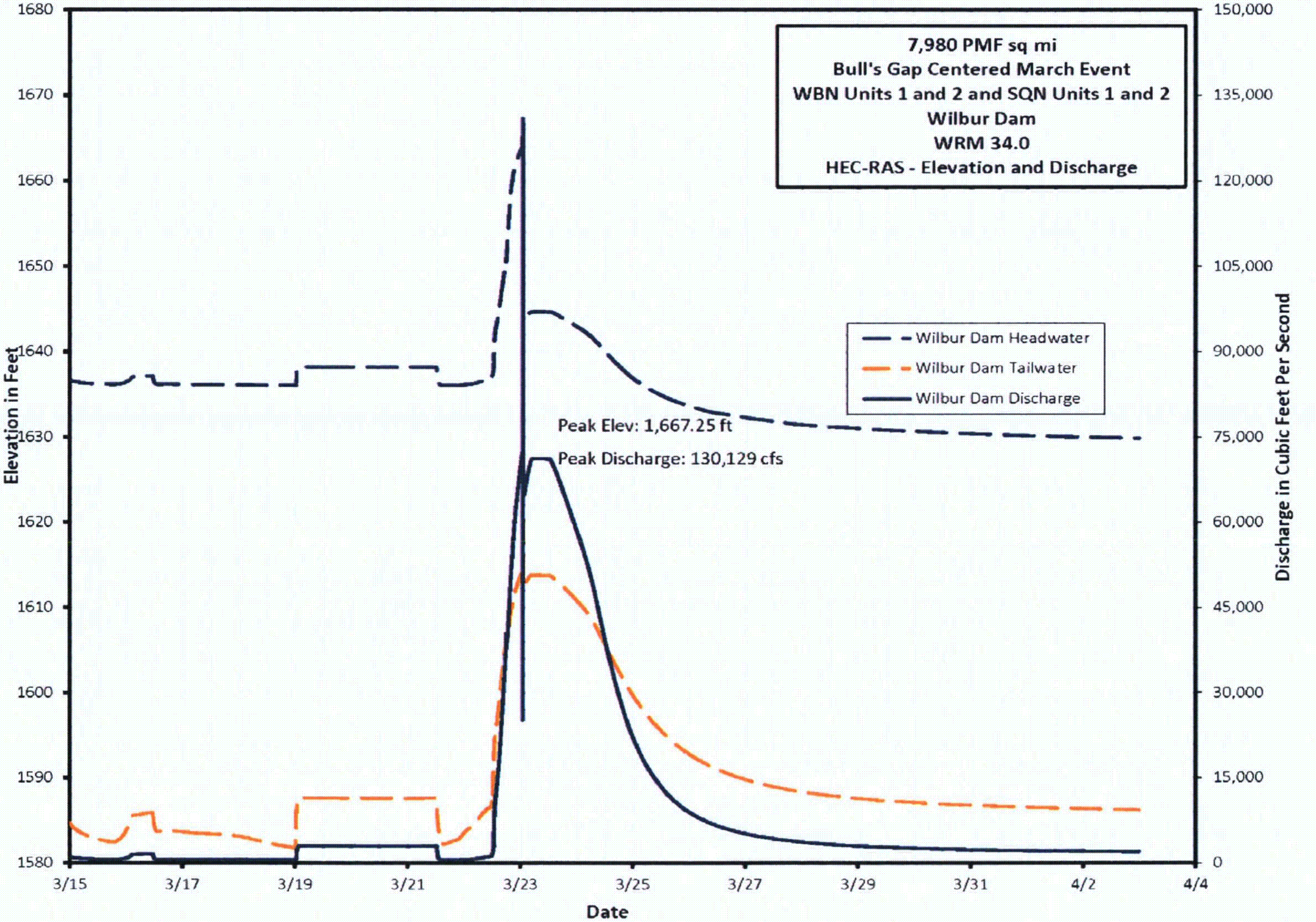


Figure 2.4- 25 Wilbur Dam Hydrograph (Sheet 17 of 27)

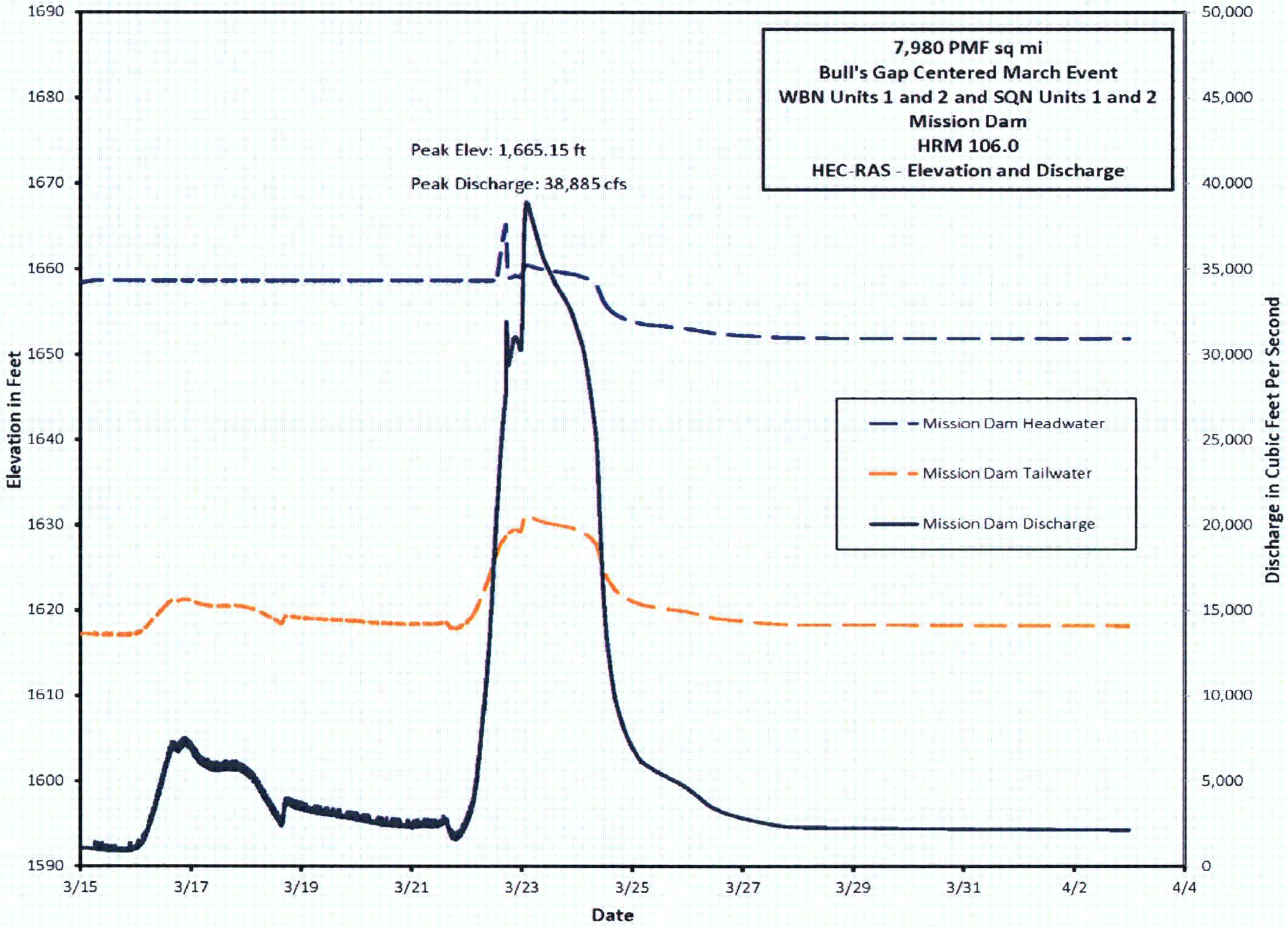


Figure 2.4- 25 Mission Dam Hydrograph (Sheet 18 of 27)

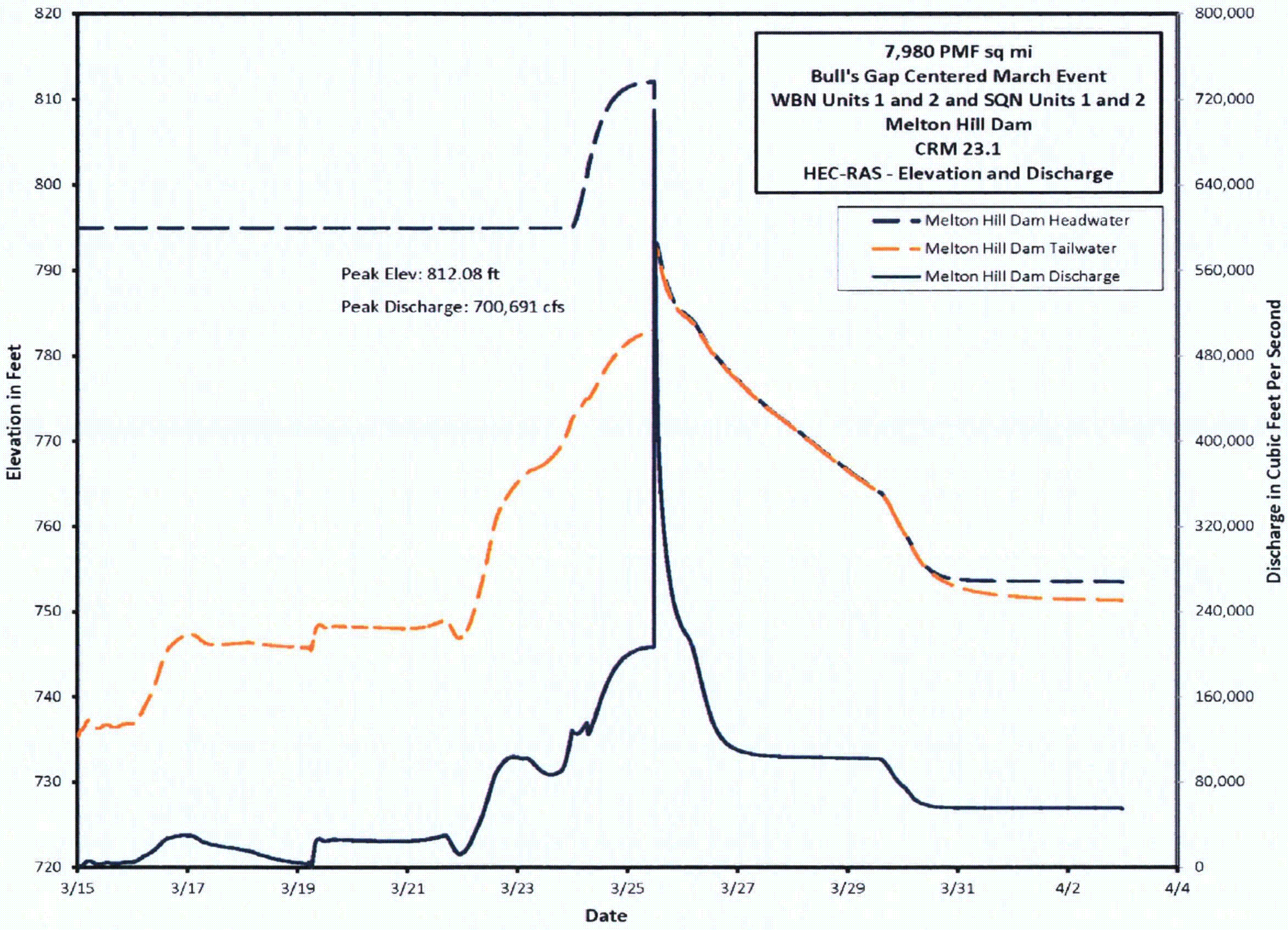


Figure 2.4- 25 Melton Hill Dam Hydrograph (Sheet 19 of 27)

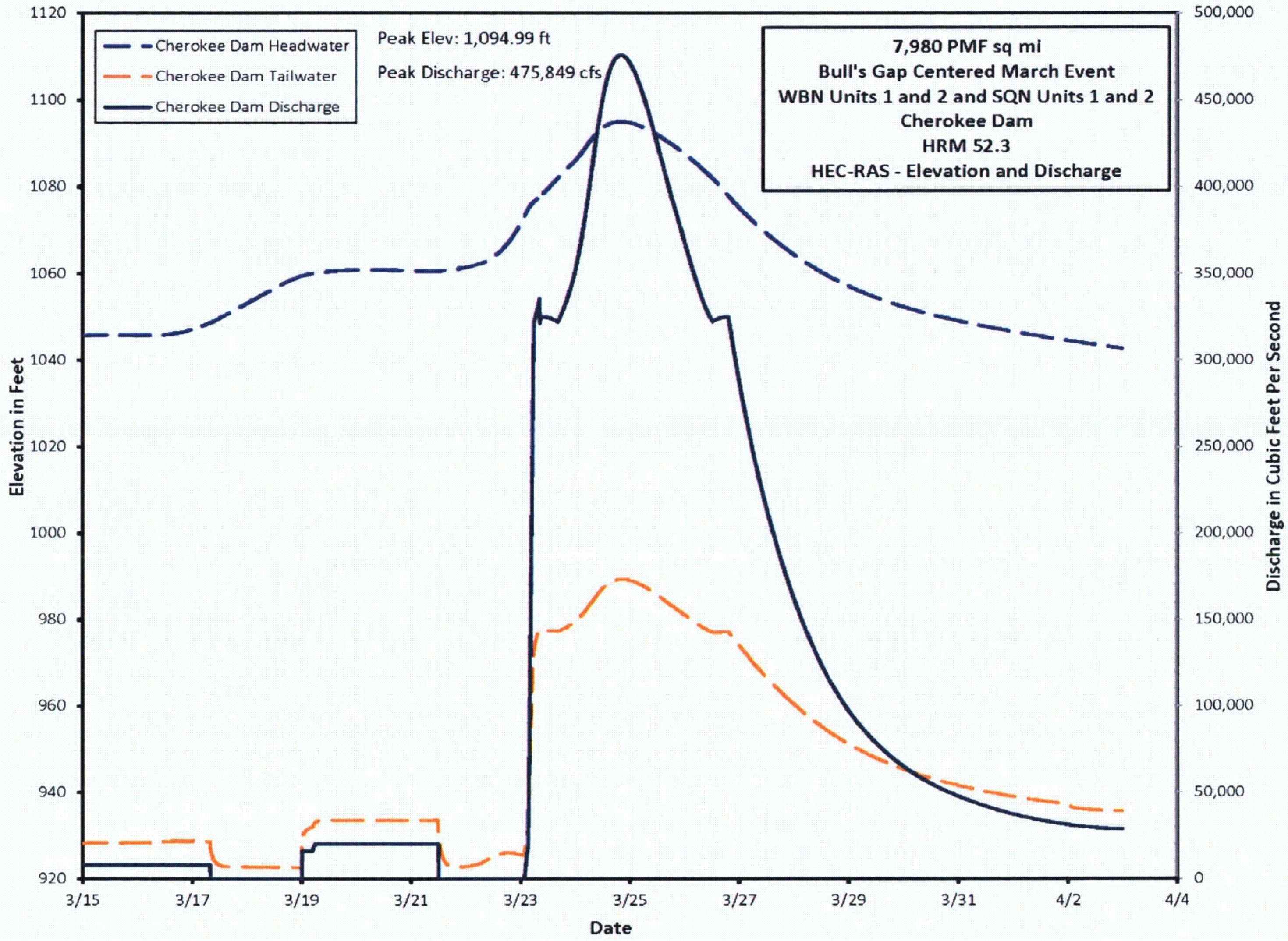


Figure 2.4- 25 Cherokee Dam Hydrograph (Sheet 20 of 27)

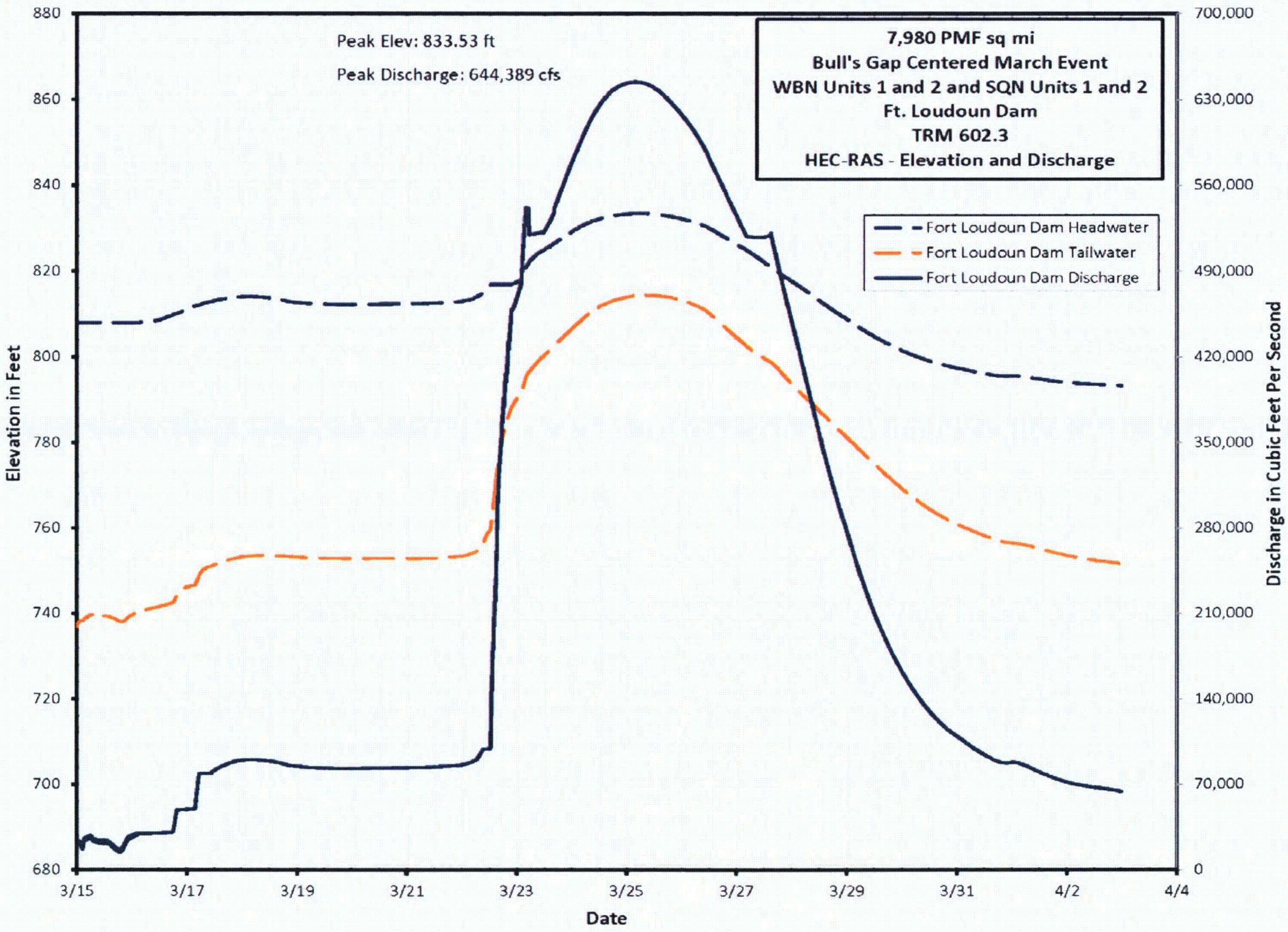


Figure 2.4- 25 Ft Loudoun Dam Hydrograph (Sheet 21 of 27)

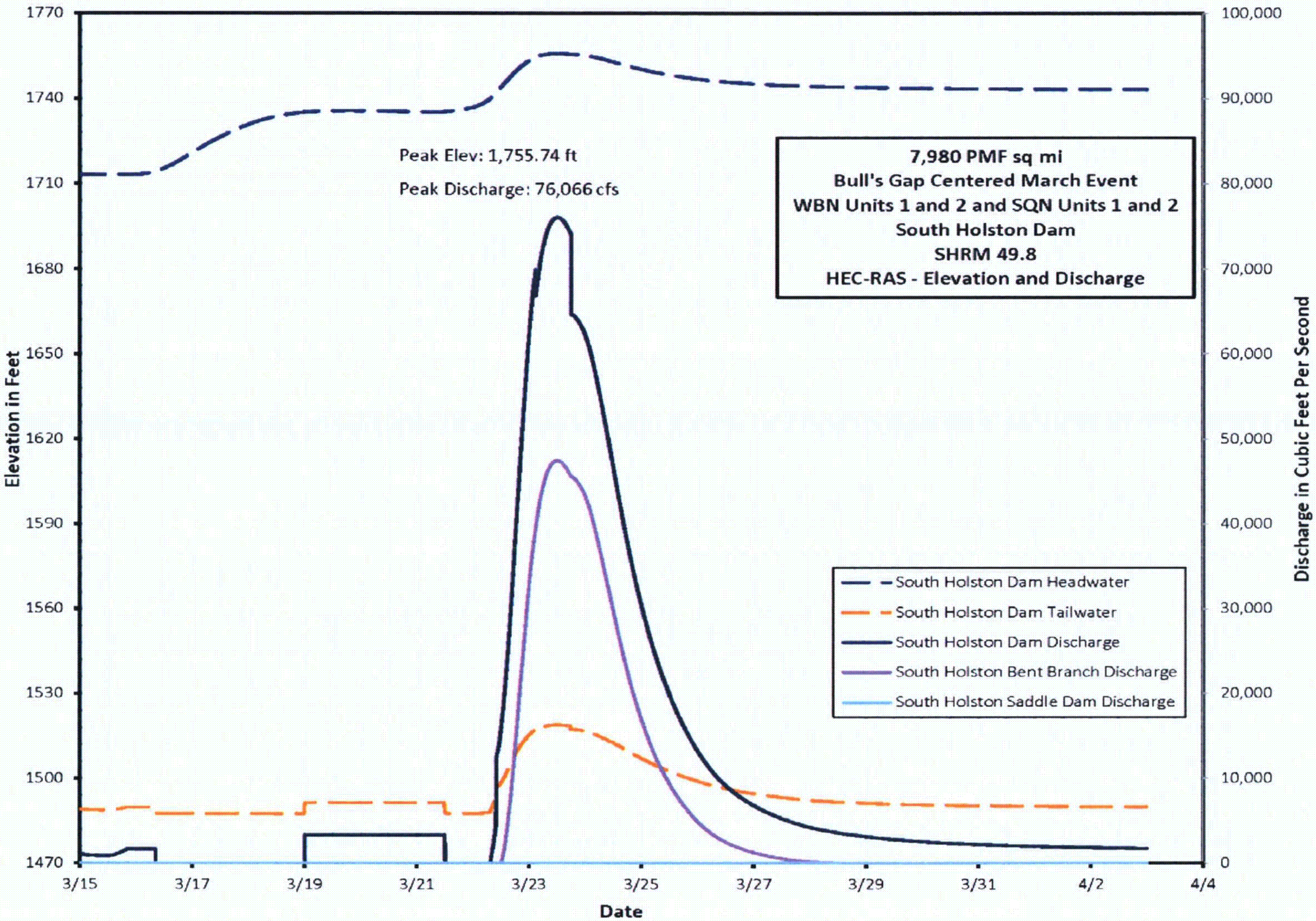


Figure 2.4- 25 South Holston Dam Hydrograph (Sheet 22 of 27)

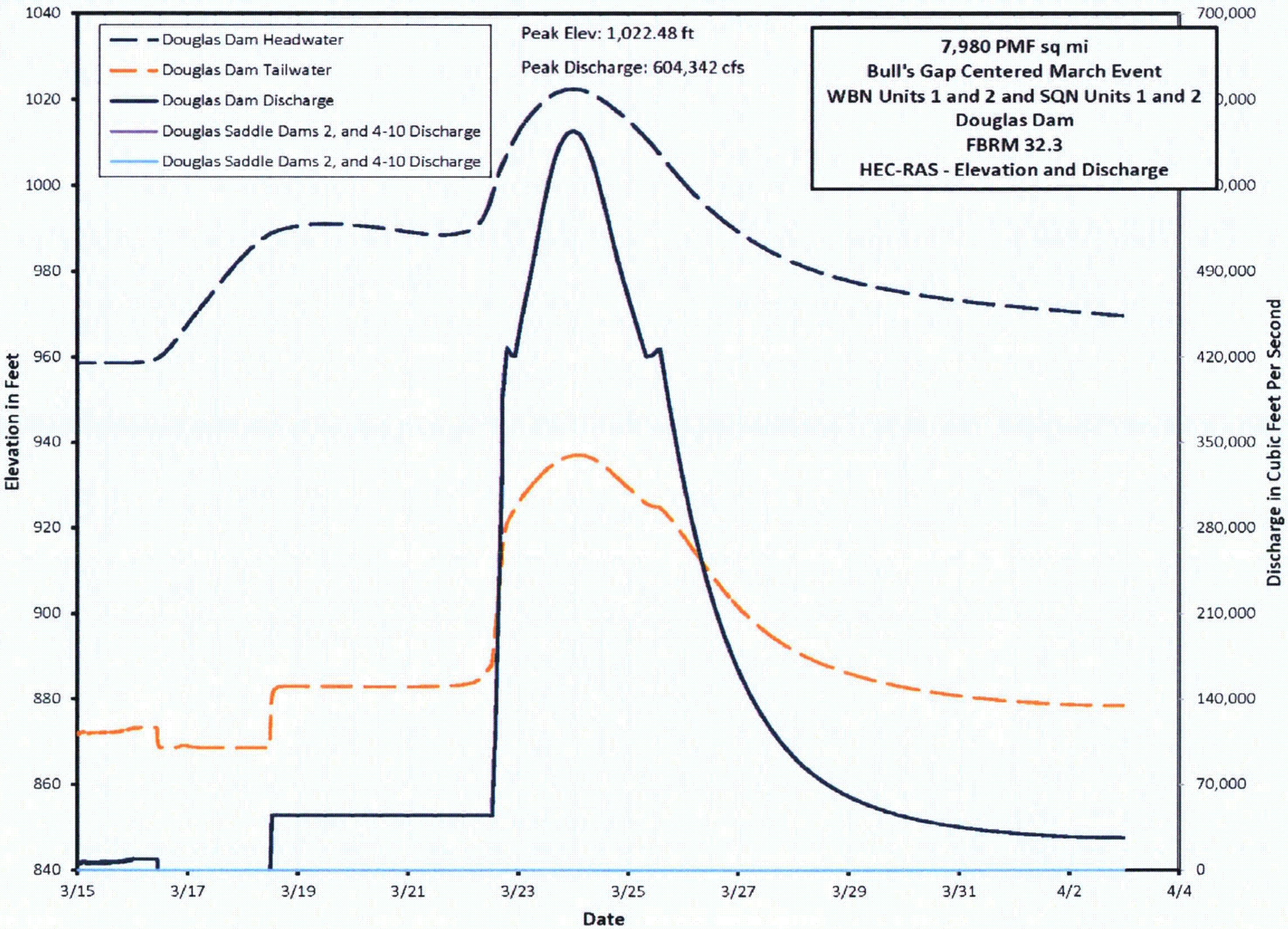


Figure 2.4- 25 Douglas Dam Hydrograph (Sheet 23 of 27)

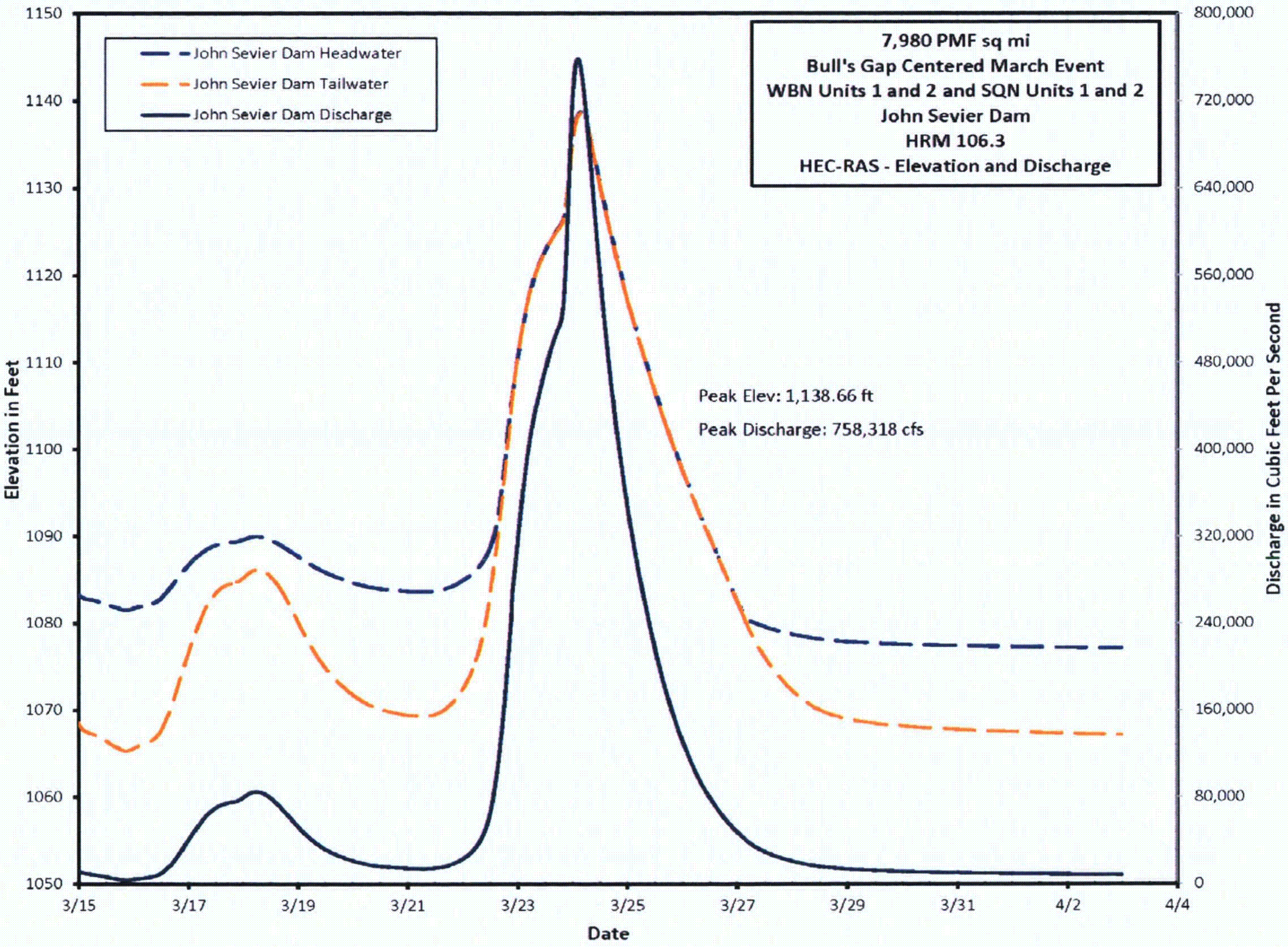


Figure 2.4- 25 John Sevier Dam Hydrograph (Sheet 24 of 27)

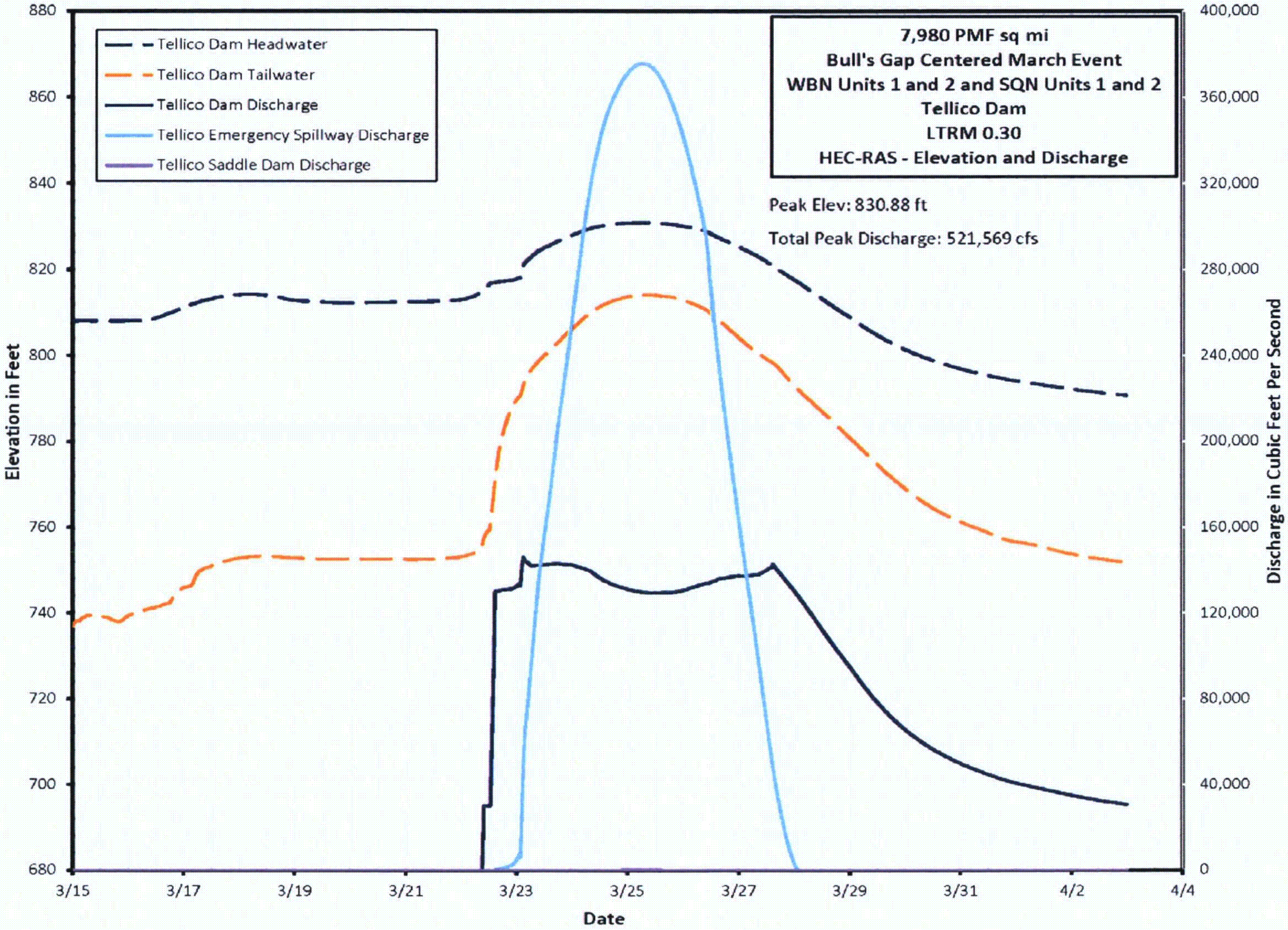


Figure 2.4- 25 Tellico Dam Hydrograph (Sheet 25 of 27)

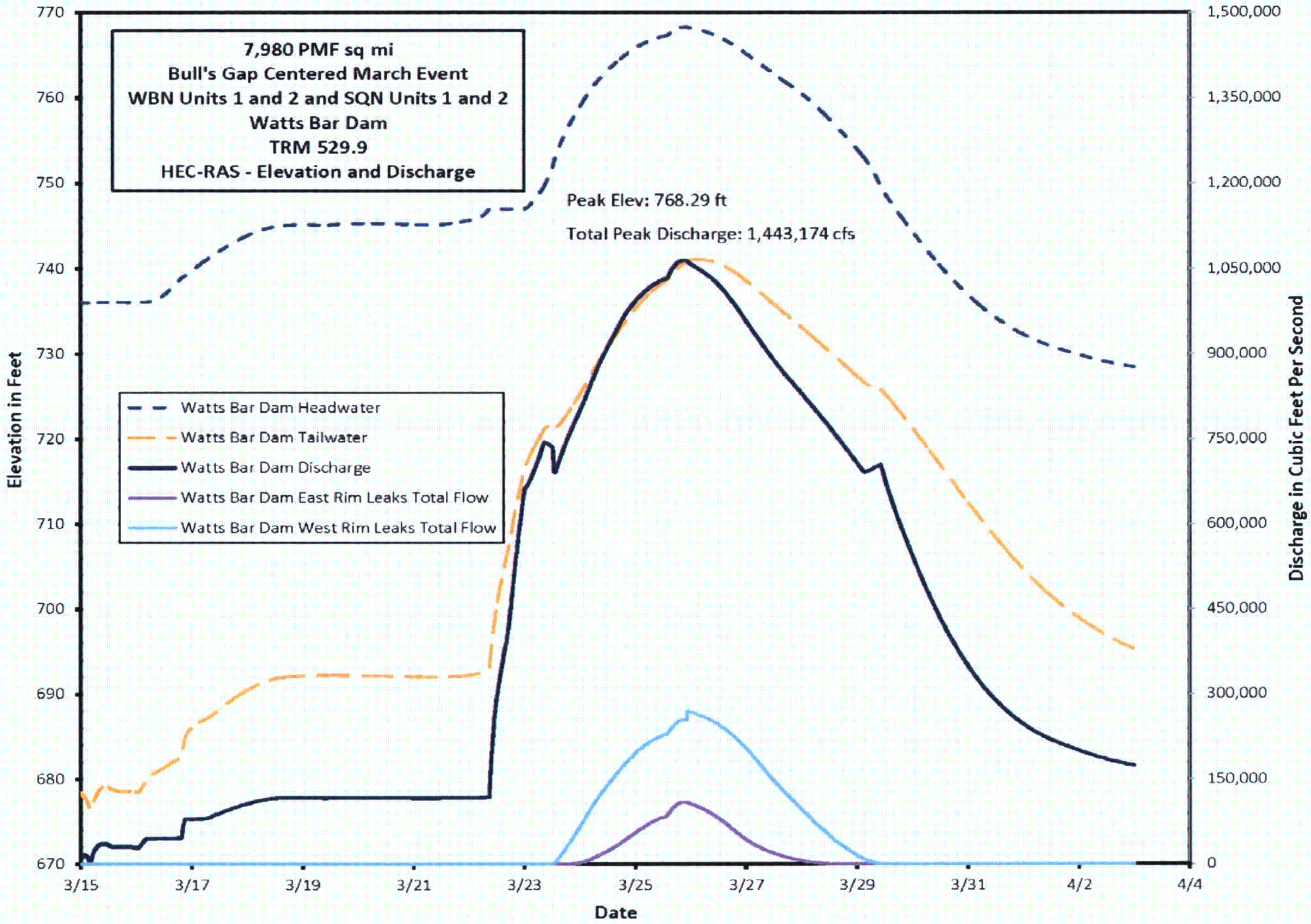


Figure 2.4- 25 Watts Bar Dam Hydrograph (Sheet 26 of 27)

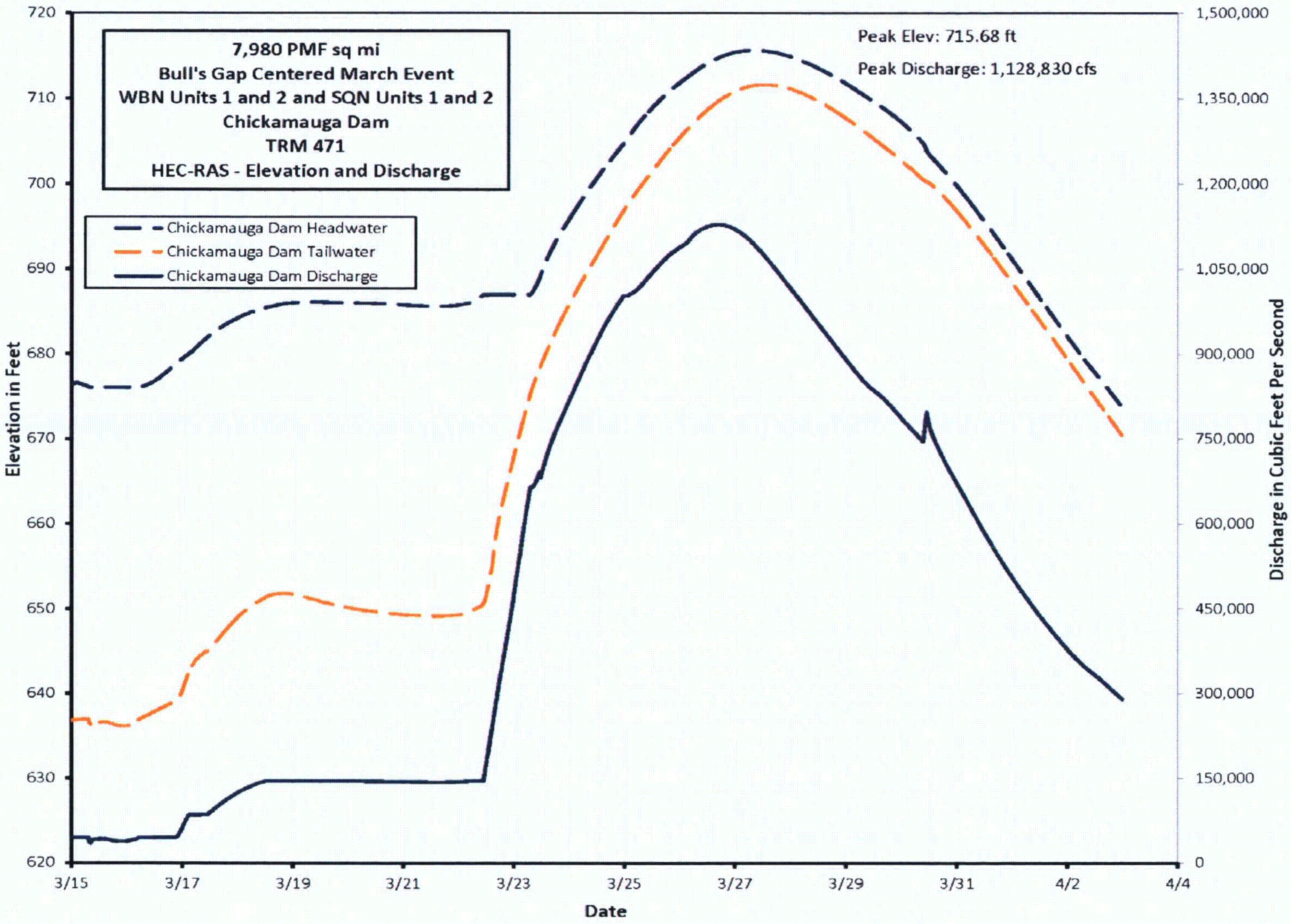


Figure 2.4- 25 Chickamauga Dam Hydrograph (Sheet 27 of 27)

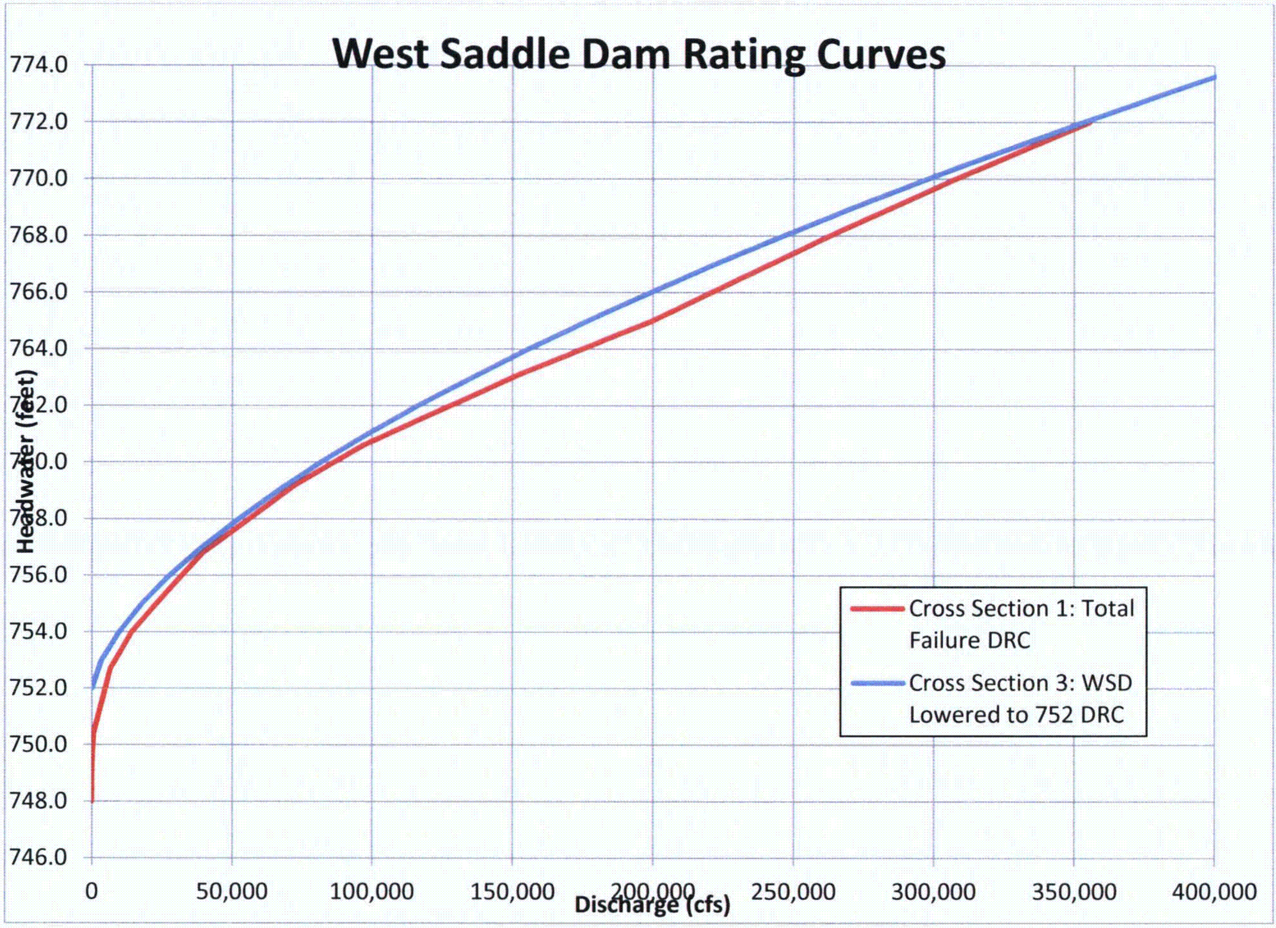


Figure 2.4- 31 West Saddle Dam Discharge Rating Curves

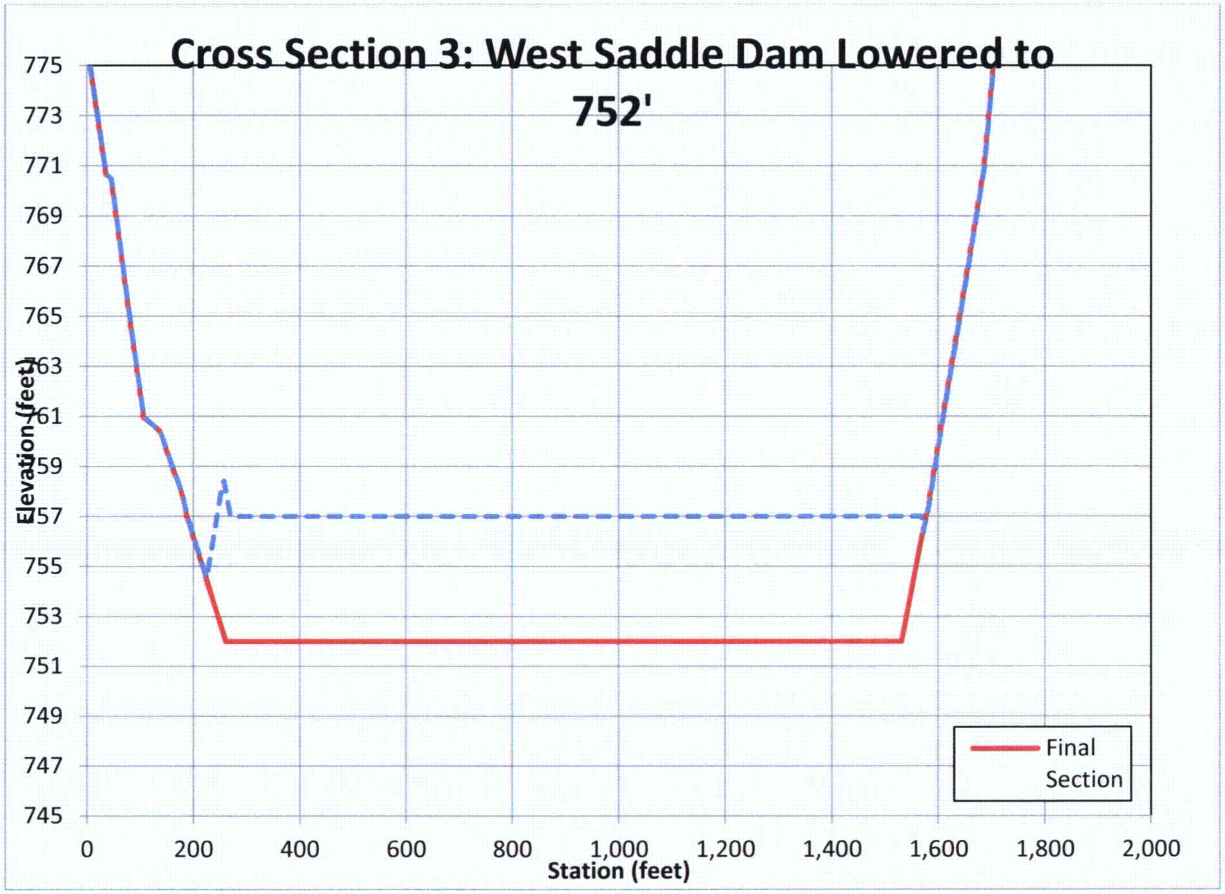


Figure 2.4- 30 West Saddle Dike Cross Section #3

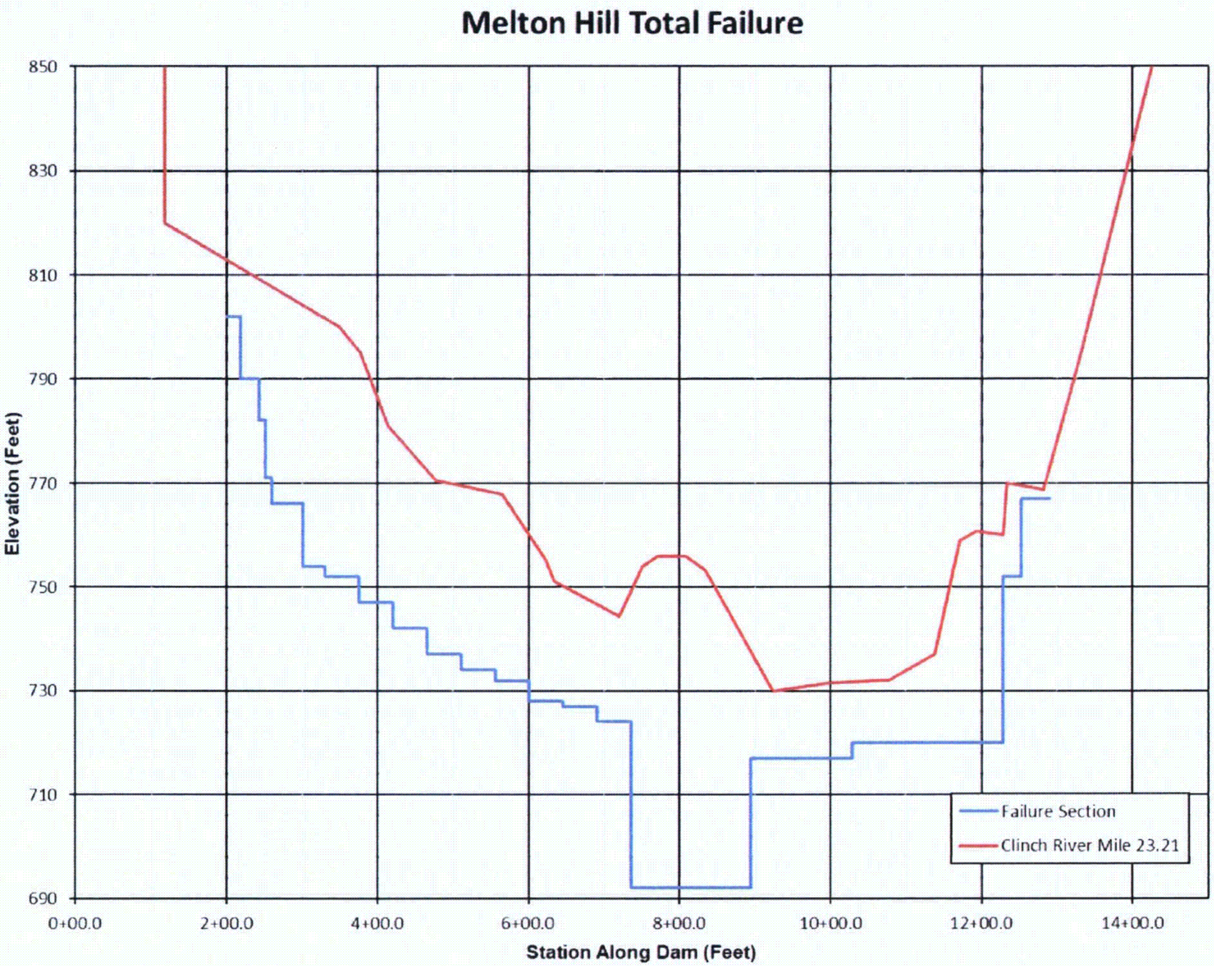


Figure 2.4-32 Melton Hill Total Failure Plot (Sheet 1 of 4)

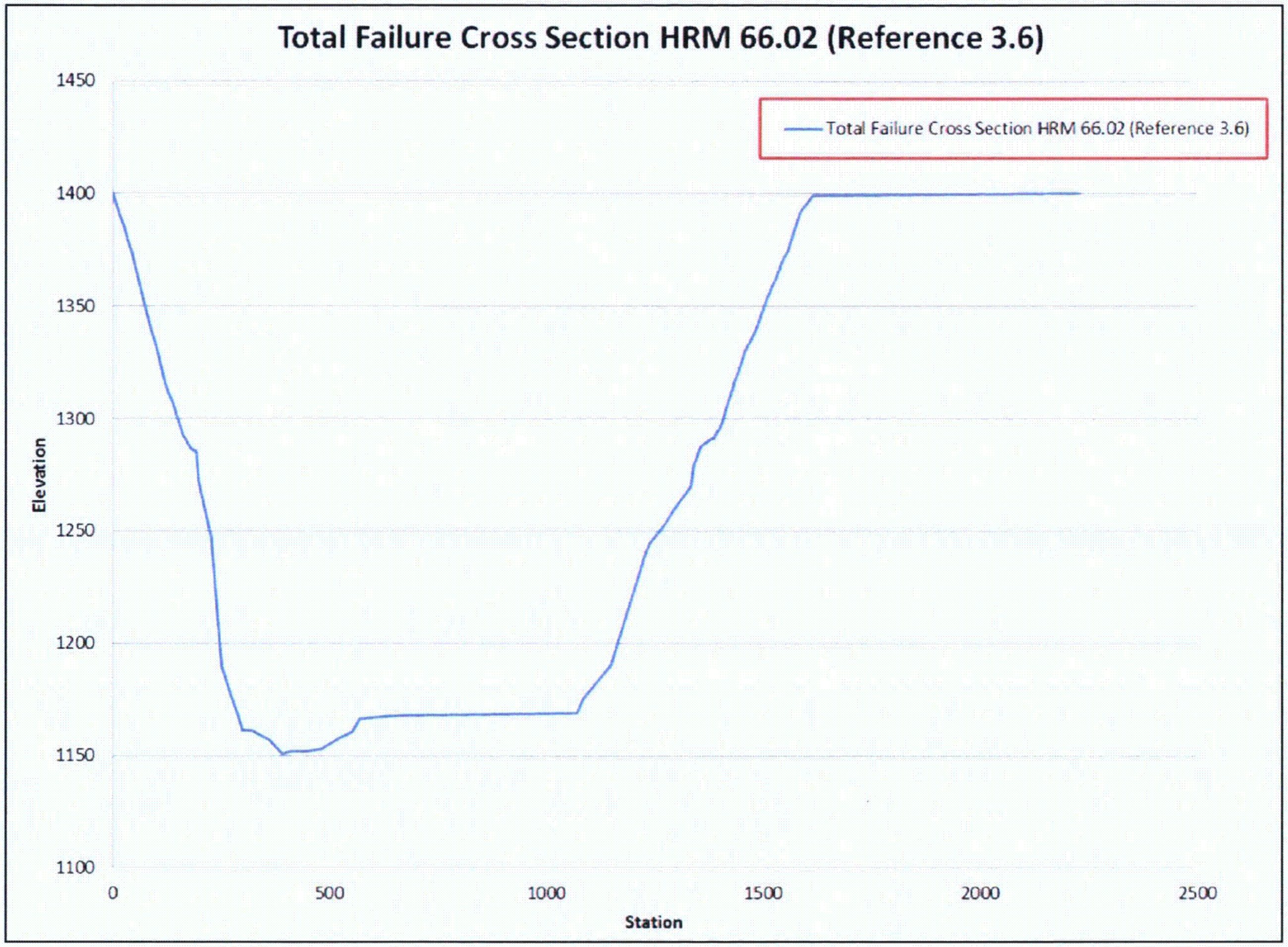


Figure 2.4-32 Total Failure Cross Section for Apalachia Dam Failure (Sheet 2 of 4)

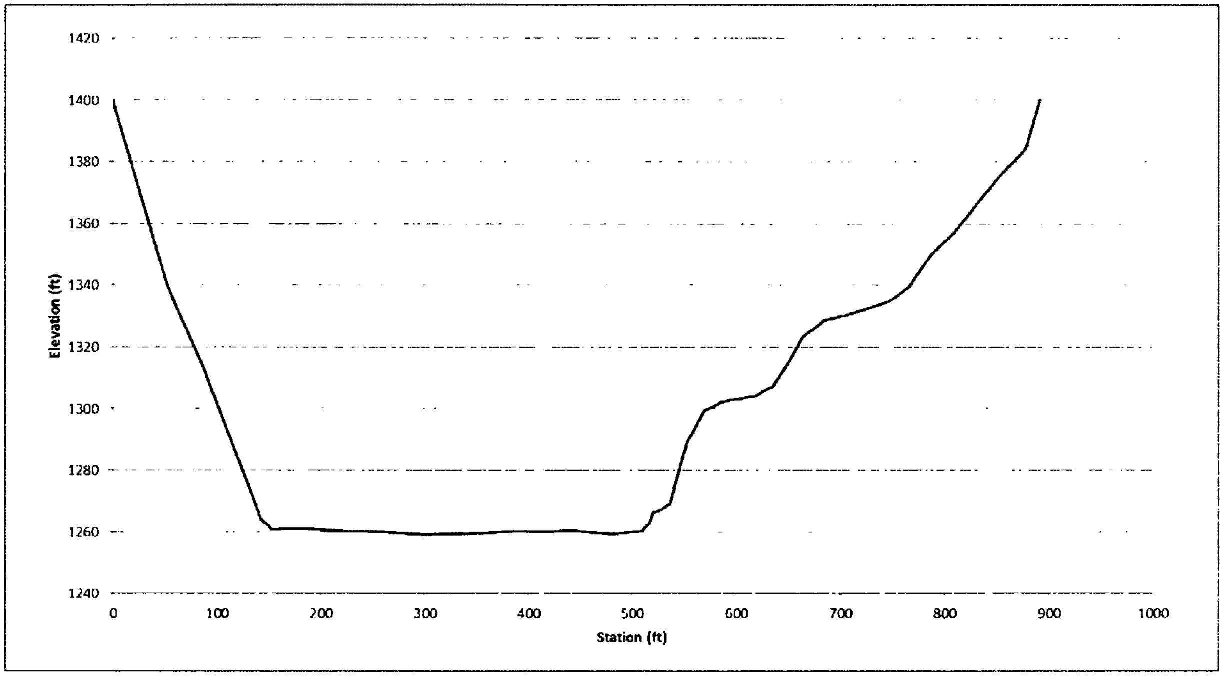


Figure 2.4-32 Boone Total Failure Section (Sheet 3 of 4)

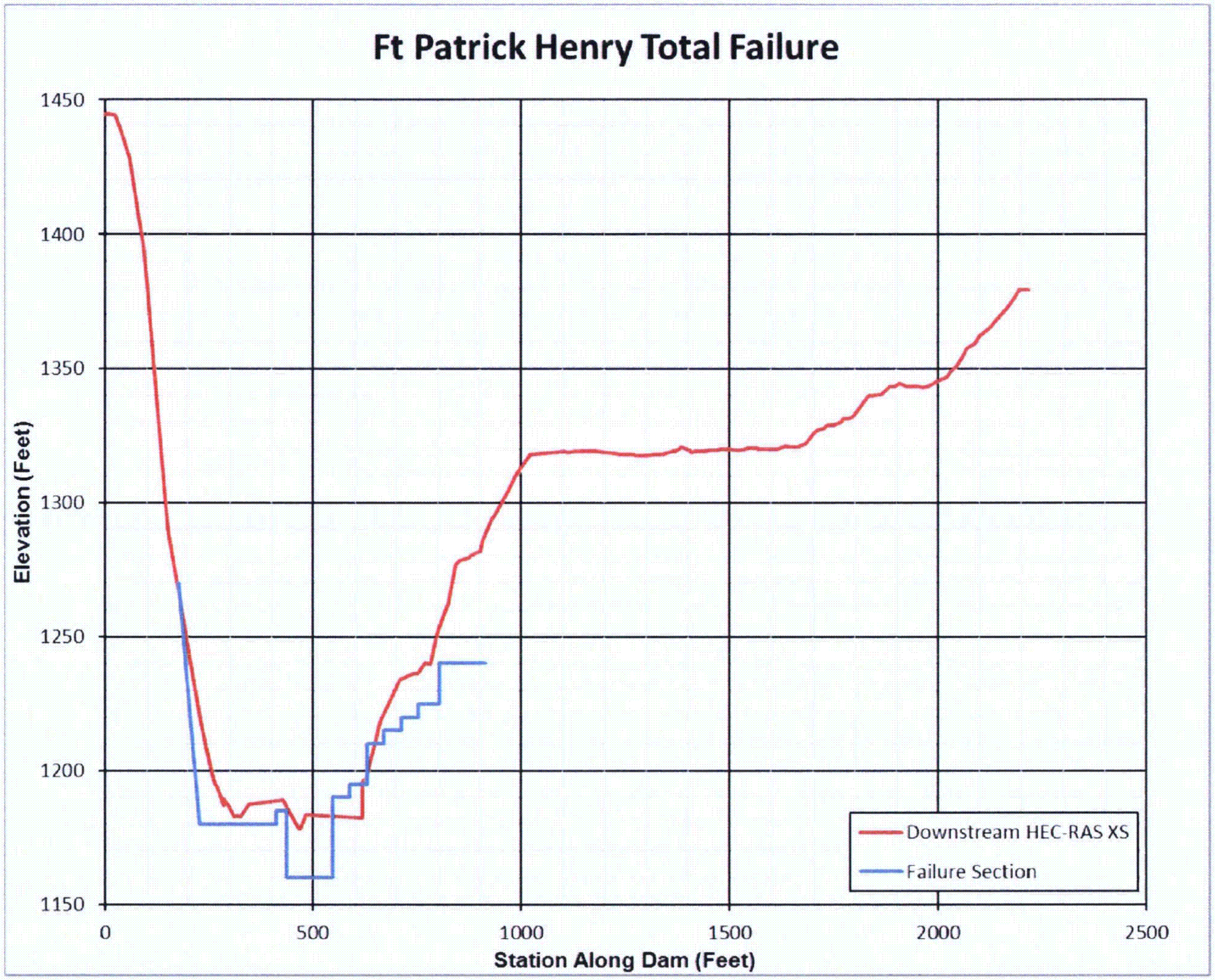


Figure 2.4-32 Fort Patrick Henry Total Failure Plot (Sheet 4 of 4)

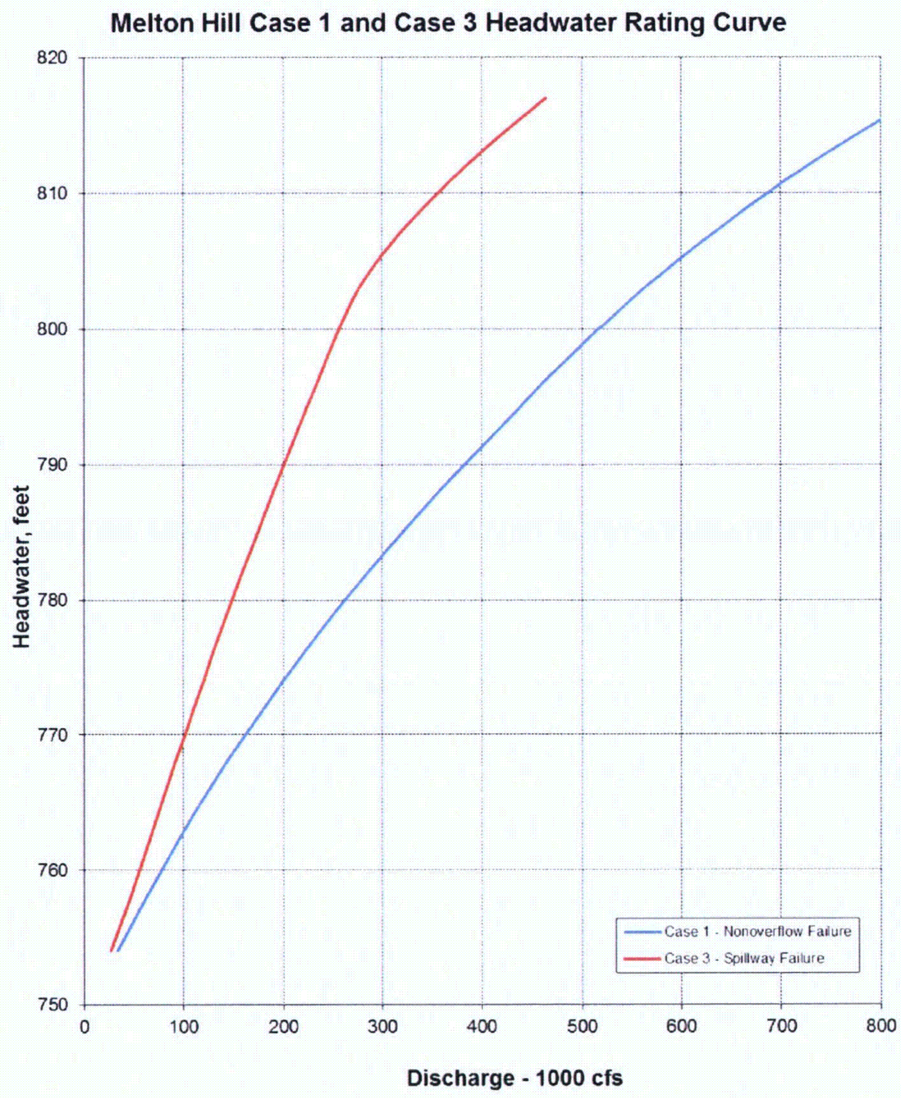


Figure 2.4-33 Dam Rating Curves for Melton Hill Dam

Figure 2.4-34 thru Figure 2.4-40 Are Not Used

APPENDIX 2.4A

SOCH MODEL

Note: Appendix 2.4A contains information regarding the SOCH model that is used in Sections 2.4.4, 2.4.11, and 2.4.14.8.

Runoff and Stream Course Model

The runoff model used to determine Tennessee River flood hydrographs at Watts Bar Nuclear Plant is divided into 40 unit areas and includes the total watershed above Chickamauga Dam. Unit hydrographs are used to compute flows from the unit areas. The watershed unit areas are shown in Figure 2.4-9. The unit area flows are combined with appropriate time sequencing or channel routing procedures to compute inflows into the most upstream tributary reservoirs which in turn are routed through the reservoirs using standard routing techniques. Resulting outflows are combined with additional local inflows and carried downstream using appropriate time sequencing or routing procedures including unsteady flow routing. Unit hydrographs were developed for each unit area for which discharge records were available from maximum flood hydrographs either recorded at stream gaging stations or estimated from reservoir headwater elevation, inflow, and discharge data using the procedures described by Newton and Vineyard Reference 1. For non gaged unit areas synthetic unit graphs were developed from relationships of unit hydrographs from similar watersheds relating the unit hydrograph peak flow to the drainage area size, time to peak in terms of watershed slope and length, and the shape to the unit hydrograph peak discharge in cfs per square mile. Unit hydrograph plots are provided in Figure 2.4-10 (11 Sheets). Table 2.4-13 contains essential dimension data for each unit hydrograph.

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

Unsteady flow routings were computer solved with the Simulated Open Channel Hydraulics (SOCH) mathematical model based on the equations of unsteady flow Reference 2. The SOCH model inputs include the reservoir geometry, upstream boundary inflow hydrograph, local inflows, and the downstream boundary headwater discharge relationships based upon operating guides or rating curves when the structure geometry controls. Seasonal operating curves are provided in Figure 2.4-3 (12 Sheets).

Discharge rating curves are provided in Figure 2.4A-11 (13 Sheets) for the reservoirs in the watershed at and above Chickamauga. The discharge rating curve for Chickamauga Dam is for the current lock configuration with all 18 spillway bays available. Above Watts Bar Nuclear Plant, temporary flood barriers have been installed at four reservoirs (Watts Bar, Fort Loudoun, Tellico and Cherokee Reservoirs) to increase the height of embankments and are included in the discharge rating curves for these four dams. Increasing the height of embankments at these four dams prevents embankment overflow and failure of the embankment. The vendor supplied temporary flood barriers were shown to be stable for the most severe PMF headwater/tailwater conditions using vendor recommended base friction values. A single postulated Fort Loudoun Reservoir rim leak north of the Marina Saddle Dam which discharges into the Tennessee River at Tennessee River Mile (TRM) 602.3 was added as an additional discharge component to the Fort Loudoun Dam discharge rating curve. Seven Watts Bar Reservoir rim leaks were added as additional discharge components to the Watts Bar Dam discharge rating curve. Three of the rim leak locations discharge to Yellow Creek, entering the Tennessee River three miles downstream of Watts Bar Dam. The remaining four rim leak locations discharge to Watts Creek, which enters Chickamauga Reservoir just below Watts Bar Dam.

The unsteady flow mathematical model configuration for the Fort Loudoun Tellico complex is shown by the schematic in Figure 2.4A-12. The Fort Loudoun Reservoir portion of the model from TRM 602.3 to TRM 652.22 is described by 29 cross sections with additional sections being interpolated between the original sections for a total of 59 cross-sections in the SOCH model, with a variable cross-section spacing of about 1 mile. The unsteady flow model was extended upstream on the French Broad and Holston Rivers to Douglas and Cherokee Dams, respectively.

The French Broad River from the mouth to Douglas Dam at French Broad River mile (FBRM) 32.3 was described by 25 cross sections with additional sections being interpolated between the original sections for a total of 49 cross sections in the SOCH model, with a variable cross section spacing of about 1 mile. The Holston River from the mouth to Cherokee Dam at Holston River mile (HRM) 52.3 was described by 29 cross sections with one additional cross section being interpolated between each of the original sections for a total of 57 cross sections in the SOCH model, with a variable cross section spacing of about 1 mile.

The Little Tennessee River was modeled from Tellico Dam, Little Tennessee River mile (LTRM) 0.3 to Chilhowee Dam at Little Tennessee River mile (LTRM) 33.6. The Little Tennessee River from Tellico Dam to Chilhowee Dam at LTRM 33.6 was described by 23 cross sections with additional sections being interpolated between the original sections for a total of 49 cross sections in the SOCH model, with a variable cross-section spacing of up to about 1.8 miles.

Fort Loudoun and Tellico unsteady flow models are joined by an interconnecting canal. The canal was modeled using nine cross sections with an average cross section spacing of about 0.18 miles.

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

Using the available data for the March 1973 flood on Fort Loudoun Reservoir and for the French Broad and Holston rivers. The verification of the 1973 flood is shown in Figure 2.4A-13 (2 Sheets). Because there were limited data to verify against on the French Broad and Holston rivers, the steady state HEC RAS model was used to replicate the Federal Emergency Management Agency (FEMA) published 100 and 500 year profiles. Tellico Dam was not closed until 1979, thus was not in place during the 1973 flood for verification.

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

A schematic of the steady state SOCH model for Watts Bar Reservoir is shown in Figure 2.4A-15. The model for the 72.4 mile long Watts Bar Reservoir was described by 39 cross sections with two additional sections being added in the upper reach for a total of 41 sections in the SOCH steady state model with a variable cross section spacing of up to about 2.8 miles. The model also includes a junction with the Clinch River at Tennessee River mile (TRM) 567.7. The Clinch River arm of the model goes from Clinch River mile (CRM) 0.0 to CRM 23.1 at Melton Hill Dam with one additional section being interpolated between each of the original 13 sections and cross section spaces of up to about 1 mile. Another junction at TRM 601.1 connects the Little Tennessee River arm of the model from the mouth to Tellico Dam at LTRM 0.3 with cross section spaces of about 0.08 miles. The time step was tested between 5 and 60 seconds which produced stable and comparable results over the full range. A time step of 5 seconds was used for the analysis to allow multiple reservoirs and/or river segments to be coupled together with different cross section spacing. The verification of Watts Bar Reservoir for the March 1973 and the May 2003 floods are shown in Figure 2.4A-16 and Figure 2.4A-17, respectively.

A schematic of the unsteady flow model for Chickamauga Reservoir is shown in Figure 2.4A-18. The model for the 58.9 mile long Chickamauga Reservoir was described by 29 cross sections with one additional section being interpolated between each of the original 29 sections for a total of 53 sections in the SOCH model with a variable cross section spacing of up to about 1 mile. The model also includes a junction with the Dallas Bay embayment at TRM 480.5. The Dallas Bay arm of the model goes from Dallas Bay mile (DB) 5.23 to DB 2.86, the control point for flow out of Chickamauga Reservoir. Another junction at TRM 499.4 connects the Hiwassee River arm of the model from the mouth to the Charleston gage at HRM 18.9. The time step was tested between 5 and 50 seconds producing stable and comparable results over the full range. A time step of 5 seconds was used for the analysis to allow multiple reservoirs and/or river segments to be coupled together with different cross section spacing. The verification of Chickamauga Reservoir for the March 1973 and the May 2003 floods are shown in Figure 2.4A-19 and Figure 2.4A-20, respectively.

Verifying the reservoir models with actual data approaching the magnitude of the PMF is not possible, because no such events have been observed. Therefore, using flows in the magnitude of the PMF (1,200,000 - 1,300,000 cfs), steady state profiles were computed using the HEC RAS steady state model and compared to computed elevations from the SOCH model. An example of the comparison between HEC RAS and SOCH profiles is shown for Chickamauga Reservoir in Figure 2.4A-21. This approach was applied for each of the SOCH reservoir models. Similarly, the tailwater rating curve was compared at each project as shown for Watts Bar Dam in Figure 2.4A-22. In this figure, the initial tailwater curve is compared to results from the HEC RAS or SOCH models.

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

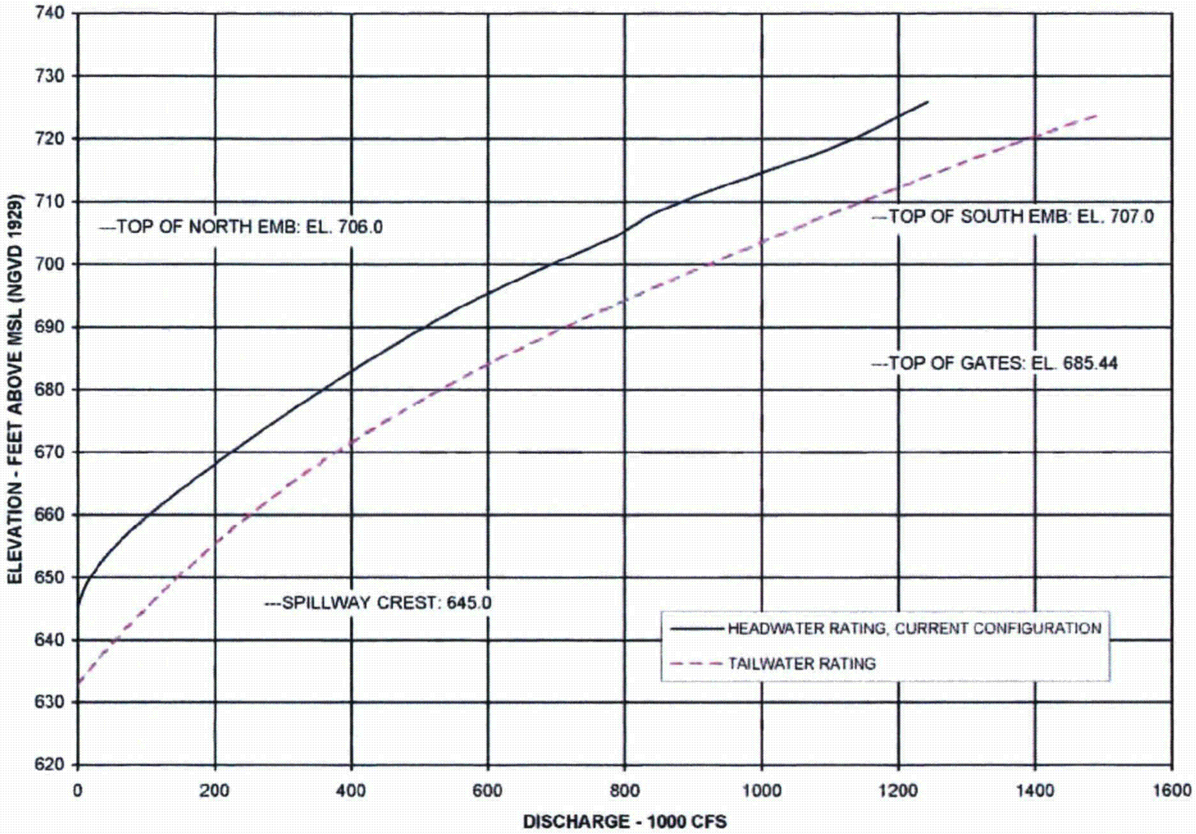
Median initial reservoir elevations for the appropriate season were used at the start of the PMF storm sequence. Use of median elevations is consistent with statistical experience and avoids unreasonable combinations of extreme events.

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

REFERENCES

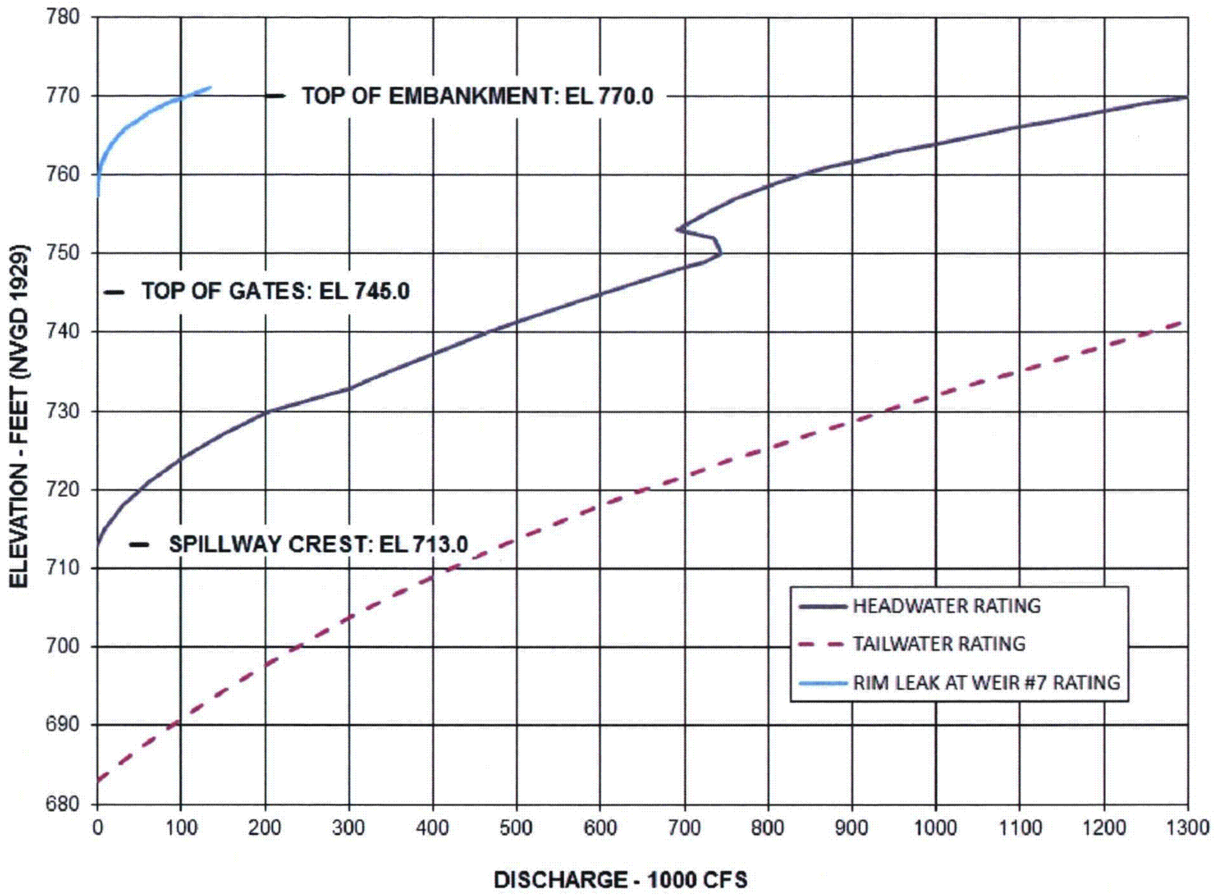
1. Newton, Donald W., and Vineyard, J. W., "Computer-Determined Unit Hydrographs From Floods," Journal of the Hydraulics Division, ASCE, Volume 93, No. HY5, September 1967.

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



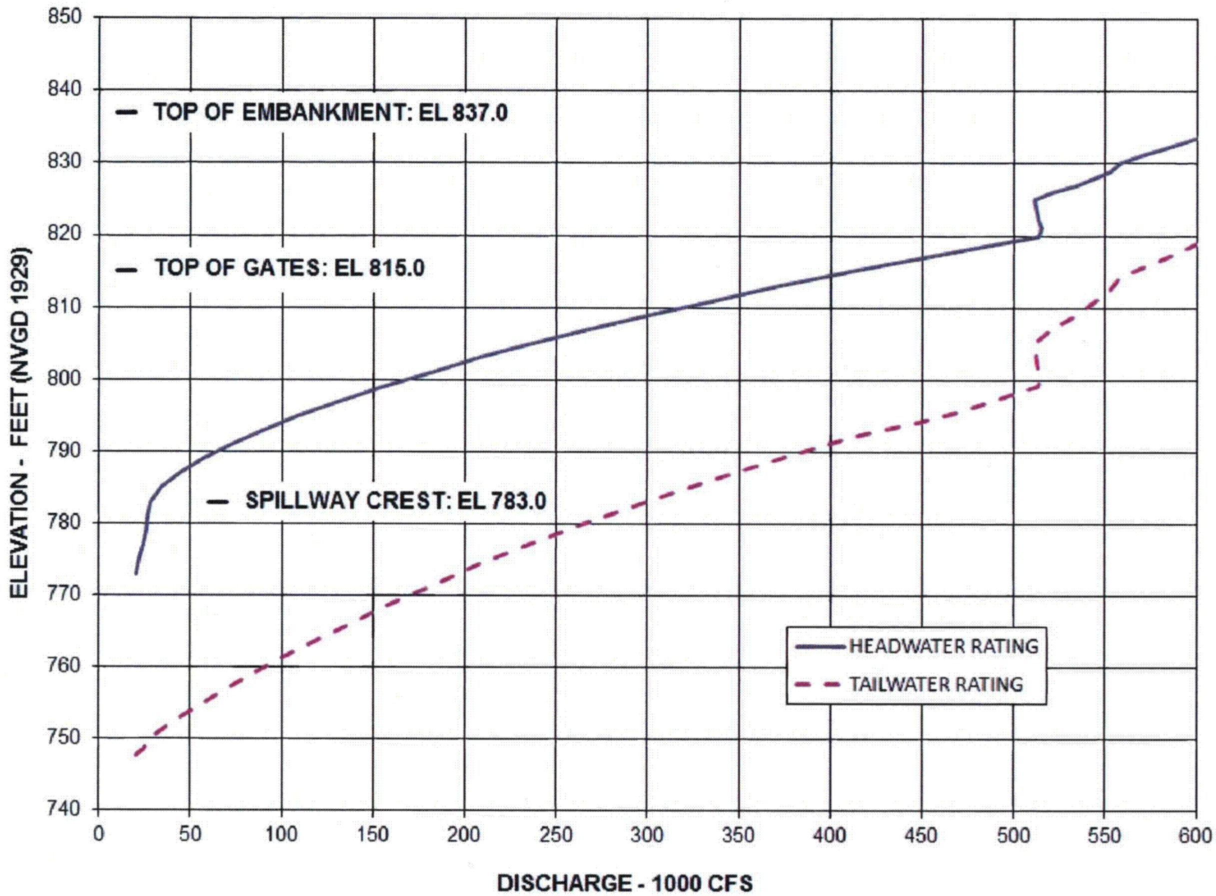
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Chickamauga Dam
Figure 2.4A - 11 (Sheet 1 of 13)

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



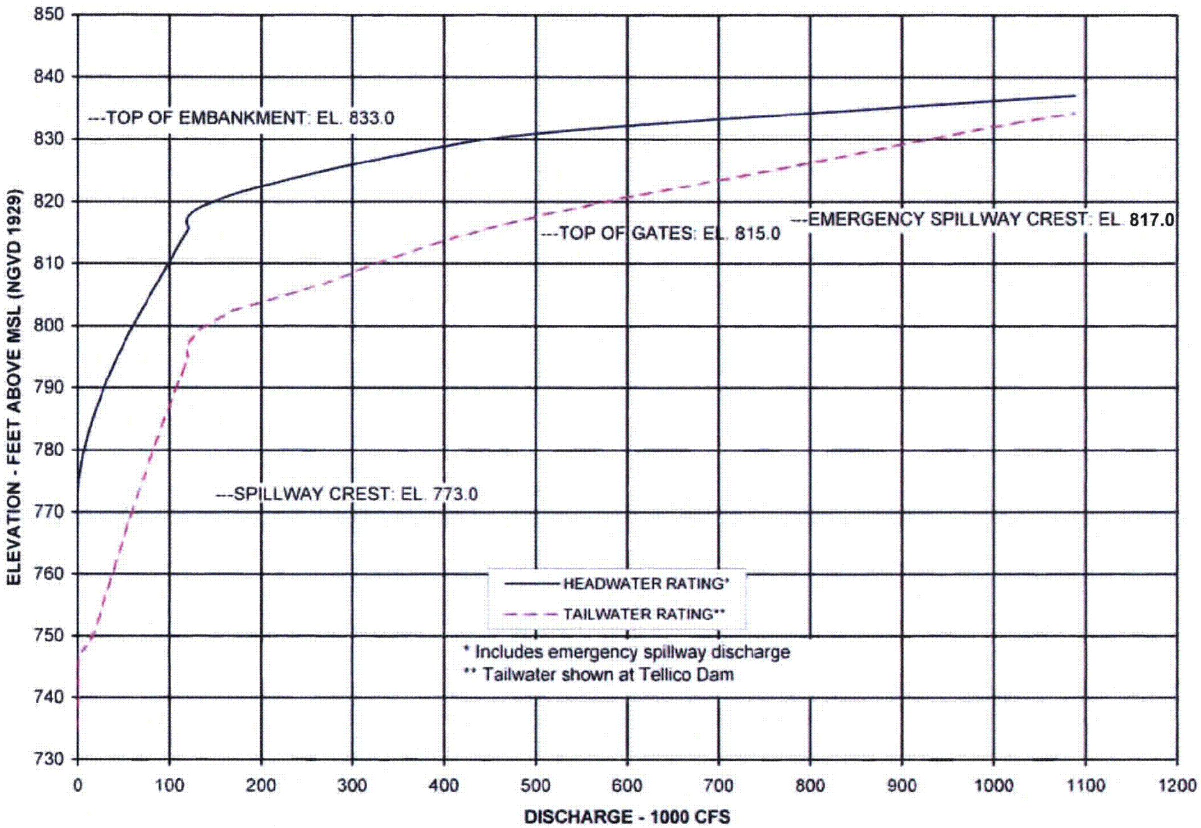
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Watts Bar Dam

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



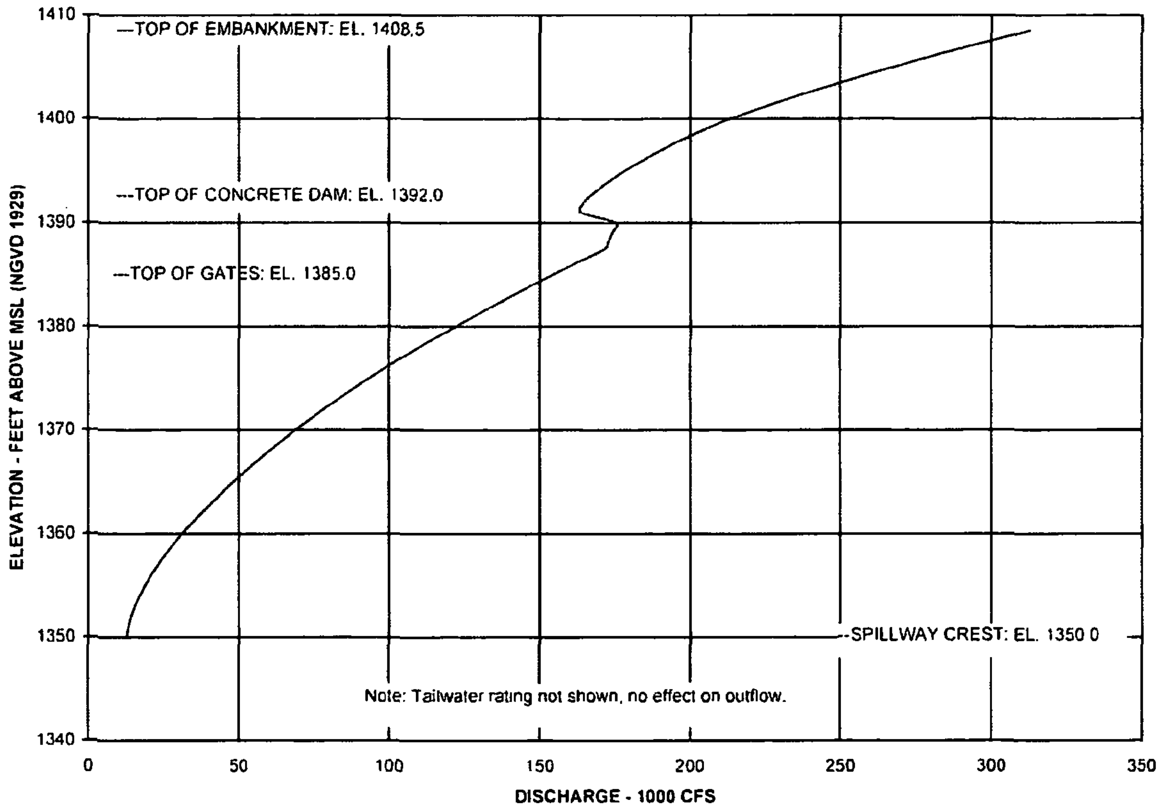
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Fort Loudoun Dam

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



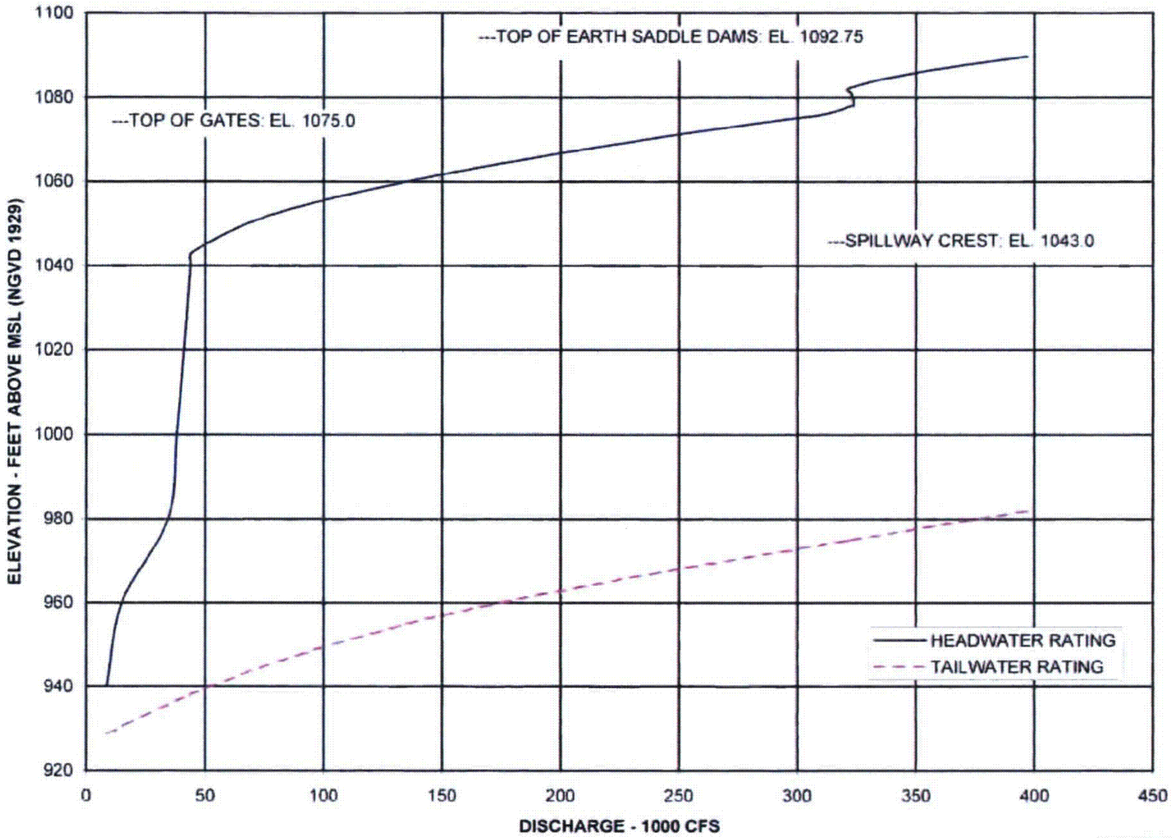
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Tellico Dam

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



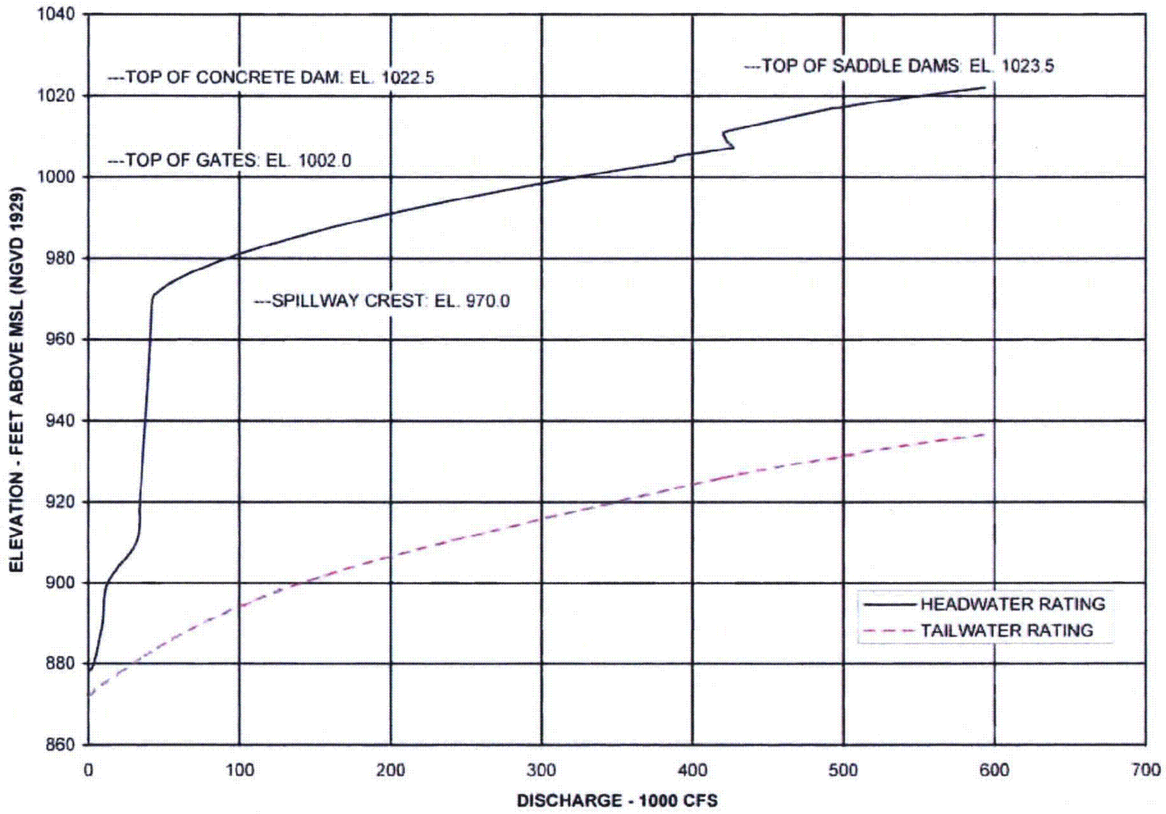
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Boone Dam

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



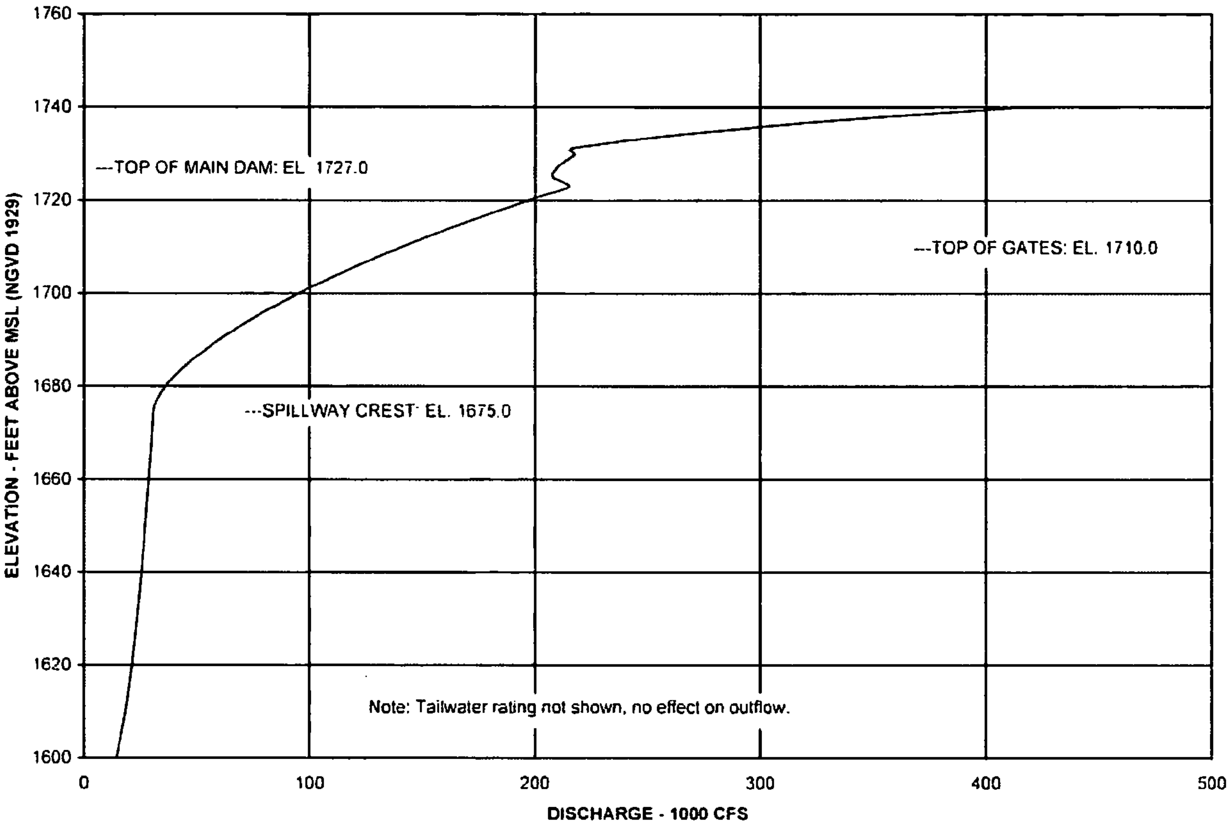
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Cherokee Dam

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



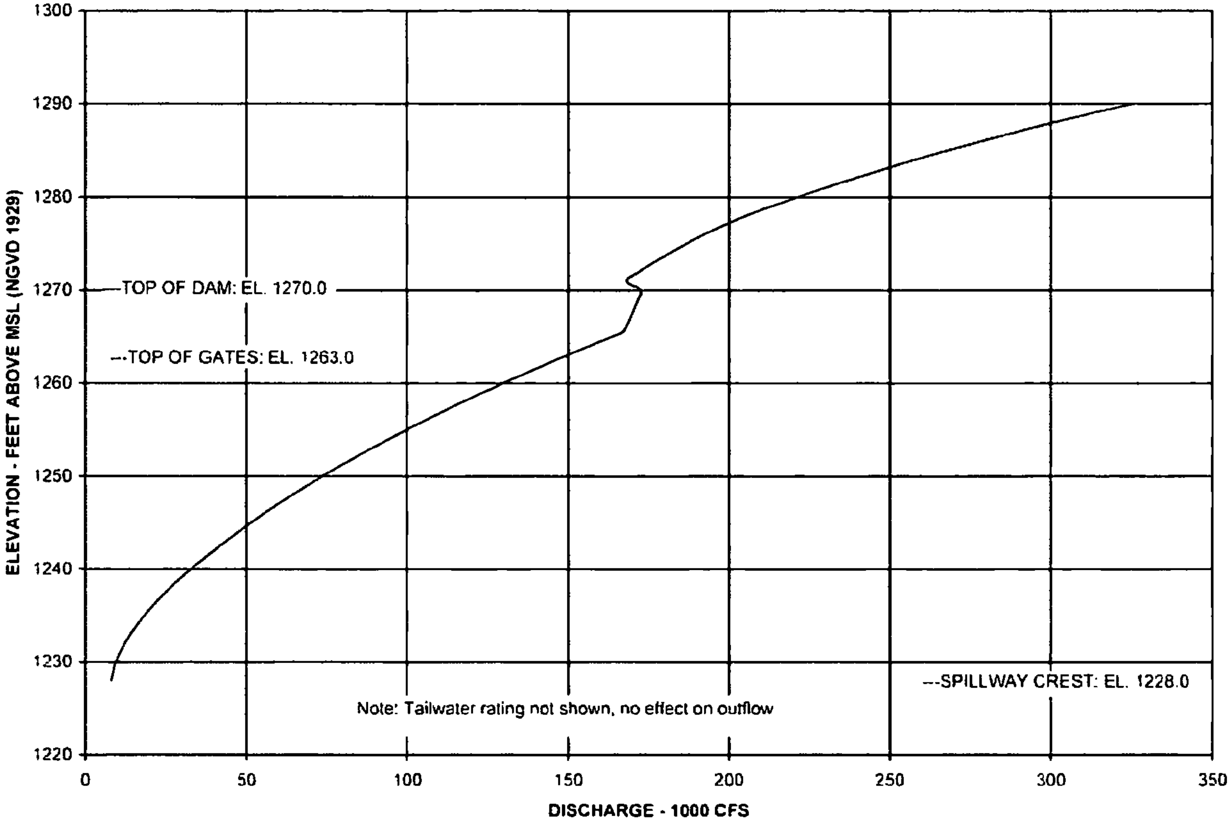
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Douglas Dam

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



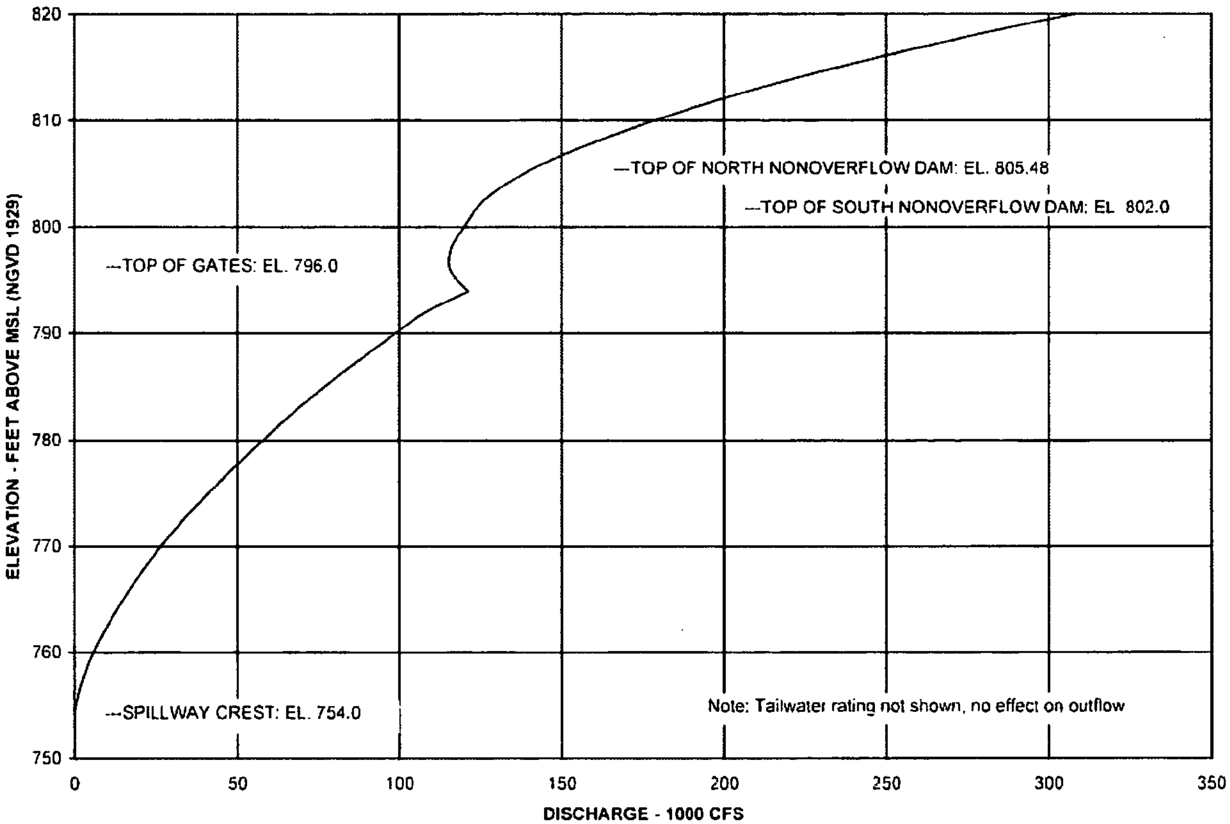
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Fontana Dam

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



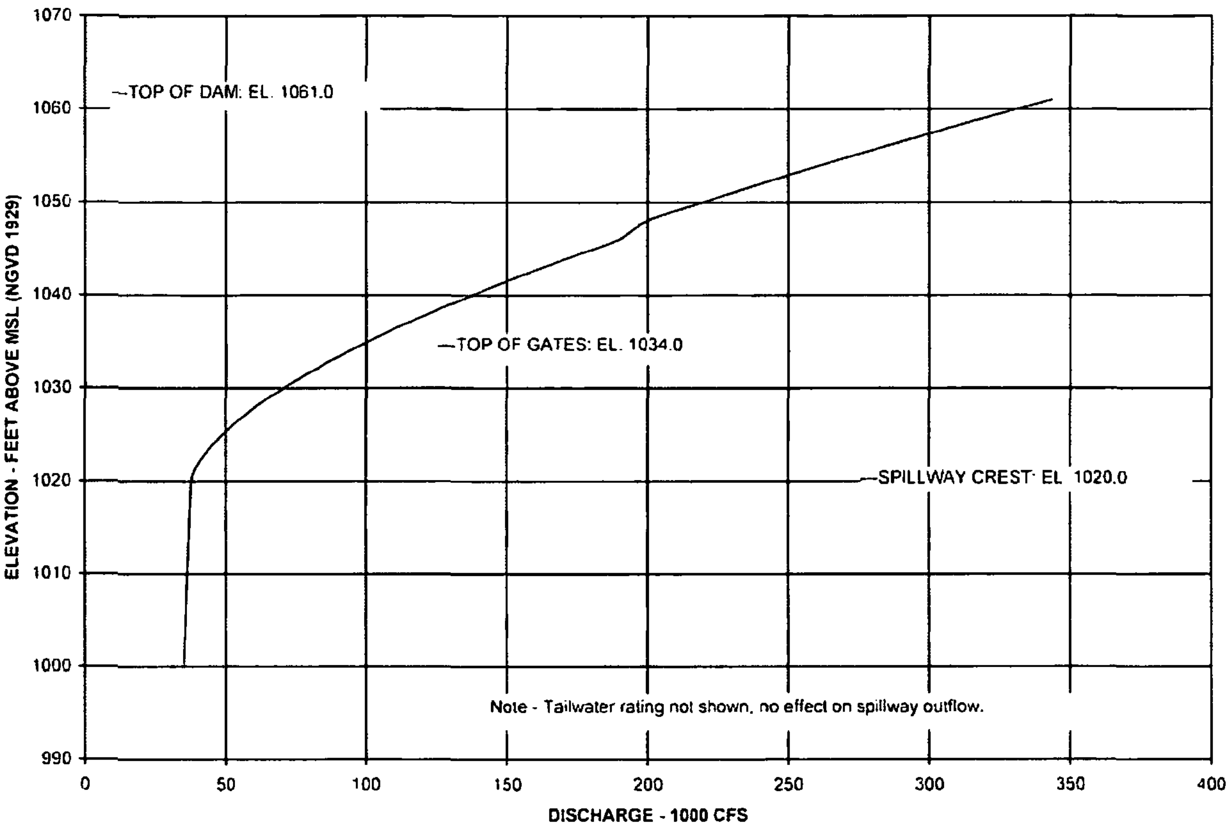
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Fort Patrick Henry Dam

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



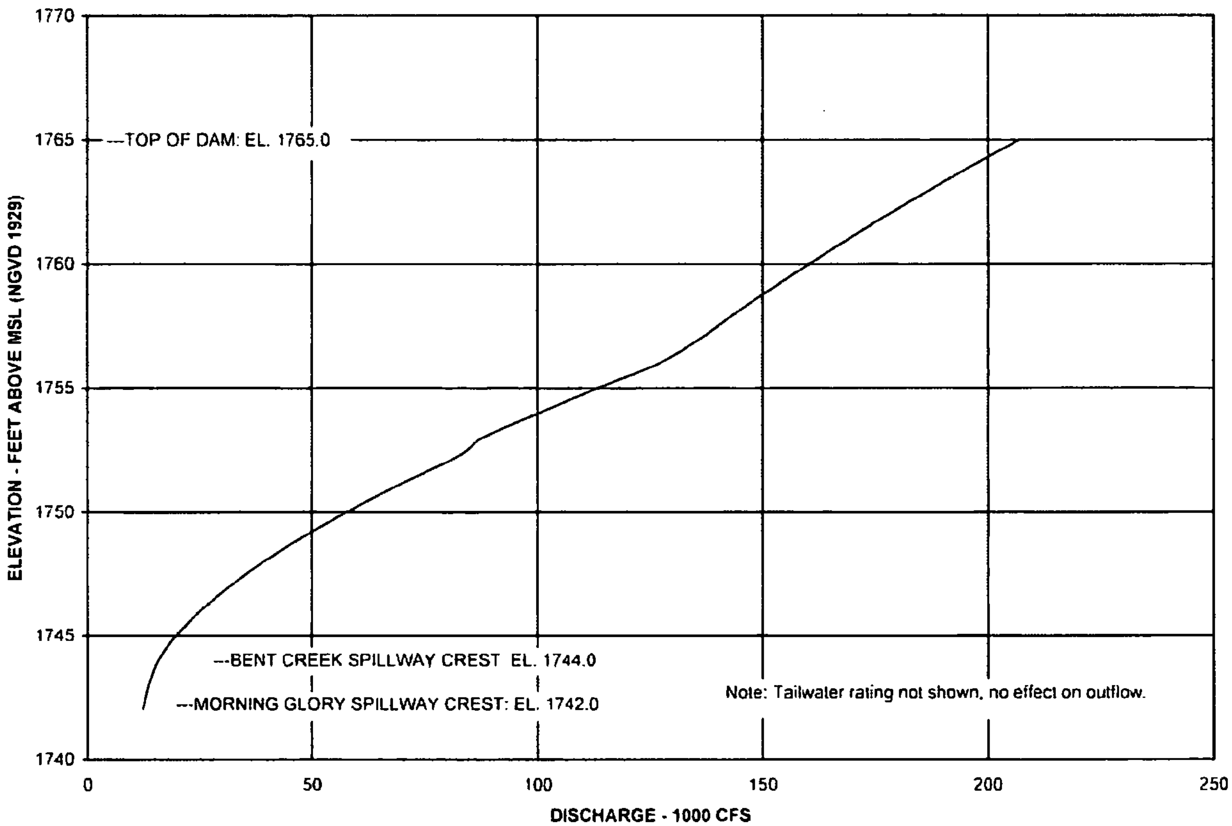
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Melton Hill Dam

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



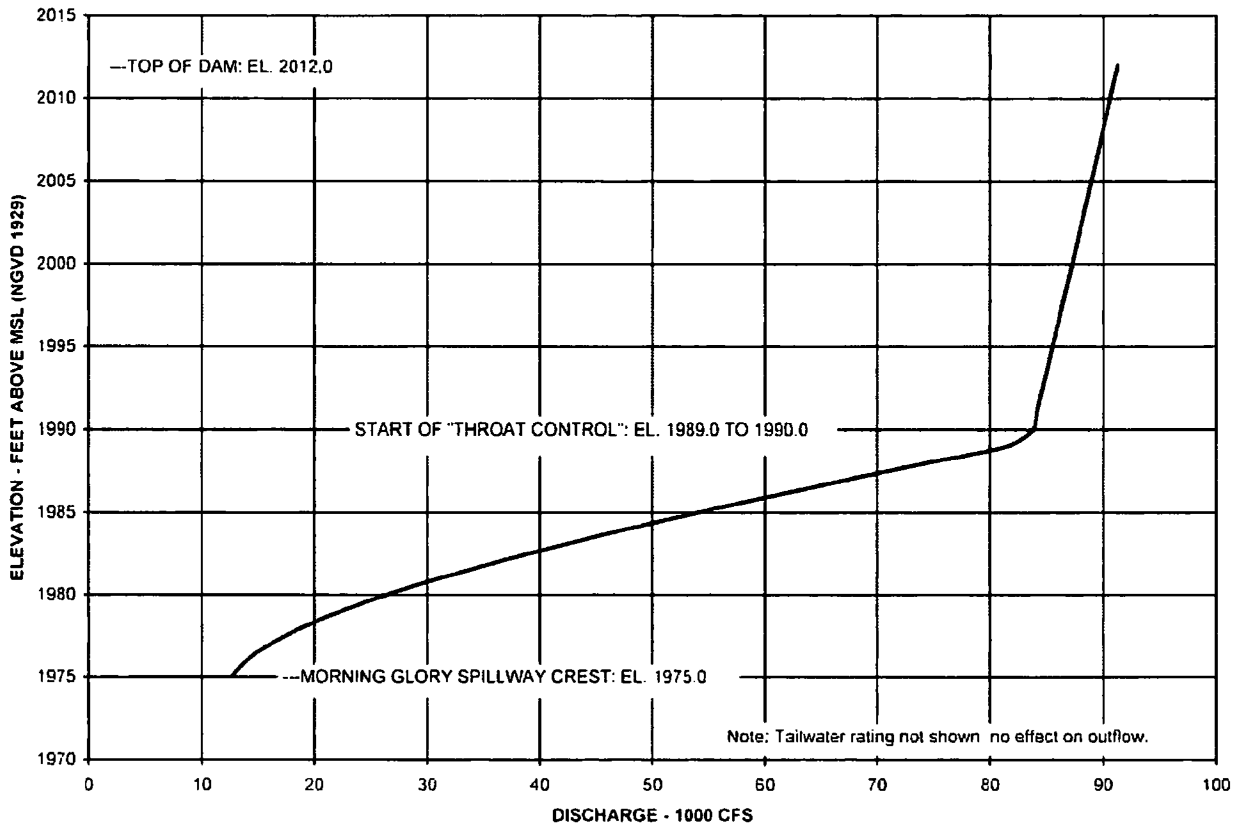
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Norris Dam

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



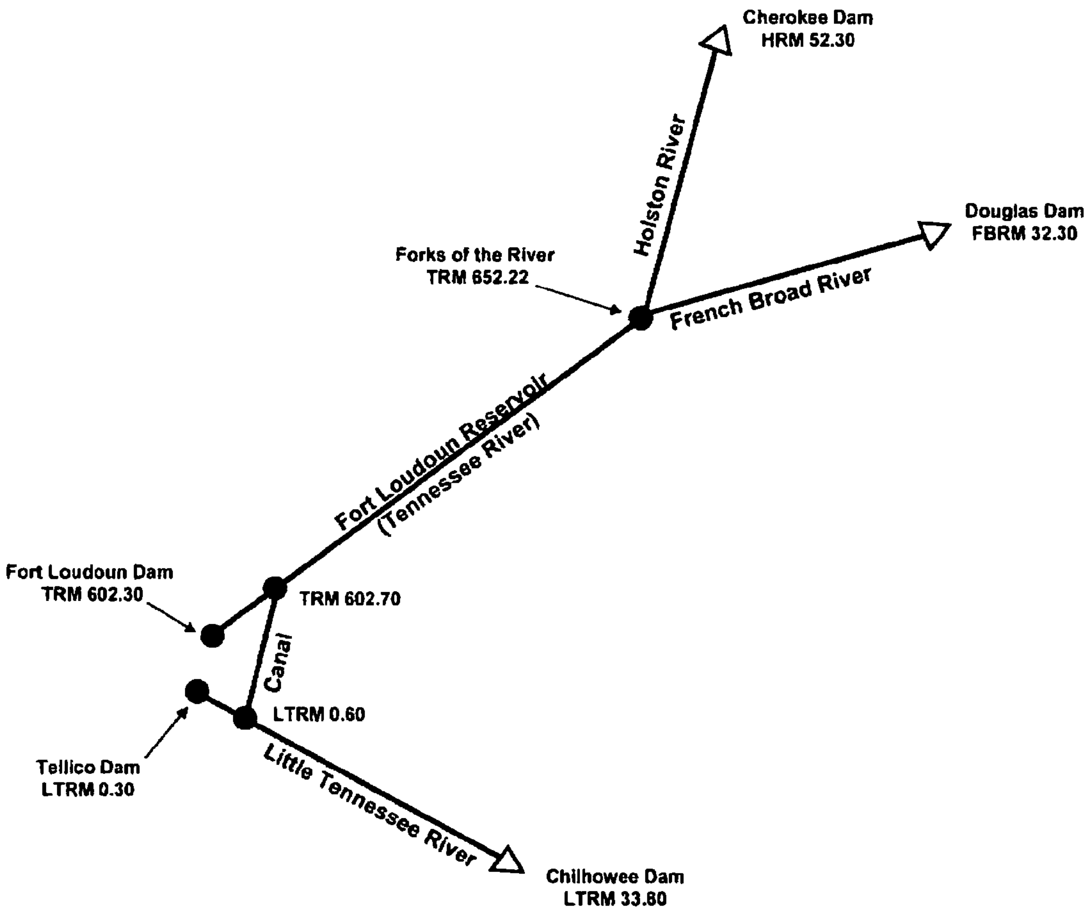
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, South Holston Dam

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



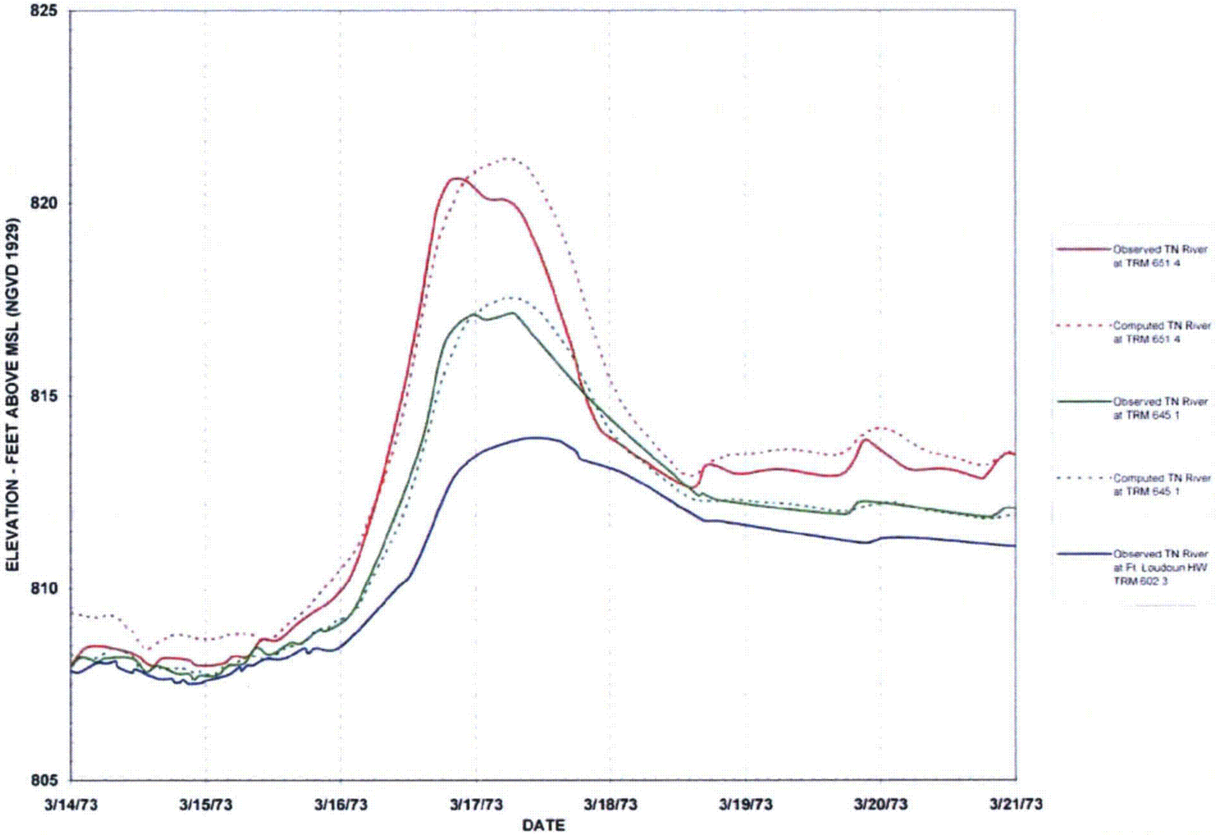
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Discharge Rating Curve, Watauga Dam

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



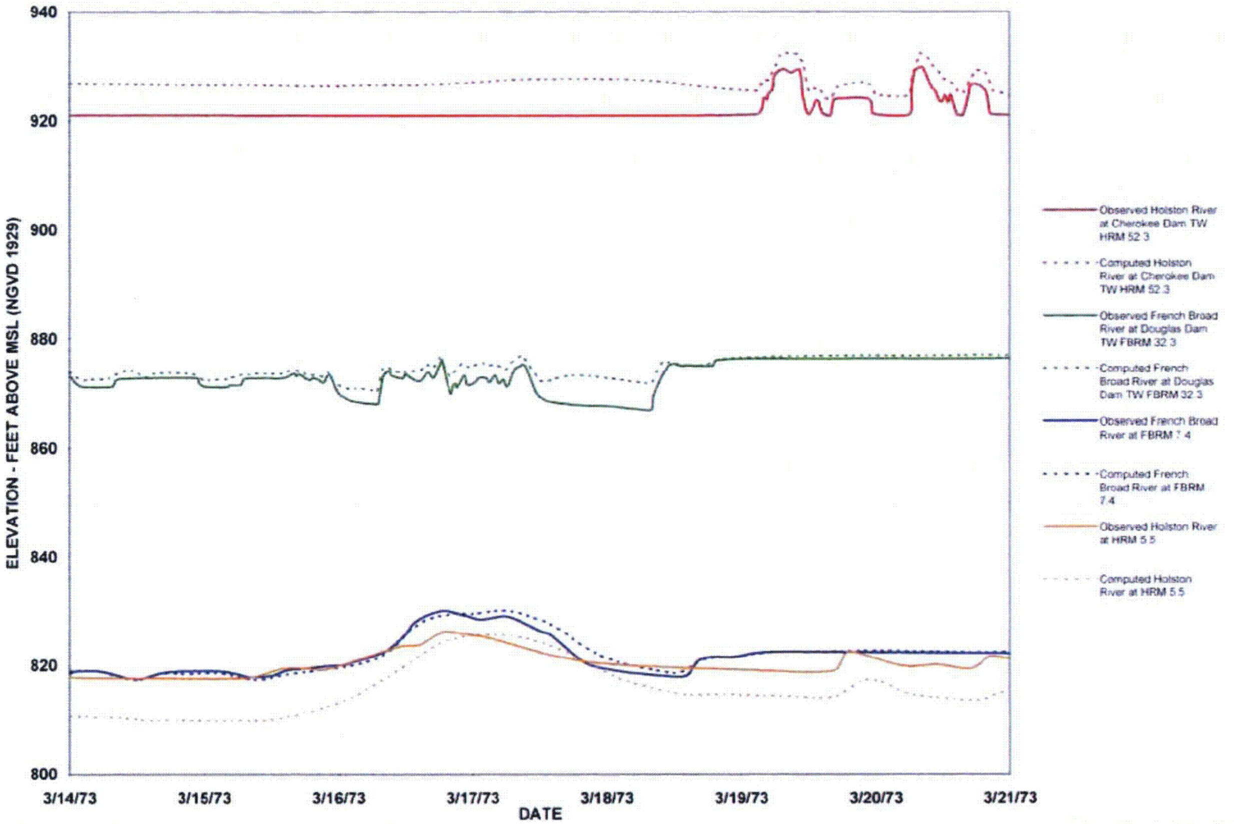
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Fort Loudoun - Tellico SOCH Unsteady Flow Model Schematic

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



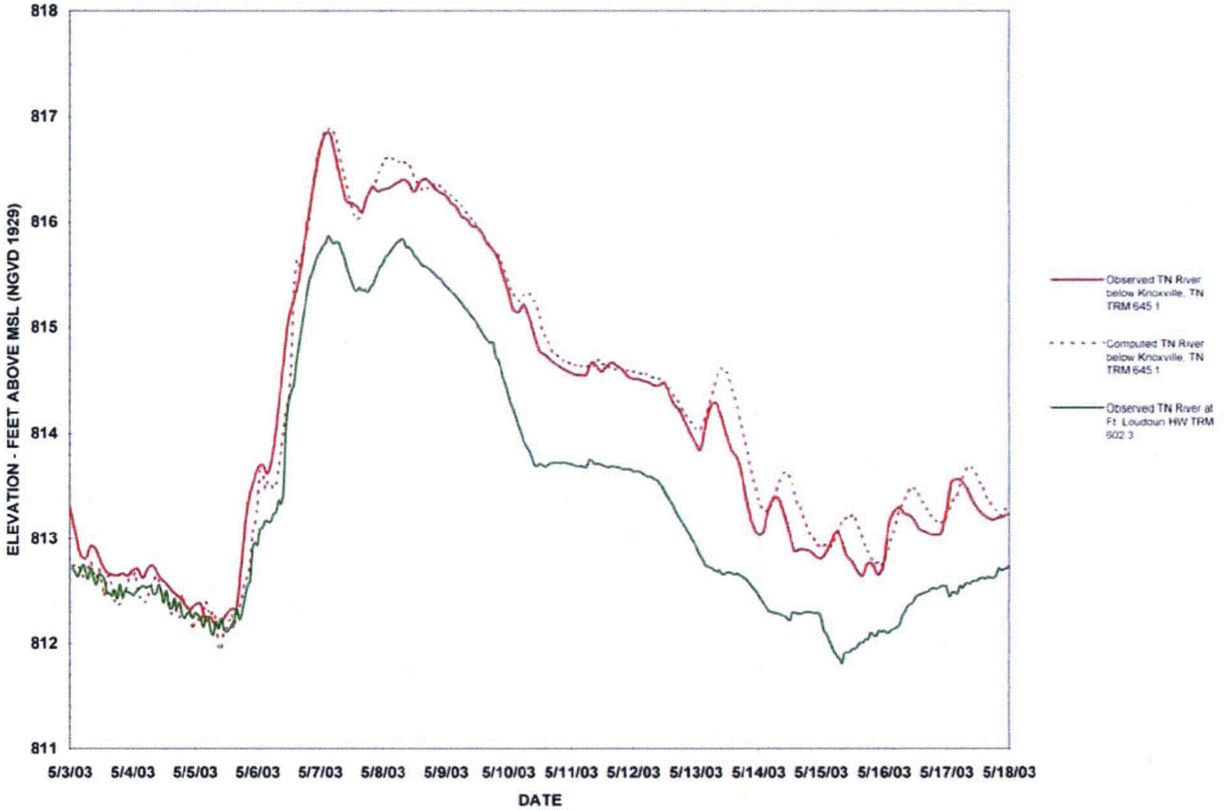
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood

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



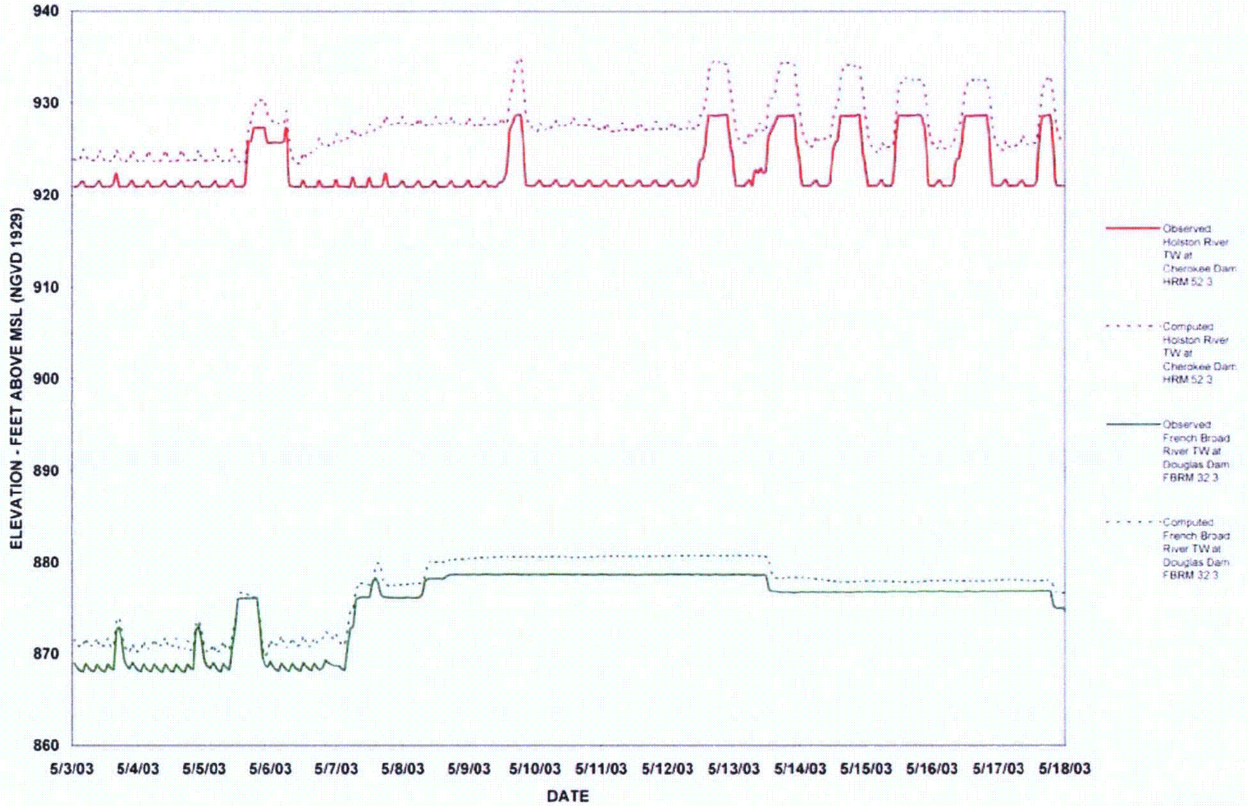
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun Reservoir March 1973 Flood

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



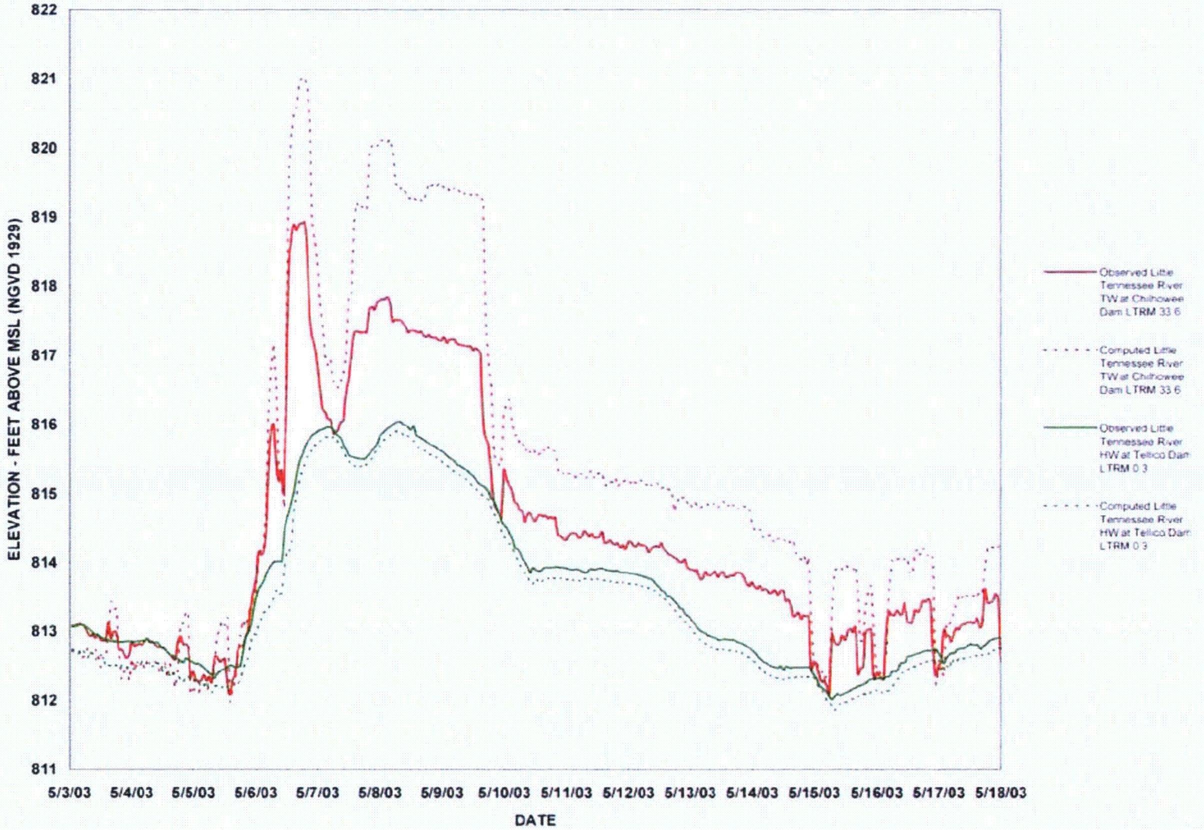
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood

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



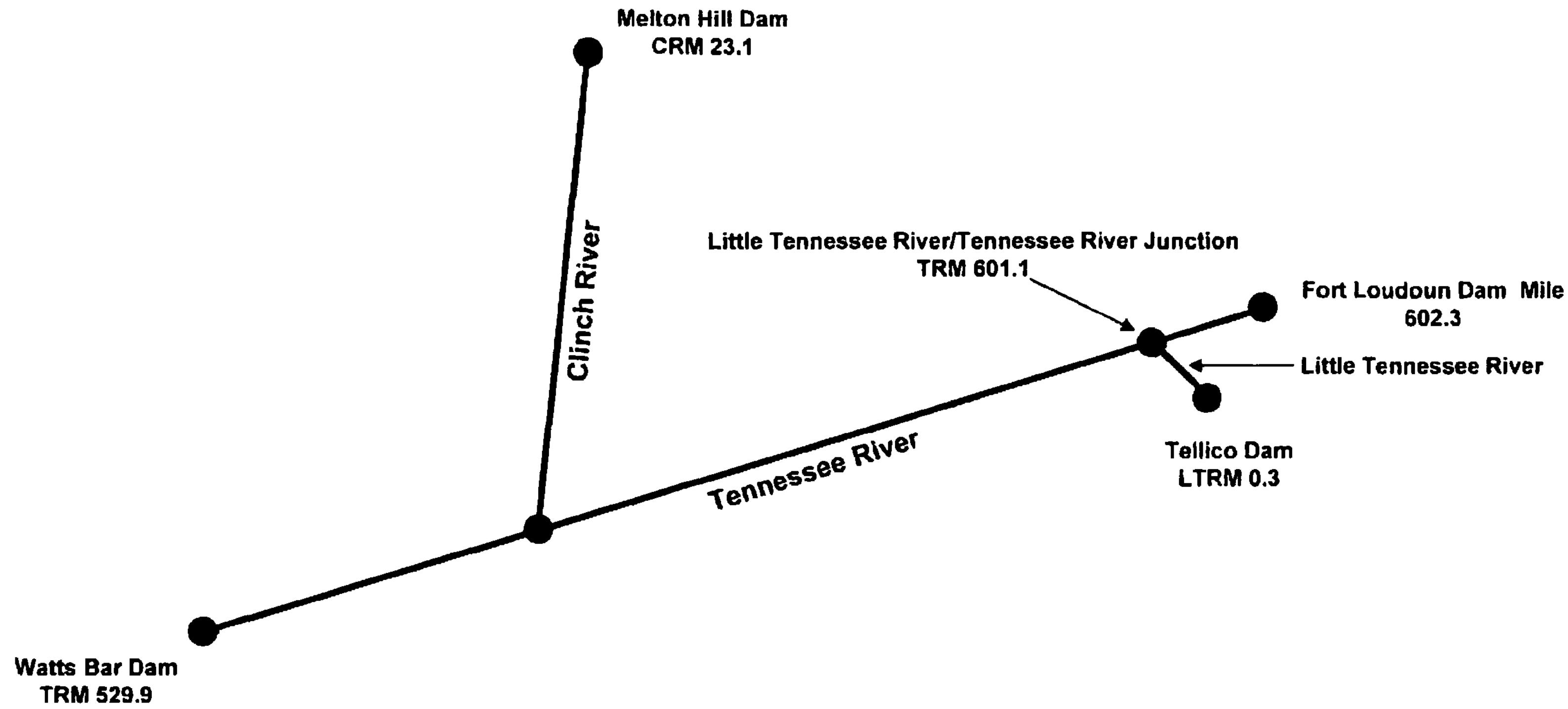
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood

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



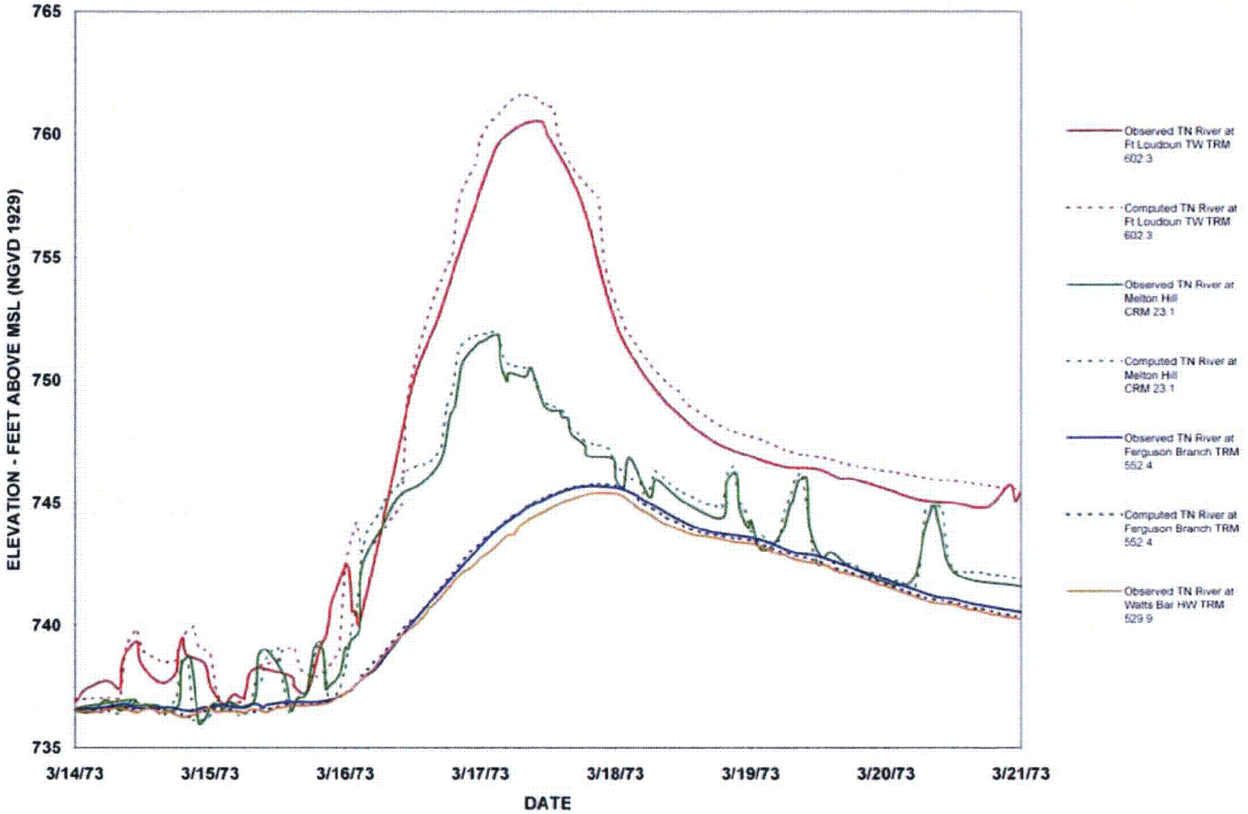
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Fort Loudoun - Tellico Reservoir May 2003 Flood

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



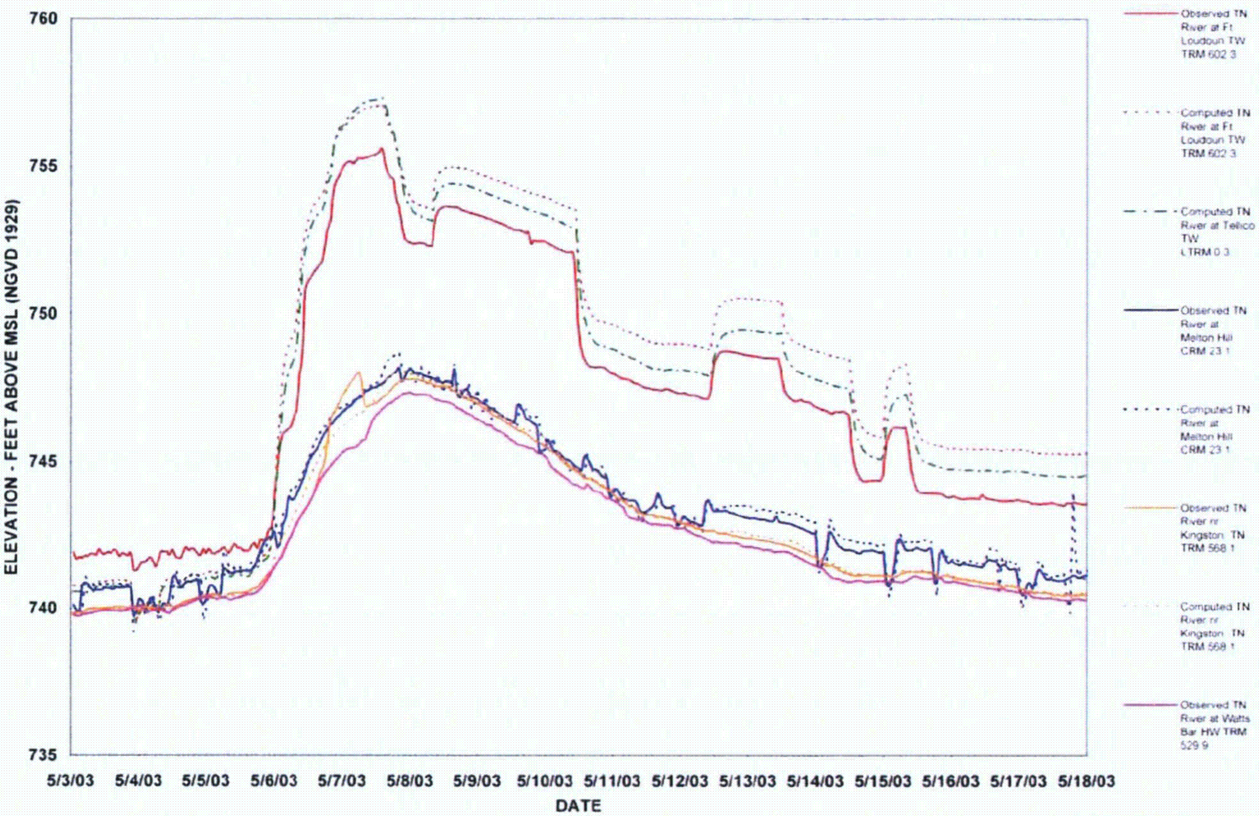
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Watts Bar SOCH Unsteady Flow Model Schematic

Figure 2.4A -15 Watts Bar SOCH Unsteady Flow Model Schematic



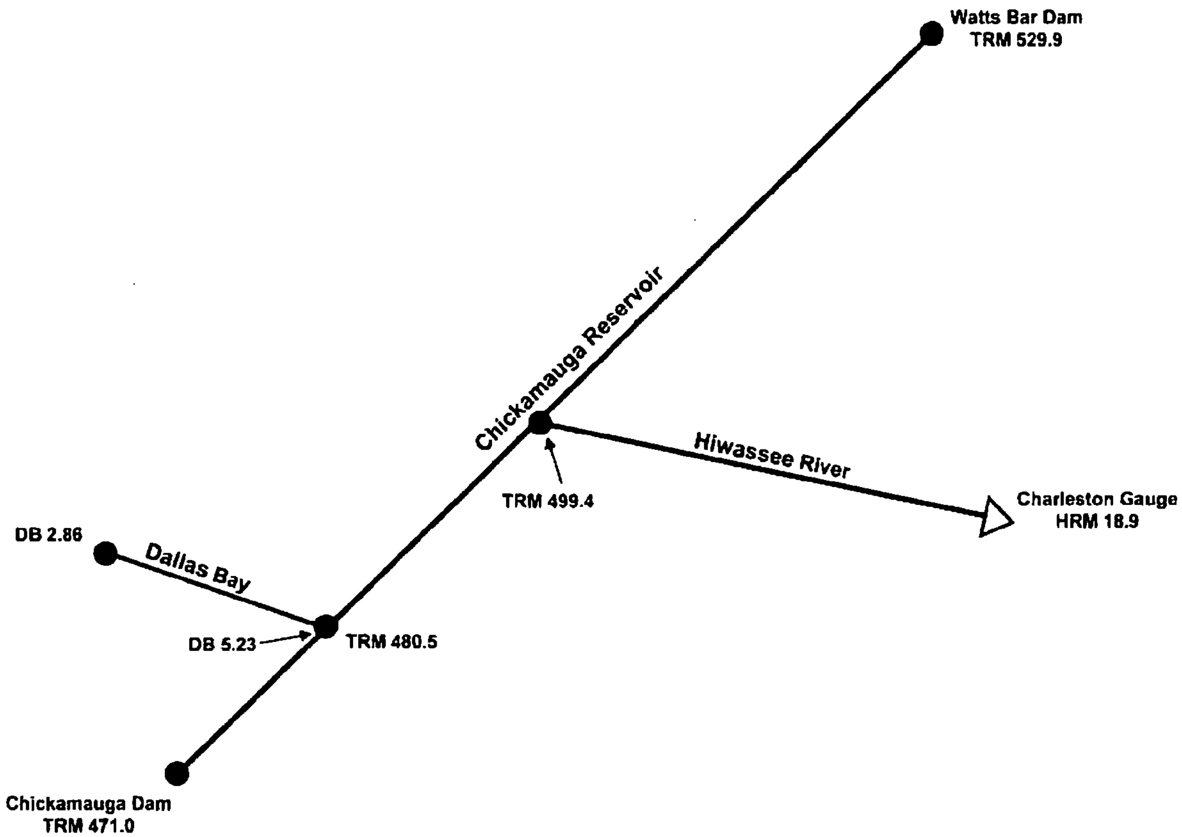
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Watts Bar Reservoir March 1973 Flood

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



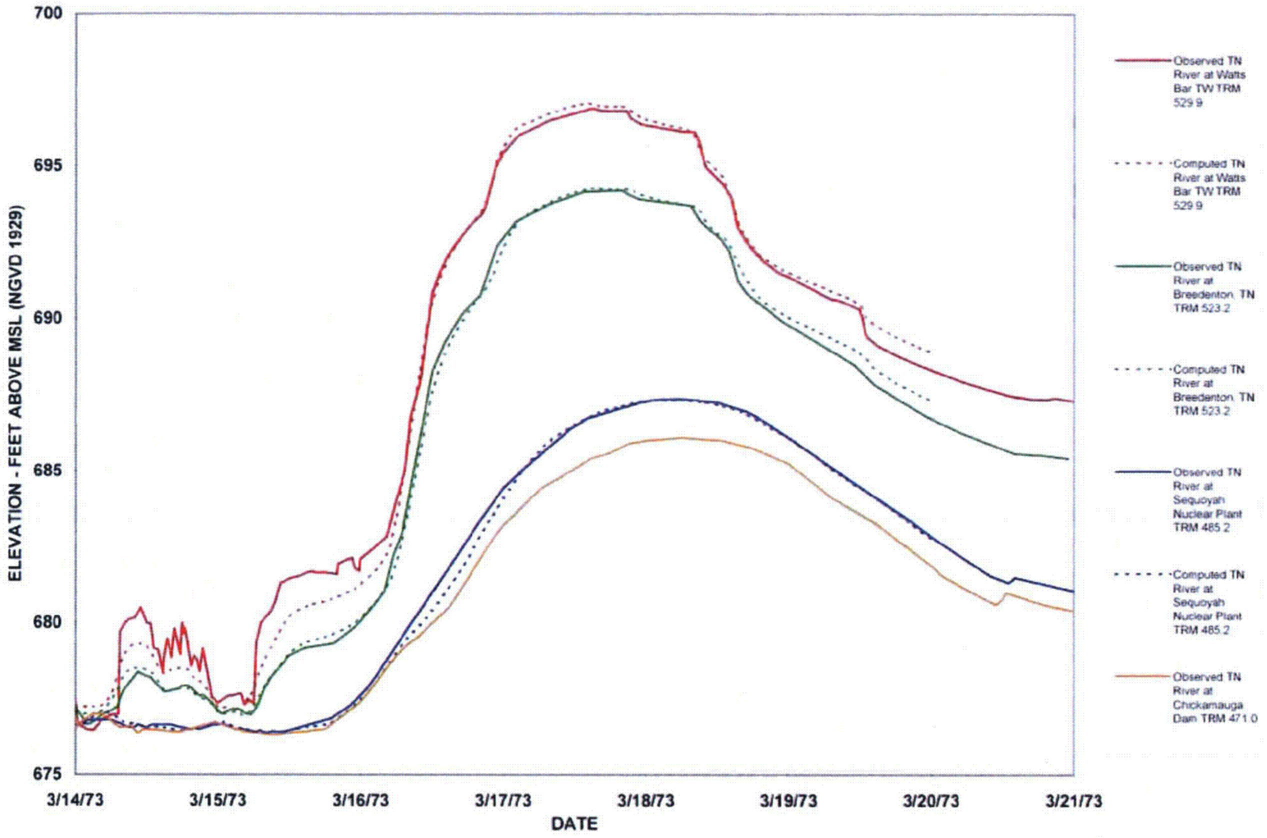
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Watts Bar Reservoir May 2003 Flood

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



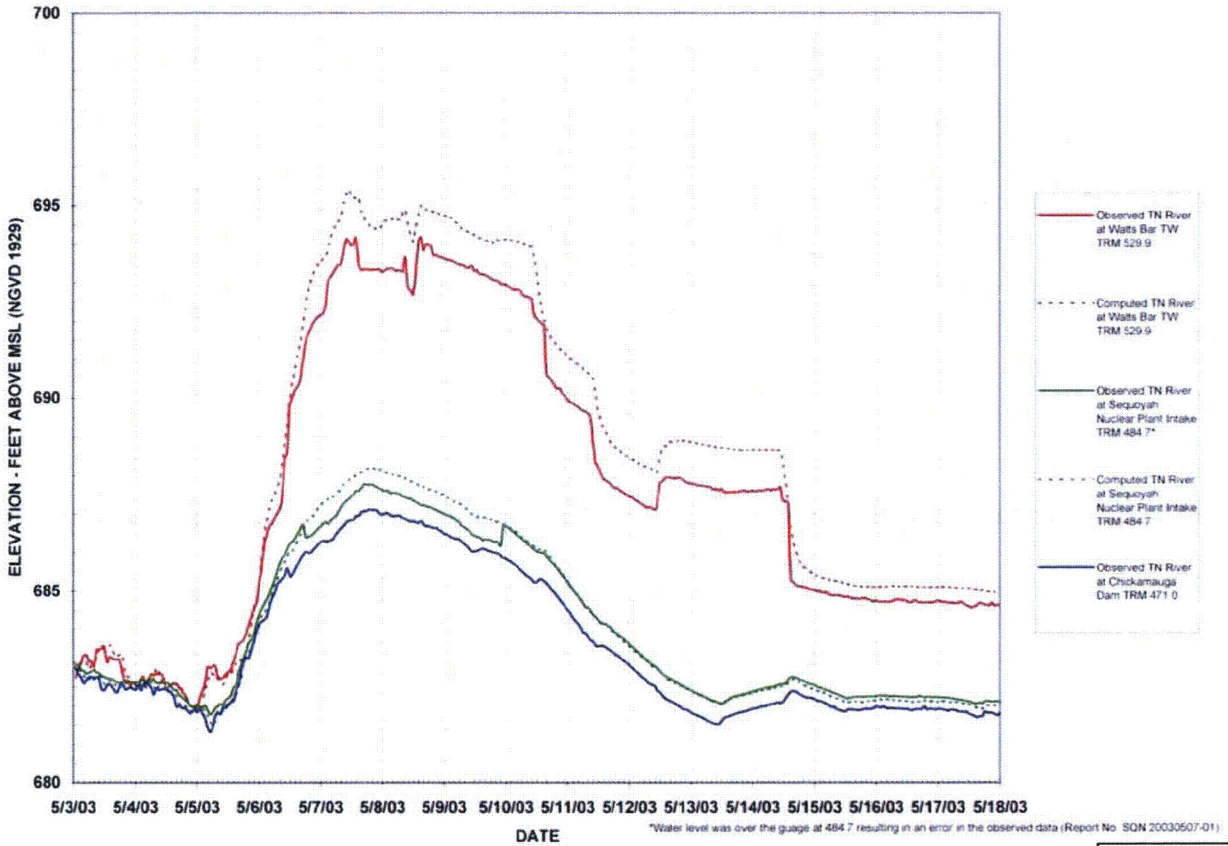
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Chickamauga SOCH Unsteady Flow Model Schematic

Figure 2.4A -18 Chickamauga SOCH Unsteady Flow Model Schematic



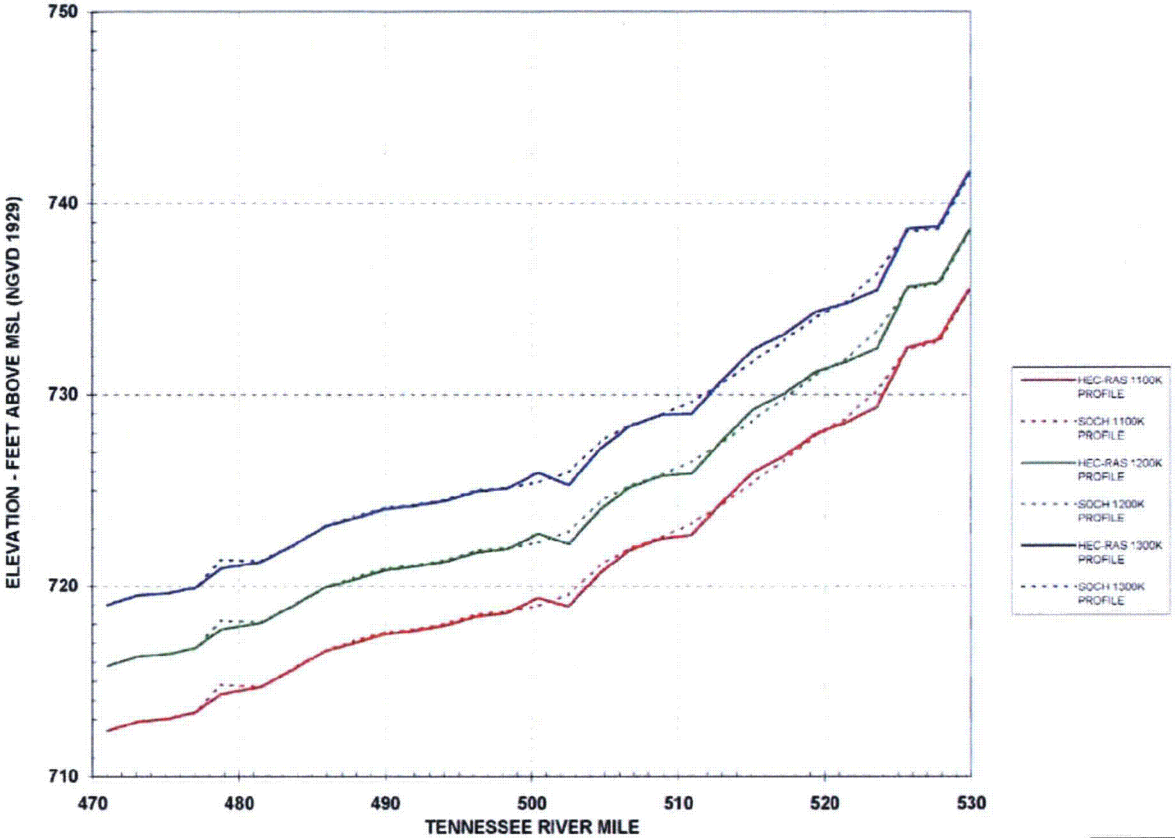
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Chickamauga Reservoir March 1973 Flood

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



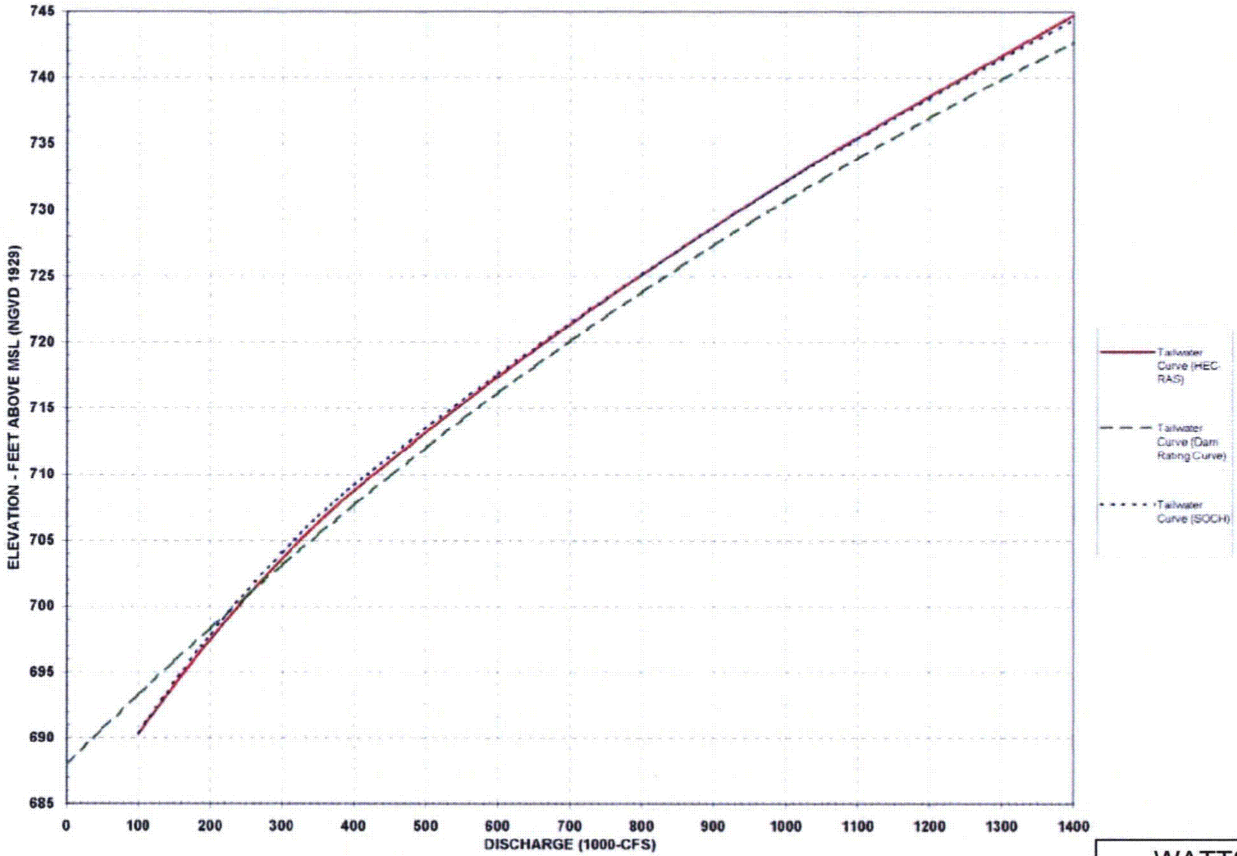
WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Unsteady Flow Model Chickamauga Reservoir May 2003 Flood

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



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Chickamauga Steady State Profile Comparisons **SOCH Model**

Figure 2.4A -21 Chickamauga Steady State Profile Comparisons



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
Tailwater Rating Curve, Watts Bar Dam **SOCH Model**

Figure 2.4A-22 Tailwater Rating Curve, Watts Bar Dam

ENCLOSURE 2
Tennessee Valley Authority
Watts Bar Nuclear Plant, Unit 2
Docket No. 50-391
Unit 2 FSAR Section 2.4 Clean Pages

2.4 HYDROLOGIC ENGINEERING

Watts Bar Nuclear Plant (WBN) is located on the west bank of Chickamauga Lake at Tennessee River Mile (TRM) 528 with plant grade at elevation 728.0 ft MSL. The plant has been designed to have the capability for safe shutdown in floods up to the computed maximum water level, in accordance with regulatory position 2 of 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 calculated probable maximum flood elevation 738.9 ft combined with 0.3 ft additional margin provides a design basis probable maximum flood elevation of 739.2 ft. The design basis flood elevation and plant protection during external flood events is discussed in Section 2.4.14.

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

The probable minimum flow past the site is estimated to be 3,200 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 Station	(1)	Access Hatches	3	728.0
	(2)	Stairwell Entrances	2	741.0
	(3)	Access Hatches	6	741.0
Auxiliary and Control Bldgs.	(1)	Door to Turbine Bldg.	1	708.0
	(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
Shield Building	(1)	Personnel Lock	1	714.0
	(2)	Equipment Hatch	1	753.0
	(3)	Personnel Lock	1	755.0
Diesel Generator	(1)	Equipment Access Doors	4	742.0

Building	(2)	Emergency Exits	4	742.0
	(3)	Personnel Access Door	1	742.0
	(4)	Emergency Exit	1	760.5
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 ft MSL to 728.5 ft 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 WBN 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. WBN is located in the Middle Tennessee Chickamauga watershed, U.S. Geological Survey (USGS) hydrologic unit code 06020001, one of 32 watersheds in the Region 06 – Tennessee River Watershed (Figure 2.4-1).

The Tennessee River above the Watts Bar plant site drains 17,319 square miles. Watts Bar Dam, 1.9 miles upstream, has a drainage area of 17,310 square miles. Chickamauga Dam, the next dam downstream, has a drainage area of 20,790 square miles. Two major tributaries, Little Tennessee and French Broad Rivers, rise to the east in the rugged Southern Appalachian Highlands. They flow northwestward through the Appalachian Divide which is essentially defined by the North Carolina-Tennessee border to join the Tennessee River which flows southwestward. The Tennessee River and its Clinch and Holston River tributaries flow southwest through the Valley and Ridge physiographic province which, while not as rugged as the Southern Highlands, features a number of mountains including the Clinch and Powell Mountain chains. The drainage pattern is shown on Figure 2.1-1. About 20% of the watershed rises above elevation 3,000 ft with a maximum elevation of 6,684 ft at Mt. Mitchell, North Carolina. The watershed is about 70% forested with much of the mountainous area being 100% forested.

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

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

Watershed snowfall is relatively light, averaging about 14 inches annually above the plant. Snowfall above the 3,000-ft 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 WBN.

Chickamauga Dam, 57 miles downstream, affects water surface elevations at WBN. Normal full pool elevation is 682.5 ft. At this elevation the reservoir is 58.9 miles long on the Tennessee River and 32 miles long on the Hiwassee River, covering an area of 36,050 acres, with a volume of 622,500 acre-ft. The reservoir has an average width of nearly 1 mile, ranging from 700 ft to 1.7 miles. At the Watts Bar site the reservoir is about 1100 ft wide with depths ranging between 18 ft and 26 ft at normal pool elevation.

There are 12 major dams (South Holston, Boone, Fort Patrick Henry, Watauga, Fontana, Norris, Cherokee, Douglas, Tellico, Fort Loudoun, Melton Hill, and Watts Bar) in the TVA system upstream from WBN, ten of which (those previously identified excluding Fort Patrick Henry and Melton Hill) provide about 4.4 million acre-ft of reserved flood-detention (March 15) capacity during the main flood season. Table 2.4-2 lists pertinent data for TVA's dams and reservoirs. Figure 2.4-2 presents a simplified flow diagram for the Tennessee River system. Table 2.4-3 provides the relative distances in river miles of dams to the WBN site. Details for TVA dam outlet works are provided in Table 2.4-4. In addition, there are four major dams owned by Brookfield Renewable Energy Partners (Calderwood, Chilhowee, Santeetlah, and Cheoah Dams) and two major dams owned by Duke Energy (Nantahala and Mission Dams). These reservoirs often contribute to flood reduction, but they do not have dependable reserved flood detention capacity. Table 2.4-5 lists pertinent data for the non-TVA owned dams and reservoirs. The locations of these dams are shown on Figure 2.1-1.

Flood control above the plant is provided largely by eight tributary reservoirs. Tellico Dam is counted as a tributary reservoir because it is located on the Little Tennessee River although, because of canal connection with Fort Loudoun Dam, it also functions as a main river dam. On March 15, near the end of the flood season, these provide a minimum of 3,937,400 acre-ft of detention capacity equivalent to 5.5 inches on the 13,508-square-mile area they control. This is 89% of the total available above the plant. The two main river reservoirs, Fort Loudoun and Watts Bar, provide 490,000 acre-ft equivalent to 2.4 inches on the remaining 3,802-square-mile area above Watts Bar Dam.

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

Chickamauga Dam, the headwater elevation of which affects flood elevations at the plant, has a drainage area of 20,790 square miles, 3,480 square miles more than Watts Bar Dam. There are seven major tributary dams (Chatuge, Nottely, Hiwassee, Apalachia, Blue Ridge, Ocoee No. 1 and Ocoee No. 3) in the 3,480-square-mile intervening watershed, of which four have substantial reserved capacity. On March 15, near the end of the flood season, these provide a minimum of 379,300 acre-ft equivalent to 5.9 inches on the 1,200-square-mile controlled area. Chickamauga Dam contains 345,300 acre-ft of detention capacity on March 15 equivalent to 2.8 inches on the remaining 2,280 square miles. Figure 2.4-3 (Sheet 1) shows the seasonal operating guide for Chickamauga.

Elevation-storage relationships for the reservoirs above the site and Chickamauga, downstream, are shown in Figure 2.4-4 (13 sheets).

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

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

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

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

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

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

2.4.2 Floods

2.4.2.1 Flood History

The nearest location with extensive formal flood records is 64 miles downstream at Chattanooga, Tennessee, where continuous records are available since 1874. Knowledge about significant floods extends back to 1826 based upon newspaper and historical reports. Flood flows and stages at Chattanooga have been altered by TVA's reservoir system beginning with closure of Norris Dam in 1936 and reaching essentially the present level of control in 1952 with closure of Boone Dam, the last major dam with reserved flood detention capacity constructed above Chattanooga prior to construction of Tellico Dam. Tellico Dam provides additional reserved flood detention capacity; however, the percentage increase in the total detention capacity above the Watts Bar site is small. Therefore, flood records for the period 1952 to date can be considered representative of prevailing conditions. Table 2.4-7 provides annual peak flow data at Chattanooga. Figure 2.4-5 shows the known flood experience at Chattanooga in diagram form. The maximum known flood under natural conditions occurred in 1867. This flood was estimated to reach elevation 716.3 ft at WBN site with a discharge of about 440,000 cfs. The maximum flood elevation at the site under present-day regulation would be approximately elevation 698 ft based on a maximum tailwater elevation of 698.23 ft at Watts Bar Dam located just upstream.

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

Date	Elevation, Ft	Discharge, cfs
February 2, 1957	No Record	157,600
November 19, 1957	No Record	151,600
March 13, 1963	694.75	167,700
December 31, 1969	693.28	167,300
March 17, 1973	696.95	184,800

May 28, 1973	695.24	175,200
April 5, 1977	694.79	181,600
May 8, 1984	698.23	208,400
April 20, 1998	694.67	167,500
May 7, 2003	694.17	153,100

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

2.4.2.2 Flood Design Considerations

TVA has planned the Watts Bar project to conform with Regulatory Guide 1.59 including position 2 as described herein.

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

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

The maximum PMF plant site flood level would result from the 7,980 square-mile Bulls Gap centered storm, as described in Section 2.4.3.

Wind waves based on an overland wind speed of 21 miles per hour were assumed to occur coincident with the flood peak. This would create maximum wind waves up to 2.2 ft high (trough to crest).

All safety-related facilities, systems, and equipment are housed in structures which provide protection from flooding for all flood conditions up to plant grade at elevation 728.0 ft. See Section 2.4.10 for more specific information.

Other rainfall floods will also exceed plant grade elevation 728.0 ft and require plant shutdown. Section 2.4.14 describes emergency protective measures to be taken in flood events exceeding plant grade.

Seismic and flood events could cause dam failure surges exceeding plant grade elevation 728.0 ft. 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.

At the Diesel Generator Building, the wind wave run up with wind setup during the PMF is determined to be 2.4 ft. The wind wave combined with the design basis probable maximum flood elevation provides a design basis flood elevation of 741.6 ft for the Diesel Generator Building, 0.4 ft below the operating floor.

Those Class 1E electrical system conduit banks located below the PMF plus wind wave runup and setup 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.

Equipment that is required during an external flood event is protected to the design basis flood elevation within the specific structure. The equipment in the Intake Pumping Station is protected to a design basis flood elevation of 741.7 ft. The Auxiliary and Control Buildings will flood at elevation 729.0 ft with protection to the design basis external flood elevation 739.7 ft.

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 one-hour rainfall of four inches. The one-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 PMP 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.[35]

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 PMP.

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. Figure 2.4-40b shows the Watts Bar general plan; Figure 2.4-40c shows the yard site grading and drainage system for flood studies 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 one-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 two-minute time intervals.

Computed maximum water surface elevations are below critical floor elevation 729.0 ft. 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 elevation 729.0 ft.

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 729.0 ft 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 ft and 728.25 ft respectively. Method (2) was used to route the inflow hydrograph through this pool from an initial elevation of 728.00 ft with outflow over the railroads. Maximum water surface elevations at the Turbine and Reactor Buildings were less than elevation 729.0 ft. 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 729.0 ft.

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 729.0 ft.

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

Flow from the 150-acre drainage area north of the site drains two ways: (1) 50 acres drain east through the double 96-inch culvert under the access railroad shown on Figure 2.4-40c and (2) drainage from the remaining 100 acres is diverted to the west through an 81-inch 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.[35] 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-inch 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 ft (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 two-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 ft. 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 ft, from elevation 735.28 ft to 735.4 ft. 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 729.0 ft.

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 two-minute time intervals through the double 96-inch culvert using standard storage routing techniques.

The maximum elevation reached at the culvert was 725.67 ft. 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-inch by 59-inch pipe arch will not be a problem.

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

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

2.4.3 Probable Maximum Flood (PMF) on Streams and Rivers

The guidance of Appendix A of Regulatory Guide 1.59 was followed in determining the PMF.

The PMF was determined from PMP for the watershed above the plant with consideration given to seasonal and areal variations in rainfall. Two basic storm situations were found to have the potential to produce maximum flood levels at WBN. These are (1) a sequence of storms producing PMP depths on the 21,400-square-mile watershed above Chattanooga and (2) a sequence of storms producing PMP depths in the basin above Chattanooga and below the five major tributary dams (Norris, Cherokee, Douglas, Fontana, and Hiwassee), hereafter called the 7,980-square-mile storm. The maximum flood level at the plant would be caused by the March PMP 7,980-square-mile storm with hydrologic failure of low margin dams. The flood level for the 21,400-square-mile storm would be slightly less.

Based on TVA's current River Operations (RO) procedures, TVA has evaluated the stability of 18 critical dams at PMF headwater/tailwater conditions. These dams are: Apalachia, Blue Ridge, Boone, Chatuge, Cherokee, Chickamauga, Douglas, Fontana, Fort Loudoun, Fort Patrick Henry, Hiwassee, Melton Hill, Norris, Nottely, South Holston, Tellico, Watauga, and Watts Bar. Other dams in the tributary system (Ocoee 1, Ocoee 2, Ocoee 3, Chilhowee, Calderwood, Cheoah, Mission, John Sevier, and Wilbur) were not evaluated and are postulated to fail during the event.

The hydrologic failure of low margin dams is postulated during the PMF for the Boone, Fort Patrick Henry, Melton Hill and Apalachia Dams. In both storms, the West Saddle Dike at Watts Bar Dam (crest elevation 752 ft) would be overtopped and is postulated to fail. No other failure would occur. Maximum discharge at the plant would be 1,158,956 cfs for the 7,980-square-mile storm. The resulting calculated PMF elevation at the plant would be 738.9 ft, excluding wind wave effects. An additional 0.3 ft of margin is provided for a design basis PMF at elevation 739.2 ft.

2.4.3.1 Probable Maximum Precipitation (PMP)

Probable maximum precipitation (PMP) for the watershed above Chickamauga and Watts Bar Dams for determining PMF has been defined for TVA by the Hydrometeorological Report No. 41.[4] This report defines depth-area-duration characteristics, seasonal variations, and antecedent storm potentials and incorporates orographic effects of the Tennessee River Valley. Due to the temperate climate of the watershed and relatively light snowfall, snowmelt is not a factor in generating maximum floods for the Tennessee River at the site.

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

The second storm described in Hydrometeorological Report No. 41[4] would produce PMP depths on the 7,980-square-mile watershed above Chattanooga and below the five major tributary dams. The isohyetal pattern for the 7,980-square-mile storm is not geographically fixed and can be moved parallel to the long axis, northeast and southwest, along the Tennessee Valley. The isohyetal pattern centered at Bulls Gap, Tennessee, would produce maximum rainfall on the upper part of the watershed and is shown in Figure 2.4-7.

Seasonal variations were also considered. Table 2.4-10 provides the seasonal variations of PMP. The March storm was evaluated because the PMP was maximum and surface runoff was also maximum.

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

A standard time distribution pattern was adopted for the storms based upon major observed storms transposable to the Tennessee Valley and in conformance with the usual practice of Federal agencies. The adopted distribution is within the limits stipulated in Chapter VII of Hydrometeorological Report No. 41[4]. This places the heaviest precipitation in the middle of the storm. The adopted sequence closely conforms to that used by the U.S. Army Corps of Engineers (USACE). A typical distribution mass curve resulting from this approach is shown in Figure 2.4-8.

The PMF discharge at WBN was determined to result from the 7,980-square-mile Bulls Gap centered storm, as defined in Hydrometeorological Report No. 41[4]. The PMP storm would occur in the month of March and would produce 16.17 inches of rainfall in three days. The storm producing the PMP would be preceded by a three-day antecedent storm producing 6.00 inches of rainfall, which would end three days prior to the start of the PMP storm. Precipitation temporal distribution is determined by applying the mass curve (Figure 2.4-8) to the basin rainfall depths in Table 2.4-11.

2.4.3.2 Precipitation Losses

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

Basin rainfall, precipitation excess, and API are provided in Table 2.4-11. The average precipitation loss for the watershed above Chickamauga Dam is 2.32 inches for the three-day antecedent storm and 1.87 inches for the three-day main storm. The losses are approximately 39% of antecedent rainfall and 12% of the PMP, respectively. The precipitation loss of 2.32 inches in the antecedent storm compares favorably with that of historical flood events shown in Table 2.4-12.

2.4.3.3 Runoff and Stream Course Model

The runoff model used to determine Tennessee River flood hydrographs at WBN is divided into 40 unit areas and includes the total watershed above Chickamauga Dam. Unit hydrographs are used to compute flows from the unit areas. The watershed unit areas are shown in Figure 2.4-9. The unit area flows are combined with appropriate time sequencing or channel routing procedures to compute inflows into the most upstream tributary reservoirs which in turn are routed through the reservoirs using standard routing techniques. Resulting outflows are combined with additional local inflows and carried downstream using appropriate time sequencing or routing procedures including unsteady flow routing.

Unit hydrographs were developed for each unit area for which discharge records were available from maximum flood hydrographs either recorded at stream gaging stations or estimated from reservoir headwater elevation, inflow, and discharge data using the procedures described by Newton and Vineyard.[5] For non-gaged unit areas synthetic unit graphs were developed from relationships of unit hydrographs from similar watersheds relating the unit hydrograph peak flow to the drainage area size, time to peak in terms of watershed slope and length, and the shape to the unit hydrograph peak discharge in cfs per square mile. Unit hydrograph plots are provided in Figure 2.4-10 (11 Sheets). Table 2.4-13 contains essential dimension data for each unit hydrograph.

The USACE Hydrologic Engineering Center River Analysis System software (HEC-RAS) performs one-dimensional steady and unsteady flow calculations. The HEC-RAS models are used in flood routing calculations for reservoirs in the Tennessee River System upstream of Wilson Dam to predict flood elevations and discharges for floods of varying magnitudes. Model inputs include previously calibrated geometry, unsteady flow rules, and inflows. Model calibration ensures accurate replication of observed river discharges and elevations for known historic events. Once calibrated, the model can be used to reliably predict flood elevations and discharges for events of varying magnitudes.

The TVA total watershed HEC-RAS model extends along the Tennessee River from Wilson Dam upstream to its source at the confluence of the Holston and French Broad Rivers, along the Elk River from its mouth at the Tennessee River to Tims Ford Dam, along the Hiwassee from its mouth at the Tennessee River to Chatuge Dam, along the Nottely River from its mouth at the Hiwassee River to Nottely Dam, along the Ocoee River from its mouth at the Hiwassee River to Blue Ridge Dam, along the Clinch River from its mouth at the Tennessee River to a gage at RM 159.8, along the Powell River from its mouth at the Clinch River to a gage at RM 65.4, along the Little Tennessee River from its mouth at the Tennessee River to a gage at RM 92.9, along the Tuckasegee River from its mouth at the Little Tennessee River to a gage at RM 12.6, along the Holston River from its mouth at the Tennessee River to its source at the confluence of the South Fork Holston River and the North Fork Holston River, along the South Fork Holston River from its mouth at the Holston River to South Holston Dam, along the Watauga River from its mouth at the South Fork Holston River to Watauga Dam, along the French Broad River from its mouth at the Tennessee River to a gage at RM 77.5, along the Nolichucky River from its mouth at the French Broad River to a gage at RM 10.3, along Cove Creek from its mouth at the Clinch River to RM 12.2, along Big Creek from its mouth at the Clinch River to RM 11.8, and along North Chickamauga Creek from its mouth at the Tennessee River to RM 12.82. The model also incorporates the Dallas Bay / Lick Branch rim leak and the Fort Loudoun canal by modeling these reaches. Figure 2.4-1a (2 sheets) shows the extent of the model, as well as the location of dams.

This TVA total watershed HEC-RAS model performs a continuous simulation of the Tennessee River system from the uppermost tributary reservoirs downstream through Wilson Dam. The composite model is used to perform flood simulations, such as the 7,980 and 21,400 square mile design storms.

Discharge rating curves are provided in Figure 2.4-11 (13 Sheets) for the reservoirs in the watershed at and above Chickamauga. The discharge rating curve for Chickamauga Dam is for the current lock configuration with all 18 spillway bays available. Above WBN, temporary flood barriers have been installed at Fort Loudoun Reservoir to increase the height of embankments and are included in the discharge rating curves for this dam. Increasing the height of embankments at this dam prevents embankment overflow and failure of the embankment. The vendor supplied temporary flood barriers were shown to be stable for the most severe PMF headwater/tailwater conditions using vendor recommended base friction values. A single postulated Fort Loudoun Reservoir rim leak north of the Marina Saddle Dam which discharges into the Tennessee River at Tennessee River Mile (TRM) 602.3 was added as an additional discharge component to the Fort Loudoun Dam discharge rating curve. Seven Watts Bar Reservoir rim leaks were added as additional discharge components to the Watts Bar Dam discharge rating curve. Three of the rim leak locations discharge to Yellow Creek, entering the Tennessee River three miles downstream of Watts Bar Dam. The remaining four rim leak locations discharge to Watts Creek, which enters Chickamauga Reservoir just below Watts Bar Dam.

2.4.3.3.1 PMF Determination

The HEC-RAS computer model is used to determine the PMF elevations and discharges at WBN. The HEC-RAS Model has been calibrated for each major reservoir to reasonably replicate observed river discharges and elevations for known historic events.

The hydrologic failure of low margin dams is postulated during the PMF (the 7,980 square-mile, Bull's Gap centered, March storm event) for the Boone, Fort Patrick Henry, and Melton Hill. The hydrologic failure of low margin dams is postulated during the 21,400 square mile, downstream centered, March storm event for the Boone, Fort Patrick Henry and Apalachia Dams. Failure sections for Boone, Fort Patrick Henry, Melton Hill and Apalachia Dams are shown in Figure 2.4-32. The discharge rating curve for Melton Hill is shown in Figure 2.4-33

The postulated dam failures occur when the peak headwater elevation occurs for each dam except for Fort Patrick Henry, which fails coincident with the arrival of the flood wave from the failure of Boone Dam. The failures are complete and instantaneous down to original ground elevation.

All upstream dams that have not been analyzed for stability are postulated to fail when headwaters reach the top of the dam, or when their peak headwater elevation occurs if the headwater does not reach the top of the dam. The postulated failure of these dams are complete and instantaneous down to original ground. This includes Ocoee 1, Ocoee 2, Ocoee 3, Chilhowee, Calderwood, Cheoah, Mission, Wilbur, and John Sevier.

Watts Bar West Saddle Dike (crest at elevation 752 feet) is postulated to fail when its peak headwater elevation occurs. The failure is conservatively assumed to be complete and instantaneous down to original ground. Discharge through the failed West Saddle Dike is controlled by a critical section immediately downstream (Figure 2.4-30). The discharge rating curves for the West Saddle Dike are shown in Figure 2.4-31.

Median pool levels for the appropriate season are used for the initial elevations at the beginning of the event. Use of median elevations is consistent with statistical experience and avoids unreasonable combinations of extreme events. Flood Operational Guides are used to operate the dams before gates are fully opened. Operational allowances are implemented at Fort Patrick Henry, Boone, Douglas, Fontana, Hiwassee, Norris, South Holston, and Watauga to maximize storage in these reservoirs for the controlling 7,980 square mile PMF storm.

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

Inflows were distributed for use in the composite HEC-RAS model of the Tennessee River System upstream of Wilson Dam. Inflow hydrographs presented in the inflow calculation[41] were used as an input to the composite HEC-RAS model. The hydrographs provide inflow data for individual basins in the Tennessee River System. Dam hydrographs are provided in Figure 2.4-25 (27 sheets).

Using the inflows, flood-routing simulations were performed for both the 21,400 square-mile and the 7,980 square-mile March storm events. Storm-specific decisions, such as if dam failure is necessary, were required to perform the simulations and are documented for each storm simulation. In general, a specific storm simulation was performed and the headwater/tailwater/discharge results were reviewed at each dam. If it was determined that a dam should fail, the model was modified to allow dam failure at a specified headwater and the model was re-run. The new results were analyzed and new failures simulated in an iterative fashion until the composite floodwave was routed downstream to WBN. Checking tools were used to verify the headwater/tailwater/discharge relationship predicted by HEC-RAS at each dam agreed with approved dam rating curves (DRC). DRCs are provided in Figure 2.4-11 (13 sheets). Volume checks were performed as well to ensure that volume was preserved in the model simulation.

Unsteady flow rules have been developed for the main Tennessee River and its tributaries and have been incorporated into the verified HEC-RAS unsteady flow model.

Elevation and discharge hydrographs for the 21,400 square-mile March storm event and 7,980 square-mile March storm are presented in Figure 2.4-23 (2 sheets). Hydrographs for dams in the PMF simulation are provided in Figure 2.4-25 (27 sheets). A summary of the results at the dams for the PMF is provided in Table 2.4-16.

2.4.3.3.2 Model Setup

The TVA total watershed HEC-RAS model extends along the Tennessee River from Wilson Dam upstream to its source at the confluence of the Holston and French Broad Rivers, along the Elk River from its mouth at the Tennessee River to Tims Ford Dam, along the Hiwassee from its mouth at the Tennessee River to Chatuge Dam, along the Nottely River from its mouth at the Hiwassee River to Nottely Dam, along the Ocoee River from its mouth at the Hiwassee River to Blue Ridge Dam, along the Clinch River from its mouth at the Tennessee River to a gage at RM 159.8, along the Powell River from its mouth at the Clinch River to a gage at RM 65.4, along the Little Tennessee River from its mouth at the Tennessee River to a gage at RM 92.9, along the Tuckasegee River from its mouth at the Little Tennessee River to a gage at RM 12.6, along the Holston River from its mouth at the Tennessee River to its source at the confluence of the South Fork Holston River and the North Fork Holston River, along the South Fork Holston River from its mouth at the Holston River to South Holston Dam, along the Watauga River from its mouth at the South Fork Holston River to Watauga Dam, along the French Broad River from its mouth at the Tennessee River to a gage at RM 77.5, and along the Nolichucky River from its mouth at the French Broad River to a gage at RM 10.3, along Cove Creek from its mouth at the Clinch River to RM 12.2, along Big Creek from its mouth at the Clinch River to RM 11.8, and along North Chickamauga Creek from its mouth at the Tennessee River to RM 12.82. The model also incorporates the Dallas Bay / Lick Branch rim leak and the Fort Loudoun canal by modeling these reaches. Figure 2.4-1a shows the extent of the model.

HEC-RAS models developed for the individual reservoirs had to be connected into a composite model in order to perform a continuous simulation of the Tennessee River system from TVA's uppermost tributary reservoirs downstream to Wilson Dam. The calibrated geometry for each reservoir was imported into the composite geometry file within HEC-RAS. HEC-RAS *Inline Structures* were added to model the dams and utilized data presented in DRC calculations[39] and tributary unsteady flow rules.[40] When an *Inline Structure* is used to model a dam, the headwater cross-section is located 0.01 mile upstream and the tailwater section 0.01 mile downstream of the dam. Reach lengths are modified to account for adjustments at the dam river station. HEC-RAS *Lateral Structures* are used at Apalachia, Chatuge, Douglas, Nottely, Ocoee No. 2, Ocoee No.3, and South Holston, Tellico and Watts Bar Dams, to model saddle dams and turbine discharges. After compiling the separate river geometry files into a composite model, the overall geometry file requires additional modifications before it is adequate for use. These modifications include the addition of junctions and inline structures, copying or interpolating additional cross-sections to allow for the application of inflows or to enhance model stability, and the addition of pilot channels. If a cross-section is copied or interpolated, the reach lengths associated with the new section are adjusted.

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

The unsteady flow rules incorporate the Flood Operational Guides[42], as they provide operating ranges of reservoir levels for the 32 reservoirs upstream of Wilson Dam. The rules reflect the flexibility provided in the guides to respond to unusual or extreme circumstances, such as the PMF event, through the use of primary guide and recovery curves. If the maximum discharge of the primary guide or recovery curve is exceeded, the discharges are from the DRCs.[39] The DRCs account for flow over other components such as non-overflow sections, navigation locks, tops of open spillway gates, tops of spillway piers, saddle dams, and rim leaks. Therefore, the DRCs and the flood operational guides define the dam discharge as a function of headwater elevation, tailwater elevation, and outlet configuration. If, during the event, the headwater elevation does not exceed the elevation of the operating deck, discharges are determined in accordance with the flood operational guides during the flood recession. In the event the operating deck is inundated, the dam rating curves determine the discharge during flood recession.

There are configuration parameters in each set of rules that are simulation specific. Model configuration parameters including failure elevation, gate position, operational allowances, armoring embankments, failure timing, and seismic triggers are initially set with input from the modeler. The HEC-RAS model is set-up to run all modeled rivers and reservoirs as a contiguous system to be run continuously. The model cannot be started and stopped in the middle of a simulation; however, some scenarios will require iterative simulations to determine necessary configuration parameters.

Inflows were distributed for use in the composite HEC-RAS model of the Tennessee River System upstream of Wilson Dam. Inflow hydrographs presented in the inflow calculation[41] were used as an input to the composite HEC-RAS model. The hydrographs provide inflow data for individual basins in the Tennessee River System.

2.4.3.3.3 Main Stem Geometry

The validated geometry for each reservoir was previously calibrated for use in the SOCH model. This validated geometry consists of Fort Loudoun, Tellico, Melton Hill, Watts Bar, Chickamauga, Nickajack, Guntersville, and Wheeler Reservoirs. Wilson Reservoir was not included in the validated geometry calculations for the SOCH runs. Therefore, Wilson Reservoir geometry, although a part of the "main stem," was provided by RO and verified in the same manner as the tributary geometry.

Cross-section data was obtained from the geometry verification calculations[44-51] and used to develop the HEC-RAS geometry. Cross-section data obtained from the geometry verification calculations were generally spaced about two miles apart on the "main stem." Generally, constricted channel locations were selected for cross-section locations. These smaller, constricted sections do not accurately represent the reach storage available (the storage capacity between cross sections) in an unsteady flow model. Therefore, a mathematical augmentation of selected cross sections with "off-channel" ineffective flow areas was performed, so the constricted geometry could accurately account for the additional reach storage available. To account for total reach storage, the reach storage contained between the "constricted" cross-sections was compared to the total reservoir volume information,[51] if available. If reservoir storage information was not available, such as at higher elevations of steep reaches, GIS obtained volumes were used for comparison to the model reach storage capacities. The reach storage between cross-sections was evaluated at incremental elevations. Reach storage was adjusted until the desired total cumulative storage was reached. Where additional reach storage was required, an additional ineffective flow area was added. A check of reach volume for the entire reservoir is also performed to verify that model volume is representative of the published actual reservoir volume.

2.4.3.3.4 Tributary Geometry

The tributary geometry has been developed for use in the HEC-RAS models. The tributary geometry was developed in one of two manners:

1. TVA RO developed the geometry. The geometry was verified in accordance with 10CFR50 Appendix B Quality Assurance requirements for use in safety related applications. The tributary geometry developed by RO included the following: Apalachia Reservoir, Ocoee River, Toccoa River, Blue Ridge Reservoir, Boone Reservoir, Watauga River, Wilbur Reservoir, South Fork Holston River, Holston River, French Broad River, Nolichucky River, Little Tennessee River, Fort Patrick Henry Reservoir, Hiwassee River and Reservoir, Nottely River, and the Elk River.

2. If no geometry previously existed, the geometry was generated and verified for nuclear application. The tributaries that required geometry generation and verification are: Fontana Reservoir, Tuckasegee River, Norris Reservoir, Powell River, Big Creek, and Cove Creek.

Verification of RO Developed Geometry

The verification of the tributary geometry previously developed by TVA RO included verification of the location and orientation of each section, the Manning's n values, the cross-section shape with respect to historic channel geometry, the underwater portion of the section, and storage volume between sections.

The location of each cross-section provided by RO and its orientation were examined. Adjustments were made to the cross-sections and additional cross-sections were added if required to better represent the river. The RO provided cross-sections were compared to geographic information system (GIS) generated cross-sections above the water surface and historical channel geometry below the water surface elevation.

The revised cross-sections were plotted with historic channel geometry cross-sections and the width at the water surface of the new cross-section was compared to and verified against the historic cross-sections. The composite GIS/historic channel geometry cross-sections were then compared to those developed by RO.

When the shape of each cross-section had been verified, additional geometry data including Manning's n values, ineffective flow areas, and flow lengths were evaluated and adjustments or corrections were made if necessary. Manning's n values were confirmed using aerial photographs. USGS topographic maps were used to identify and confirm ineffective flow areas, as well as to confirm reach lengths.

Generation and Verification of New Geometries

Development of the HEC-RAS geometry for Fontana Reservoir, Tuckasegee River, Norris Reservoir, Powell River, Big Creek and Cove Creek were developed by extracting cross-sections from a GIS TIN and comparing the cross-sections to historic cross-sections. Available stream centerline and elevation data were compiled in GIS. USGS topographic maps were examined to identify desired cross-section locations. Once the cross-section locations were established, generic Manning's n values were added in the HEC-RAS geometry. The revised cross-sections were plotted with historic channel geometry cross-sections. The width at the water surface of the new cross-section was compared to and verified against the historic cross-sections.

When the shape of each cross-section had been verified, additional geometry data including Manning's n values, ineffective flow areas, and flow lengths were evaluated and adjustments or corrections were made if the data were not representative of the cross-section. Manning's n values were confirmed using aerial photographs. USGS topographic maps were used to identify and confirm ineffective flow areas, as well as confirm reach lengths.

Once the cross-sections were developed and/or verified, a reach storage augmentation procedure was performed so the model storage accurately reflects the actual reach storage capacities. For more information on the reach storage augmentation procedure see Section 2.4.3.3.3.

2.4.3.3.5 Calibration

Model calibration is performed to adjust model parameters so that the model will accurately predict the outcome of a known historic event. In the case of the HEC-RAS models, the model results must accurately replicate observed elevations and discharges for known historic flood events. A calibrated model is therefore considered reliable at predicting the outcome of events of other magnitudes.

2.4.3.3.5.1 Main Stem River

The main river model uses the USACE HEC-RAS software. The main river model extends from Wilson Dam upstream to Norris, Cherokee, Douglas, and Chilhowee Dams, and the Charleston Gage at River Mile (RM) 18.9 on the Hiwassee River. The nine reservoirs upstream of Wilson Dam (Wilson, Wheeler, Guntersville, Nickajack, Chickamauga, Watts Bar, Tellico, Fort Loudoun, and Melton Hill) were individually calibrated for use to reliably predict flood elevations and discharges for events of varying magnitudes. The reservoirs that impact PMF elevations at the Watts Bar site are: Chickamauga, Watts Bar, Tellico, Fort Loudoun, and Melton Hill.

Initial unsteady-flow runs are conducted to replicate the historic flood events. Initial unsteady-flow runs are conducted for each individual reservoir's model. The initial runs used channel roughness (Manning's n) values from the calibrated SOCH models in an attempt to replicate the historic flood events. Following the initial runs, roughness values for each of the model segments were evaluated and adjusted as needed. The model was rerun and the results were again compared to the observed elevations at the gage stations. The process was repeated in an iterative fashion until good agreement was reached between the HEC-RAS computed elevations and the observed gage elevations. Adjustments to the roughness values in the HEC-RAS models were kept within a reasonable range for the ground coverage in the vicinity of the cross section.

In general, the computed peak elevations are within one foot, but not below, the observed gage elevations. In some cases, the computed elevations are more than 1 foot above the observed gage elevations; however this was necessary to avoid impacts to the computed peak elevations at other gage locations.

A schematic of the model for Watts Bar Reservoir is shown in Figure 2.4-15. The calibration results of the March 1973 flood is shown in Figure 2.4-16 (2 Sheets) and the calibration results of the May 2003 is shown in Figure 2.4-17 (2 Sheets).

A schematic of the unsteady flow model for Chickamauga Reservoir is shown in Figure 2.4-18. The calibration results of the March 1973 flood is shown in Figure 2.4-19 (3 Sheets) and the calibration results of the May 2003 is shown in Figure 2.4-20 (3 Sheets).

The configuration for the Fort Loudoun-Tellico complex is shown by the schematic in Figure 2.4-12. The Fort Loudoun Tellico complex was verified by two different methods as follows:

> Using the available data for the March 1973 flood on Fort Loudoun Reservoir and for the French Broad and Holston rivers. The verification of the 1973 flood is shown in Figure 2.4-13 (4 Sheets). Because there were limited data to verify against on the French Broad and Holston Rivers, the steady state HEC-RAS model was used to replicate the Federal Emergency Management Agency (FEMA) published 100- and 500-year profiles. Tellico Dam was not closed until 1979, thus was not in place during the 1973 flood for verification.
>
> Using available data for the May 2003 flood for the Fort Loudoun Tellico complex. The verification of the May 2003 flood is shown in Figure 2.4-14 (5 Sheets). The Tellico Reservoir steady state HEC-RAS model was also used to replicate the FEMA published 100- and 500-year profiles.

In addition to roughness adjustments, the calibration sequence is used to verify that an adequate time step and appropriate mixed flow parameters are selected. To verify the time step, a series of simulations were conducted using PMF flows and varying time steps. The results indicated that a time step of five minutes provides for a stable simulation and the results are comparable with shorter time steps. Above five minutes, there is more variation in the results. The mixed flow regime option is used in the HEC-RAS models because the topographic relief, dam failures, and high flows evaluated for the PMF could produce supercritical flow or hydraulic jumps. Higher values of the mixed flow regime parameters produce more accurate results, but if too high can cause model instability. A comparison of water surface errors between simulations with varying parameters is used to verify appropriate values for the parameters are selected.

Once each reservoir's model was adequately calibrated, they were combined into a composite model of the entire main stem for use in a continuous run simulation.

This calibration process provided model results that satisfactorily reproduced the two historic floods (1973 and 2003). The HEC-RAS unsteady flow model accurately replicated observed gage elevations and discharges for two large historic flood events. Therefore, the HEC-RAS unsteady flow model of main stem reservoirs upstream of Wilson Dam can be used to reliably predict flood elevations and discharges for events of other magnitudes and is adequate for use in predicting flood elevations and discharges for the PMF.

2.4.3.3.5.2 Tributary Calibration

Tributaries were calibrated using a combination of steady-state and unsteady simulations. Steady-state calibration was to Federal Emergency Management Agency (FEMA) 100 and 500 year flood profiles or, if not available, project manuals. Unsteady calibration, at a minimum, utilized the worst two historical storms experienced on each tributary, as tabulated below:

Tributary	Calibration – Largest Recorded Storms
Hiwassee River between Hiwassee and Apalachia Dams	March 1994 and April 1998
Ocoee River and Toccoa River from Ocoee 1 to Blue Ridge Dam	April 1998, May 2003, and September 2004
Boone Reservoir	March 2002 and November 2003
Wilbur Reservoir	March 2002 and November 2003
Cherokee Reservoir, Holston and South Fork Holston Rivers	March 2002 and February 2003
French Broad River and Nolichucky River	May 2003 and September 2004
Little Tennessee River and Tuckasegee River	May 2003 and September 2004
Fort Patrick Henry Reservoir	March 2002 and November 2003
Hiwassee River and Nottely River	May 2003 and December 2004
Hiwassee River below Apalachia Dam and Ocoee River below Ocoee #1 Dam	May 2003 and September 2004
Clinch River above Norris Dam	March 2002 and February 2003
Elk River, Subbasin 1	March 2002 and February 2004
Elk River, Subbasins 2 and 3	February 2004 and January 2006
Elk River, Subbasins 4 and 5	March 1973 and December 2004

Initial tributary geometry segments were obtained from the HEC-RAS Tributary Geometry Development calculation.[43] The required local inflows and associated distribution for unsteady flow modeling were determined from the HEC-RAS Model Calibration and Model Set-up calculations.[38, 53]

In most cases, tributary segments were calibrated to FEMA 100-Year and the 500-Year flood profiles. In some cases, flood profiles were available in published flood insurance studies, in others the profiles were reproduced by running HEC-RAS or HEC-2 files from various TVA studies (e.g., reservoir sedimentation studies, floodplain models, and FEMA flood studies). Some tributary segments only had one FEMA profile available. Some did not have any profiles, in those cases other steady-state profile data were used such as those provided in project manuals. Roughness (Manning's n) values were adjusted iteratively until the steady-state computed profiles were in good agreement with the FEMA or project manual profiles.

Following the steady-state calibration procedure, unsteady calibration simulations were performed on the tributary models, similar to the main stem calibration process. Observed historic flood event data were obtained from various available sources such as unit hydrograph calculations or gage data. Results of the unsteady flow simulations were compared to the observed elevation and discharge hydrographs. If the computed results were in good agreement with the observed hydrographs, the calibration was considered complete. In some cases, Manning's n values required further adjustment after comparison of unsteady-flow results. In those cases, the steady-state profiles were rerun to verify agreement with FEMA profiles.

This calibration process provided model results that, through the combination of reach storage, unit hydrograph runoff, and inflow distribution, satisfactorily reproduced historic floods and available steady state profiles (FEMA or project manual flood profiles) for the tributary reaches. The HEC-RAS unsteady flow model produced elevations and discharges for large historic flood events appropriate for the intended use of predicting elevations at WBN. Therefore, the HEC-RAS unsteady flow model of the tributaries of the Tennessee River System can be used with the model of the greater Tennessee River System to reliably predict flood elevations and discharges for events of other magnitudes and is adequate for use in predicting flood elevations and discharges for the PMF.

2.4.3.4 Probable Maximum Flood Flow

The PMF discharge at WBN was determined to be 1,158,956 cfs. This flood would result from the 7,980-square-mile storm in March with a Bulls Gap centered storm pattern (Figure 2.4-7).

The PMF discharge hydrograph is shown in Figure 2.4-23. The West Saddle Dike at Watts Bar Dam would be overtopped and is postulated to fail. The discharge from the failed West Saddle Dike flows into Yellow Creek which joins the Tennessee River at mile 526.82, 1.18 miles below WBN.

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

Concrete Section Analysis

For concrete dam sections, global stability was analyzed for the maximum headwater and corresponding tailwater levels that would occur in the PMF as described in Section 2.4.3. Concrete gravity dams were evaluated for static PMF loading. The force and moment equilibrium must be maintained without exceeding the limits of concrete, concrete-rock interface and foundation strength. The tensile strength of the concrete-rock interface is assumed to be zero. Theoretical base cracking is allowed provided that the crack stabilizes, the resultant of all forces remains within the base of the dam, and adequate sliding factor of safety is obtained. The acceptable factors of safety for sliding are 1.3, where cohesion is not considered, and 2.0, where cohesion is considered. [54]

The concrete dams that were outside of this acceptance criteria were postulated for failure within the model: Fort Patrick Henry Dam (total failure), Boone Dam (total failure), Melton Hill Dam (total failure in 7,980 square-mile storm) and Appalachia Dam (total failure in 21,400 square-mile storm).

Modifications performed in support of WBN Unit 2 licensing are credited for the following concrete structures: Watts Bar Dam east flood wall, Watts Bar Dam neck of the non-overflow section, Tellico Dam neck of the non-overflow section, Fort Loudoun Dam non-overflow section, Cherokee non-overflow section and Douglas non-overflow section.

Embankment Structures

For embankment dam sections, global stability was analyzed for the maximum headwater and corresponding tailwater levels that would occur in the PMF as described in Section 2.4.3. Conventional limit equilibrium methods of slope stability analysis are used to investigate the equilibrium of a soil mass tending to move downslope under the influence of gravity. A comparison is made between forces, moments, or stresses tending to cause instability of the mass and those that resist instability. The acceptable factor of safety is 1.4.

Modifications performed in support of WBN Unit 2 licensing are credited for the following embankment structures: Watts Bar Dam east embankment; Watts Bar West Saddle Dike; Douglas Saddle Dams; Cherokee Dam embankments, Fort Loudoun Dam embankments, and Tellico Dam embankments.

Spillway Gates

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

Waterborne Objects

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

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

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

Lock Gates

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

2.4.3.5 Water Level Determinations

The controlling PMF elevation at WBN was determined to be 738.9 ft, produced by the 7,980-square-mile storm in March. An additional 0.3 ft of margin is provided for a design basis PMF at elevation 739.2 ft. The PMF elevation hydrograph is shown in Figure 2.4-23. Elevations were computed concurrently with discharges using the unsteady flow reservoir model described in Section 2.4.3.3. The PMF profile together with the regulated maximum known flood, median summer elevation and bottom profiles along a four-mile reach of the Chickamauga Reservoir which encompasses the plant location, is shown in Figure 2.4-26.

2.4.3.6 Coincident Wind Wave Activity

Some wind waves are likely when the PMF crests at WBN. The flood would be near its crest for a day beginning about 2 days after cessation of the probable maximum storm (Figure 2.4-23). The day of occurrence would be in the month of March or possibly the first week in April.

Figure 2.4-27 shows the main plant general grading plan. The Diesel Generator Buildings to the north and the pumping station to the southeast of the main building complex must be protected from flooding to assure plant safety. The Diesel Generator Buildings operating floors are at elevation 742.0 ft which are above the maximum computed elevation including wind wave runup. The equipment in the Intake Pumping Station is protected to elevation 741.7 ft. The Auxiliary and Control Buildings are allowed to flood. All equipment required to maintain the plant safely during the flood is either designed to operate submerged, is located above the maximum flood level, or is otherwise protected. Those safety-related facilities, systems, and equipment located in the containment structure are protected from flooding by the Shield Building structure with those accesses and penetrations below the maximum flood level designed and constructed as watertight elements.

The maximum effective fetches for the structures are shown on Figure 2.4-28. Effective fetch accounts for the sheltering effect of several hills on the south riverbank which become islands at maximum flood levels. The maximum effective fetch in all cases, except for the west face of the Intake Pumping Station occurs from the northeast or east northeast direction. The maximum effective fetch for the west face of the Intake Pumping Station occurs from the west direction. The Diesel Generator Building maximum effective fetch is 1.1 miles, and the critical west face of the Intake Pumping Station maximum effective fetch is 1.3 miles. The maximum effective fetch for the Auxiliary, Control, and Shield Buildings is 0.8 miles.

For the WBN site, 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 two-year extreme wind, as specified in ANSI Standard N170-1976, an appendix to Regulatory Guide 1.59. Further, the necessary 30-minute wind data are not available. To determine applicability of Chattanooga winds at the Watts Bar plant, a Kolmogorov-Smirnov (K-S) statistical test was applied to cumulative frequency distributions of daily maximum hourly winds for each direction at Chattanooga and Watts Bar. The winds compared were those recorded at Chattanooga during the period 1948-74 (the period when the necessary triple-register records were available for analysis) and the Watts Bar record. A concurrent record is not available; however, the K-S test showed that (except for the noncritical east direction) the record of daily maximum hourly velocities at Chattanooga were equal to or greater than that at Watts Bar. From this analysis it was concluded that use of the Chattanooga wind records to define seasonal maximum winds at the Watts Bar site is conservative.

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

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

Computation of wind waves used the procedures of the Corps Of Engineers.[14] Wind speed was adjusted based on the effective fetch length for over water conditions. For the Diesel Generator Building, the adjusted wind speed is 23.8 miles per hour. The Intake Pumping Station maximum adjusted wind speed is 24.2 miles per hour for the critical west face. For the Auxiliary, Control, and Shield Buildings the adjusted wind speed is 23.4 miles per hour.

For waves approaching the Diesel Generator Building, the maximum wave height (average height of the maximum 1 percent of waves) would be 1.7 ft high, crest to trough, and the significant wave height (average height of the maximum 33-1/3 percent of waves) would be 1.0 ft high, crest to trough. The corresponding wave period is 2.0 seconds. For the Intake Pumping Station, the maximum wave height would be 2.2 ft and the significant wave height would be 1.3 ft, with a corresponding wave period of 2.3 seconds. For the critical west face, the maximum wave height would be 1.9 ft high, and the significant wave height would be 1.1 ft high. The corresponding wave period is 2.1 seconds. The maximum wave height approaching the Auxiliary, Control, and Shield Buildings would be 1.5 ft high, and the significant wave height would be 0.9 ft high. The corresponding wave period is 1.9 seconds.

Computation of wind setup used the procedures of the Corps of Engineers.[14] The maximum wind setup is 0.1 ft for all structures. Computation of runup used the procedures of the Corps of Engineers.[14] At the Diesel Generator Building, corresponding runup on the earth embankment with a 4:1 slope is 2.3 ft and reaches elevation 741.3 ft, including wind setup. The runup on the critical west face of the Intake Pumping Station is 2.1 ft and reaches elevation 741.1 ft including wind setup. The configuration of the north face of the Intake Pumping Station, opposite of the intake channel, allows higher runup of 3.4 ft. The remaining south and east faces allow runup of 2.4 ft. However, there are no credible entry points to the structure on the north, south, or east faces. Therefore, the runup on these faces is discounted. The runup on the walls of the Auxiliary, Control, and Shield Buildings is 1.7 ft and reaches elevation 740.7 ft, including wind setup.

Runup at the Diesel Generator Building is maintained on the slopes approaching the structure and is below all access points to the building. Runup has no consequence at the Shield Building because all accesses and penetrations below runup are designed and constructed as watertight elements.

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

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

The dynamic effect of breaking waves on the walls of safety-related structures was investigated using a method developed by D. D. Gaillard and D. A. Molitar[16]. The concrete and reinforcing stresses were found to be less than the allowable stresses.

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

2.4.4 Potential Dam Failures, Seismically Induced

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

The plant site and upstream reservoirs are located in the Southern Appalachian Tectonic Province and, therefore, subject to moderate earthquake forces with possible attendant failure. 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 WBN against failure by floods caused by the assumed failure of dams due to seismic forces. To assure that safe shutdown of the WBN 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.

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. The TVA Dam Safety Program (DSP), which is consistent with the Federal Guidelines for Dam Safety[37], conducts technical studies and engineering analyses to assess the hydrologic and seismic integrity of agency dams and verifies that they can be operated in accordance with Federal Emergency Management Agency (FEMA) guidelines. These guidelines were developed to enhance national dam safety such that the potential for loss of life and property damage is minimized. As part of the TVA DSP, inspection and maintenance activities are carried out on a regular schedule to confirm the dams are maintained in a safe condition. Instrumentation to monitor the dams' behavior was installed in many of the dams during original construction and other instrumentation has been added since. Based on the implementation of the DSP, TVA has confidence that its dams are safe against catastrophic destruction by any natural forces that could be expected to occur.

2.4.4.1 Dam Failure Permutations

There are 12 major dams above WBN whose failure could influence plant site flood levels. Dam locations with respect to the WBN site are shown in Figure 2.4-2. 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. These were examined individually, and in combinations, to determine if failure might result from a seismic event and, if so, would failure concurrent with storm runoff create maximum flood levels at the plant.

The procedures referred to in Regulatory Guide (RG) 1.59, Appendix A, were followed for evaluating potential flood levels from seismically induced dam failures. In accordance with this guidance, seismic dam failure is examined using the two specified alternatives:

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

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

The OBE and SSE are defined in 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.

From the seismic dam failure analyses made for TVA's operating nuclear plants, it was determined that five separate, combined events have the potential to create flood levels above plant grade at WBN. These events are as follows:

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

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

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

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

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

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

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. 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.

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

Flood Routing

The runoff model of Attachment 2.4A was used to reevaluate potentially five critical seismic events involving dam failures above the plant. Other events addressed in earlier studies (the postulated OBE single failures of Watts Bar and Fort Loudoun; the postulated SSE combination failure of Fontana and Douglas, the postulated SSE combination failure of Fontana, Fort Loudoun, and Tellico; the SSE combination failure of Norris, Douglas, Fort Loudoun and Tellico, and the single SSE failure of Norris) produced plant site flood levels sufficiently lower than the controlling events and therefore were not re-evaluated.

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

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

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 in the spillway base, and the powerhouse base. Original results are given in Figure 2.4-68 and were not updated in the current analysis. Dynamic analysis of the concrete structures resulted in the determination that the base acceleration is amplified at levels above the base. The original slip circle analysis of the earth embankment section results in a factor of safety of 1.52, and the embankment is judged not to fail.

For the condition of peak discharge at the dam for one-half the PMF 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 PMF with gates in the wide-open position. Hence, bridge failure will cause no adverse effect on the flood.

Previous evaluations determined that if the dam was postulated to fail from embankment overtopping in the most severe case (gate opening prevented by bridge failure) that the resulting elevations at WBN would be several feet below plant grade elevation 728.0 ft. Therefore, this event was not reevaluated.

Fort Loudoun Dam

Stability analyses of Fort Loudoun Dam powerhouse and spillway sections result in the judgment that these structures will not fail. The analyses show low base stresses, with near two-thirds of the base in compression. The original results, given in Figure 2.4-71, were not updated for the current analysis.

Slip circle analysis of the earth embankment results in a factor of safety of 1.26, and the embankment is judged not to fail. The original results, given in Figure 2.4-72, were not updated in the current analysis.

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

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

Tellico Dam

Although, not included in the original analyses for TVA's operating nuclear plants, Tellico is judged to fail completely because the seismic stability analysis of Tellico is not conclusive. No hydrologic results are given for the single failure of Tellico because the simultaneous failure of Tellico Dams with other dams discussed under multiple failures, is more critical.

Norris Dam

Although an evaluation made in 1975 by Agbabian Associates concluded that Norris Dam would not fail in an OBE (with one-half PMF) or SSE (with 25-year flood), the original study postulated failure in both seismic events. To be consistent with prior studies, Norris was conservatively postulated to fail. Figure 2.4-76 shows the postulated condition of the dam after OBE failure. 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.

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

Cherokee Dam

Results of the original Cherokee Dam stability analysis for a typical spillway block are shown in Figure 2.4-77. The spillway is judged stable at the foundation base elevation 900.0 ft. Analyses made for other elevations above elevation 900.0 ft, but not shown in Figure 2.4-77, indicate the resultant of forces falls outside the base at elevation 1010.0 ft. The spillway is assumed to fail at this elevation.

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

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

Results of the original 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, Douglas, and Tellico Dams discussed under multiple failures, is more critical.

<u>Douglas Dam</u>

Results of the original 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 ft higher than Cherokee, but the amplification of the rock surface acceleration is the same. Therefore, based on the Cherokee analysis, it is judged that the Douglas spillway will fail at elevation 937.0 ft, which corresponds to the assumed failure elevation of the Cherokee spillway.

The Douglas non-overflow dam is similar to that at Cherokee and is embedded in fill to elevation 927.5 ft. It is considered stable below that elevation. However, based on the Cherokee analysis, it is assumed to fail above the fill line. The abutment non-overflow blocks 1-5 and 29-35, being short blocks, are considered able to resist the 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 original 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, Douglas, and Tellico Dams as discussed later under multiple failures, is more critical.

Fontana Dam

The original hydrological analysis used a conservative seismic failure condition for Fontana Dam. A subsequent review which takes advantage of later earthquake stability analysis and dam safety modifications performed for the TVA DSP has defined a conservative but less restrictive seismic failure condition at Fontana. This subsequent review used a finite element model for the analysis and considered the maximum credible earthquake expected at the Fontana Dam site.

Figure 2.4-83 shows the part of Fontana Dam judged to remain in its original position after postulated failure.

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

Multiple Failures

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

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

Figure 2.4-83 shows the postulated condition of Fontana for the OBE event. Tellico was conservatively postulated to completely fail.

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

Watts Bar Dam headwater would reach 756.13 ft, 13.87 ft below the top of the embankment. The West Saddle Dike at Watts Bar Dam with top elevation of 757.0 ft* would not be overtopped. The peak discharge at WBN would be 743,668 cfs. The elevation at WBN would be 720.65 ft, 7.35 ft below plant grade elevation 728.0 ft.

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

* Modifications to the Watts Bar West Saddle Dike result in a crest elevation of 752.0 ft. An evaluation of the impacts of this modification on the controlling seismic scenario was performed and determined that the results of the scenario were not significantly affected.

Fontana, Tellico, Hiwassee, Apalachia and Blue Ridge Dams could fail when the OBE is located within a flattened oval-shaped area located between Fontana and Hiwassee Dams (Figure 2.4-112). Failure scenarios for Fontana, Tellico, Hiwassee, Apalachia, and Blue Ridge Dams include postulated simultaneous failure of non-TVA dams on the Little Tennessee River, Cheoah, Calderwood and Chilhowee Dams and on its tributaries, Nantahala and Santeetlah Dams.

Based on previous attenuation studies, the OBE event produces maximum ground accelerations of 0.09 g at Fontana, 0.09 g at Hiwassee, 0.07 g at Apalachia, 0.08 g at Chatuge, 0.05 g at Nottely, 0.03 g at Ocoee No.1, 0.04 g at Blue Ridge, 0.04 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. Figure 2.4-83 shows the postulated condition of Fontana Dam after failure. Hiwassee, Apalachia, Blue Ridge, and Tellico Dams are postulated to completely fail. Chatuge Dam is judged not to fail in this defined OBE event.

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

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

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

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

The peak discharge at the WBN site produced by the OBE failure of Fontana, Tellico, Hiwassee, Apalachia, and Blue Ridge coincident with the one-half PMF is 742,572 cfs. The peak elevation is 722.01 ft, 5.99 ft below 728.0 ft plant grade.

* Modifications to the Watts Bar West Saddle Dike result in a crest elevation of 752.0 ft. An evaluation of the impacts of this modification on the controlling seismic scenario was performed and determined that the results of the scenario were not significantly affected.

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

Figure 2.4-76 shows the postulated condition of Norris Dam for the OBE event. Tellico was conservatively postulated to completely fail in this event.

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

The headwater at Watts Bar Dam would reach elevation 762.96 ft, 6.54 ft below top of dam. The West Saddle Dike at Watts Bar with top at elevation 757.0 ft* would be overtopped and breached. A complete washout of the dike was assumed. Chickamauga headwater would reach 701.05 ft, 4.95 ft below top of dam. The embankments at Nickajack Dam would be overtopped but was postulated not to breach which is conservative.

The peak discharge at the WBN site produced by the OBE failure of Norris and Tellico Dams coincident with the one-half PMF is 917,284 cfs. The peak elevation is 728.67 ft, 0.67 ft above 728.0 ft plant grade.

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

Figures 2.4-79 and 2.4-82 show the postulated condition after failure of Cherokee and Douglas Dams, respectively. Tellico was conservatively postulated to completely fail.

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

The peak discharge at the WBN site produced by the OBE failure of Cherokee, Douglas, and Tellico Dams with the one-half PMF is 902,687 cfs. The peak elevation is 729.07 ft, 1.07 ft above 728.0 ft plant grade.

<u>SSE Concurrent With 25-Year Flood</u>

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 Watts Bar Nuclear plant based on previous analyses would be below elevation 728.0 ft plant grade.

Fort Loudoun Dam

Results of the original 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 750.0 ft as well as the bridge supported by the spillway piers.

The results of the original slip circle analysis for the highest portion of the embankment are shown on Figure 2.4-87. 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 hydrologic routing for the single failure of Tellico is made because its simultaneous failure with other dams is more critical as discussed later in this sub-paragraph.

Norris Dam

Although an evaluation made in 1975 by Agbabian Associates concluded that Norris Dam would not fail in the SSE (with 25 year flood), Norris Dam was postulated to fail. The resulting debris downstream would occupy a greater span of the valley cross section than would the debris from the OBE but with the same top level, elevation 970.0 ft. Figure 2.4-90 shows the part of the dam judged to fail and the location and height of the resulting debris.

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

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

Cherokee

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

Douglas

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

Multiple Failures

TVA considered the following multiple SSE dam failure combinations.

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

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

Figure 2.4-91 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 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 the seismic event at upstream tributary dams occurred, the crest of the 25 year flood would likely have passed Fort Loudoun and flows would have been reduced to turbine capacity. Hence, spillway gates would be closed. As stated before, it is believed that multiple dam failure is extremely remote, and it seems reasonable to exclude Fontana on the basis of being the most distant in the cluster of dams under consideration. For the postulated failures of Norris, Cherokee, and Douglas the portions judged to remain and debris arrangements are as given in Figures 2.4-90, 2.4-79, and 2.4-82, respectively. Tellico is conservatively postulated to completely fail.

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

The maximum discharge at WBN would be 979,385 cfs. The elevation at the plant site would be 731.17 ft, 3.17 ft above 728.0 ft plant grade. This is the highest flood elevation resulting from any combination of seismic events.

The flood elevation hydrograph at the plant site is shown on Figure 2.4-114.

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

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

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

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 Figures2.4-90, 2.4-82, 2.4-88 and 2.4-89 for single dam failure. Fort Loudoun and Tellico were postulated to fail completely as the portions judged to remain are relatively small. This combination was not reevaluated.

Douglas and Fontana Dams were postulated to fail simultaneously. Figure 2.4-94 shows the location of an SSE and its attenuation, which produces 0.14 g at Douglas, 0.09 g at Fontana,

* Modifications to the Watts Bar West Saddle Dike result in a crest elevation of 752.0 ft. An evaluation of the impacts of this modification on the controlling seismic scenario was performed and determined that the results of the scenario were not significantly affected.

0.07 g at Cherokee, 0.05 g at Norris, 0.06 g at Fort Loudoun and Tellico, and 0.03 g at Watts Bar. For the postulated failures of Douglas and Fontana Dams, the portions judged to remain and the debris arrangements for Douglas Dam are as given in Figures 2.4-82 and 2.4-83 for single dam failure. Fort Loudoun and Watts Bar Dams have previously been judged not to fail for the OBE (0.09 g). Postulation of Tellico failure in this combination has not been evaluated but is bounded by the SSE failure of Norris, Cherokee, Douglas and Tellico.

2.4.4.2 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 Attachment 2.4A, the models described below were used to develop the outflow hydrographs from the postulated dam failures. The HEC-HMS storage routing was used to compute the outflow hydrograph from the postulated failure of each dam except main river dams. In the case of dams which were postulated to fail completely (Hiwassee, Apalachia and Blue Ridge), HEC-RAS or SOCH was used to develop the outflow hydrograph. For Tellico Dam, the complete failure was analyzed with the SOCH model.

The failure time and initial reservoir elevations for each dam were determined from a pre-failure TRBROUTE analysis. HEC-HMS was used to develop the post failure outflow hydrographs based on the previously determined dam failure rating curves. The outflow hydrographs were validated by comparing the HEC-HMS results with those generated by simulations using TRBROUTE.

2.4.4.3 Water Level at Plant Site

The unsteady flow analyses of the five postulated combinations of seismic dam failures coincident with floods analyzed yields a maximum elevation of 731.17 ft at WBN, excluding wind wave effects. The maximum elevation would result from the SSE failure of Norris, Cherokee, Douglas, and Tellico Dams coincident with the 25-year flood postulated to occur in June when reservoir levels are high. Table 2.4-14 provides a summary of flood elevations determined for the five failure combinations analyzed.

Coincident wind wave activity for the PMF is described in Section 2.4.3.6. Wind waves were not computed for the seismic events, but superimposed wind wave activity from guide specified two-year wind speed would result in water surface elevations several ft below the PMF elevation 738.9 ft described in section 2.4.3. For the design basis flood level, see Section 2.4.14.1.

2.4.5 Probable Maximum Surge and Seiche Flooding

Chickamauga Lake level during non-flood conditions would not exceed elevation 682.5 ft, 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 728.0 ft, some 45 ft 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 WBN 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 plant site. 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 ft 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 ft 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 ft 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 ft 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 ft 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 ft and is 50 ft 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 ft and a normal maximum of 682.5 ft. The minimum average elevation of the reservoir bottom at the intake channel is 656 ft and the elevation of the intake channel bottom is 660 ft. The 15 ft 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 system (ERCW). The maximum flow requirement for the plant for all purposes is 178 cfs based on four ERCW pumps and six RCW pumps in service.

The protection of the intake channel slopes from wind-wave activity is afforded by the placement of riprap, shown in Figure 2.4-99 in accordance with TVA design standards, from elevation 660.0 ft to elevation 690.0 ft. 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 RO 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 675.0 ft and would occur in the winter flood season as a result of Chickamauga Reservoir operation. 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 one-day flow of 3300 cfs downstream at Chattanooga might occur in a given year. At the plant site the corresponding minimum one-day flow is estimated to be 2700 cfs.

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 ft, the water surface at WBN will begin to drop 3 hours after failure of the dam and will fall at a fairly uniform rate to elevation 666.0 ft in approximately 27 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 3,200 cfs will be released from Watts Bar Dam. With flow of 3,200 cfs water surface elevation would be 665.9 ft producing 5.9-ft 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 the WBN 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 about 2.2% of the time and have been less than 10,000 cfs about 10.4% of the time.

On March 30 and 31, 1968, during special operations for the control of water milfoil, there were no releases from either Watts Bar or Chickamauga Dams during the two-day period. Over the last 25 years (1986 - 2010) the number of zero flow days at Watts Bar and Chickamauga Dams have been 0 and 2, respectively.

Since January 1940, water levels at the plant have been controlled by Chickamauga Dam. For the period (1940 - 2010), the minimum level at the dam was 673.3 ft 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. However, any control that might be considered would be evaluated before implementation.

2.4.11.5 Plant Requirements

The Engineering Safety Feature System water supply 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 ft and the ERCW pump intake is at elevation 653.33 ft. The ERCW intake will require 5 ft of submergence. Based on a minimum river surface elevation of 665.9 ft, a minimum of 12.57 ft 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 ft, the water surface at the site will begin to drop 3 hours after failure of the dam and will fall at a fairly uniform rate to elevation 666.0 ft in approximately 27 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 3,200 cfs can be released from Watts Bar Dam. This flow will give a river surface elevation of 665.9 ft, which ensures a 5.9-ft depth of water in the intake channel and approximately 10 ft in the river.

A flow of at least 3,200 cfs can be released at the upstream dam, Watts Bar Dam, through the spillway gates, the turbines or the lock. The spillway gates offer the largest flow of water. There are twenty 40-ft-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 ft, there are several gate arrangements that could be used to supply the minimum 3,200 cfs flow.

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 upstream and downstream dams, assurance that sufficient flow will be available is provided by review of the estimated low flows for the period 1903 - 2010 on the basin above Watts Bar Dam which shows that the 15 day, 30 day, 50 day, and 100 day sustained low flow would be 2907 cfs, 3158 cfs, 3473 cfs, and 4012 cfs, respectively. If additional flow is needed to supply the minimum 3,200 cfs it could be supplemented by use of upstream reservoir storage.

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 WBN'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 ft), of liquid is a release which takes place over a short period of time. Calculations have been made to determine the reduction in concentration of such a release as it progresses downstream; particular emphasis has been placed on the concentrations at the surface water intakes downstream of the plant. The model used here is based on the convective diffusion equation as applied to the dispersion in natural streams[24,25]. The major assumptions used in this analysis are:

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

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

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

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

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

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

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

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

and

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

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

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

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

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

2.4.12.2.1 Calculations

The above model was applied to predict the maximum concentrations which would be observed on the right bank of the Tennessee River at two downstream locations; the right bank concentrations will always be higher than those on the left bank. The release is assumed to occur on the right bank at Tennessee River Mile (TRM) 528; the river width is assumed constant at 1,100 ft and the river depth is assumed constant at 30 ft. 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 ft).

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/cubic 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 ft. 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 ft. 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 ft from the center of the plant. In this direction, the hydraulic gradient (dh/dl) is 26 ft (maximum) in 2,600 ft, or 0.01. The hydraulic conductivity (K) of the terrace materials is estimated to be 48 ft/day. (The basis for this estimate is described in Section 2.4.13.3.) Porosity (O) 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 ft, is estimated to be 3.75 cubic ft per square ft of surface area. In a 1000-ft wide strip extending from the plant site to the nearest point of ground water discharge, the volume of stored water would be 9.8×10^6 cubic ft.

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 one-mile to two-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 one-mile to two-mile wide belt, one mile east of the site and across Chickamauga Lake.

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

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

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

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

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

2.4.13.2 Sources

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

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

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

As elsewhere in the region, water levels normally reach maximum elevations in February or March and are at minimum elevations in late summer and early fall. Depth to the water table is generally less than 20 ft 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 ft in 3200 ft, or 0.014.

Water occurs in the Conasauga Shale in very small openings along fractures and bedding planes. Examination of records of 5500 ft of foundation exploration drilling showed only one cavity, 0.6-ft 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-ft-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 ft; the average overburden thickness is 40 ft; the saturated overburden thickness is therefore, 24 ft. No weathered zones or cavities were penetrated in the Conasauga Shale below a depth of 85 ft, so that the average saturated thickness of bedrock is assumed to be less than 50 ft.

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 ft and 33 ft, 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 ft per square ft of surface area. Water available for dilution in bedrock is very small and may be less than 0.01 cubic ft per square ft 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 ft west of the plant site, along a tributary to Yellow Creek. In this direction the maximum hydraulic gradient is 26 ft in 2600 ft, 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]

$$v = \frac{Kdh/dl}{O} \qquad 17$$

where

v = mean velocity, ft/ day;

K = hydraulic conductivity = 48 ft/ day;

dh/dl = hydraulic gradient = .01

O = porosity = 0.15 (estimated 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 ft cavity was penetrated in one hole in a total of more than 5,000 ft 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

The plant grade elevation at WBN can be exceeded by large rainfall and seismically-induced dam failure floods. Assurance that WBN can be safely shut down and maintained in these extreme flood conditions (Section 2.4.2.2 and this Section 2.4.14) is provided by the discussions given in Sections 3.4, 3.8.1, and 3.8.4.

2.4.14.1 Introduction

This subsection describes the methods by which WBN is capable of tolerating floods above plant grade without jeopardizing public safety. Since flooding of this magnitude, as illustrated in Sections 2.4.2 and 2.4.4 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) elevation is based on a still water elevation of 739.2 ft. The calculated still water PMF is elevation 738.9 ft. The table below gives representative levels of the DBF at different plant locations. The equipment within each of the structures required during an external flood event is protected to the values indicated below. The values shown include wave runup and setup.

Design Basis Flood (DBF) Levels

Probable Maximum Flood (still reservoir)	739.2 ft
DBF Runup on 4:1 sloped surfaces	741.6 ft
DBF Runup on critical vertical wall of the Intake Pumping Station	741.7 ft
DBF Surge level within flooded structures	739.7 ft

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 above plant grade. This two-stage scheme is designed to prevent excessive economic loss in case a potential flood does not fully develop. Refer to Section 2.4.14.4.

2.4.14.1.2 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 seismic events. 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 5 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 ft) 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 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 ft. 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 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 the reactor unit, 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 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 the reactor is open to the containment atmosphere during the refueling operations, then the decay heat of the fuel in the reactor and spent fuel pool heat is removed in the following manner. The refueling cavity is filled with borated water (nominal 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 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 the plant.

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 in any stage of refueling.

2.4.14.4.1 Reactor Initially Operating at Power

If the reactor is 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 revocable steps, such as moving flood mode supplies above the PMF 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 less easily revocable 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 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.

2.4.14.5 Equipment

Both normal plant components and specialized flood-oriented supplements are utilized in coping with floods. 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 ERCW 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 ft can be exceeded by rainfall floods and seismic-caused dam failure floods. A warning plan is needed to assure plant safety from these floods.

The warning plan is divided into two stages: Stage I, a minimum of 10 hours long and Stage II, a minimum of 17 hours so that unnecessary economic consequences can be avoided, while adequate time is allowed for preparing for operation in the flood mode. Stage I allows preparation steps causing minimal economic consequences to be sustained but will postpone major economic damage until the Stage II warning forecasts a likely forthcoming flood above elevation 727.0 ft.

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 RO. TVA's climatic monitoring and flood forecasting 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 WBN 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 forecasted plant site water level of elevation 715.5 ft in the winter months and elevation 720.6 ft in the summer. This assures that additional rain will not produce water levels to elevation 727.0 ft in less than 27 hours from the time shutdown is initiated. The water level of elevation 727.0 ft (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 RO determines that floodwaters will not exceed elevation 727.0 ft at the plant. The Stage II warning is issued only when enough additional rain has fallen to forecast that elevation 727.0 ft (winter or summer) is likely to be reached.

2.4.14.8.2 Seismically-Induced Dam Failure Floods

Three postulated combinations of seismically induced dam failures and coincident storm conditions were shown to result in floods which could exceed elevation 727.0 ft at the plant. WBN's notification of these floods utilizes TVA's RO forecast system to identify when a critical combination exists. Stage I shutdown is initiated upon notification that a critical dam failure combination has occurred or loss of communication prevents determining a critical case has not occurred. Stage I shutdown continues until it has been determined positively that critical combinations do not exist. If communications do not document this certainty, shutdown procedures continue into Stage II activity. Stage II shutdown continues to completion or until lack of critical combinations is verified.

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 ft at WBN. 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 ft. Use of elevation 727.0 ft, one 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 ft 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 revocable and cause minimum economic consequences. The estimated probability is small that a Stage I warning will be issued during the 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 ft is reached. The probability of a Stage II warning during the life of the plant is very small.

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

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 which includes WBN.

TVA's RO normal operation produces daily forecasts by 12 noon made from data collected at 6 a.m. Central time. During major flood events, RO may issue forecasts as frequent as 4 to 6 hours at specific site locations.

Elements of the present (2010) forecast system above WBN include the following.

1. More than 90 rain gages measure rainfall, with an average density of about 190 square miles per rain gage. All are Geostationary Operational Environmental Satellites (GOES) Data Collection Platform (DCP) satellite telemetered gages, and 27 are Data Logger telemetered gages.

   Some of the satellite gages transmit hourly rainfall data every 3 hours while others transmit hourly during normal operations.

2. Streamflow data are received from 23 gages in the system. All are GOES Data Collection Platform satellite telemetered gages. The satellite gages transmit 15-minute stage data every three hours during normal operations.

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

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 servers and personal computers are utilized and are designed to provide backup for each other. One computer is used primarily for data collection, with the 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 three-day period.

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

2.4.14.9.3 Basic Analysis

The forecast procedure to assure safe shutdown of WBN for flooding is based upon an analysis of nine hypothetical storms up to PMP magnitude. The storms enveloped potentially critical areal and seasonal variations and time distributions of rainfall. To be certain that fastest rising flood conditions were included, the effects of varied time distribution of rainfall were tested by alternatively placing the maximum daily PMP in the middle, and the last day of the three-day main storm. Earlier analysis of 17 hypothetical storms demonstrated that the shortest warning times resulted from storms in which the heavy rainfall occurred on the last day and that warning times were significantly longer when heavy rainfall occurred on the first day. Therefore, heavy rainfall on the first day was not reevaluated. The warning system is based on those storm situations which resulted in the shortest time interval between watershed rainfall and elevation 727.0 ft at WBN, 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.

2.4.14.9.4 Hydrologic Basis for Warning System

A minimum of 27 hours has been allowed for preparation of the plant for operation in the flood mode, three hours more than the 24 hours needed. An additional 4 hours for communication and forecasting computations is provided to allow TVA's RO 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 727.0 ft 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 727.0 ft, one 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 "winter" season, Stage I shutdown procedures will be started as soon as target river elevation 715.5 ft has been forecast. Stage II shutdown will be initiated and carried to completion if and when target river elevation 727.0 ft at WBN has been forecast. Corresponding target river elevations for the "summer" season at WBN are elevation 720.6 ft and elevation 727.0 ft.

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 may 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, as determined by TVA's RO, have returned to a condition in which 27 hours of warning will again be available.

2.4.14.9.5 Hydrologic Basis for Warning Times and Elevations

Figure 2.4-110 (Sheet 1) and Figure 2.4-110 (Sheet 2) for winter and summer respectively, show target forecast flood warning time and elevation at WBN which assure adequate warning times.

The fastest rising PMF for the winter at the site is shown in Figure 2.4-110 (Sheet 1A). Figure 2.4-110 (Sheets 1B and 1C) show the adopted rainfall distribution for the 21,400 square mile storm and the 7,980 square mile storm, respectively. An intermediate flood with average basin rainfall of 10 inches (rainfall heavy at the end) is shown in Figure 2.4-110 (Sheet 1D). Figure 2.4-110 (Sheet 2A) shows the 7,980 square mile fastest rising PMF for the summer with heavy rainfall at the end. The 7,980 square mile adopted rainfall distribution is shown in Figure 2.4-110 (Sheet 2B). An intermediate flood with average basin rainfall of 10 inches heavy at the end is shown in Figure 2.4-110 (Sheet 2C). All of these storms have been preceded three days earlier by a three-day storm having 40% of PMP storm rainfall.

The fastest rising flood occurs during a PMP when the six-hour increments increase throughout the storm with the maximum 6 hours occurring in the last period. Figure 2.4-110 (Sheet 1A) shows the essential elements of this storm which provides the basis for the warning plan. In this flood 8.6 inches of rain would have fallen 31 hours (27 + 4) prior to the flood crossing elevation 727.0 ft and would produce elevation 715.5 ft at the plant. Hence, any time rain on the ground results in a forecast plant elevation of 715.5 ft a Stage I shutdown warning will be issued. Examination of Figure 2.4-110 (Sheets 1B and 1C) show that following this procedure in these floods would result in longer times to reach elevation 727.0 ft after Stage I warning was issued. These times would be 41 and 46.5 hours (includes 4 hours for forecasting and communication) for Figure 2.4-110 (Sheet 1B) and (Sheet 1C), respectively. This compares to the 31 hours for the fastest rising flood Figure 2.4-110 (Sheet 1A). Stage I warning would be issued for the storm shown in Figure 2.4-110 (Sheet 1D) but would not reach a Stage II warning as the maximum elevation reached is 721.92 ft which is well below elevation 727.0 ft.

An additional 2.6 inches of rain must fall promptly for a total of 11.2 inches of rain to cause the flood to exceed elevation 727.0 ft. In the fastest rising flood, Figure 2.4-110 (Sheet 1A), this rain would have fallen in the next 6.0 hours. A Stage II warning would be issued within the next 4 hours. Thus, the Stage II warning would be issued 6.0 hours after issuance of a Stage I warning and 21.0 hours before the flood would exceed elevation 727.0 ft. In the slower rising floods, Figure 2.4-110 (Sheets 1B and 1C), the time between issuance of a Stage I warning and when the 11.2 inches of rain required to put the flood to elevation 727.0 ft would have occurred, is 7.0 and 5.0 hours respectively. This would result in issuance of a Stage II warning not more than 4 hours later or 30.0 or 37.5 hours, respectively, before the flood would reach elevation 727.0 ft.

The summer flood, shown by Figure 2.4-110 (Sheet 2A), with the maximum one-day rain on the last day provides controlling conditions when reservoirs are at summer levels. At a time 31 hours (27 + 4) before the flood reaches elevation 727.0 ft, 9.3 inches of rain would have fallen. This 9.3 inches of rain under these runoff conditions would produce elevation 720.6 ft, so this level becomes the Stage I target. An additional 2.0 inches of rain must fall promptly for a total of 11.3 inches of rain to cause the flood to exceed elevation 727.0 ft. In this fastest rising summer flood, Figure 2.4-110 (Sheet 2A), this rain would have fallen in the next 4.5 hours. A Stage II warning would be issued within the next 4 hours. Thus, the Stage II warning would be issued 4.5 hours after issuance of a Stage I warning and 22.5 hours before the flood would exceed elevation 727.0 ft.

The above criteria all relate to forecasts which use rain on the ground. In actual practice quantitative rain forecasts, which are already a part of daily operations, would be used to provide advance alerts that 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 forecast elevations which will assure adequate shutdown times are:

	Forecasts Elevations at Watts Bar	
Season	Stage I shutdown	Stage II shutdown
Winter	715.5 ft	727.0 ft
Summer	720.6 ft	727.0 ft

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 System, 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. Additionally, RO have available satellite telephone communications with the TVA hydro projects upstream of Chattanooga (listed in Section 2.4.14.9.2).

RO 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 RO office.

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

Plant grade would be exceeded by three of the five candidate seismic failure combinations evaluated, thus requiring emergency measures. The seismic dam failure combination, the SSE failure of Norris, Cherokee, Douglas and Tellico Dams concurrent with the 25-year flood would result in a maximum flood elevation of 731.17 ft at WBN. The OBE failure of Norris and Tellico Dams and the OBE failure of Cherokee, Douglas, and Tellico Dams concurrent with the one-half PMF would result in flood elevations above WBN plant grade. Table 2.4-14, shows the maximum elevations at WBN for the candidate combinations.

The times from seismic failure to the time elevation 727.0 ft is reached at WBN in the three critical events is about 35, 27, and 44 hours as shown in Figures 2.4-114, 2.4-115, and 2.4-116 for the Norris, Cherokee, Douglas and Tellico Dams SSE failure combination, the Norris and Tellico Dams OBE failure combination and the Cherokee, Douglas, and Tellico Dams OBE failure combinations, respectively. These times are adequate to permit safe plant shutdown in readiness for flooding.

Dam failure during non-flood periods was not evaluated, but would be bounded by the three critical failure combinations.

The warning scheme for safe plant shutdown is based on the fact that a combination of critically centered large earthquake conditions must coincide before the flood wave from seismically caused dam failures will approach plant grade. In flood situations, an extreme earthquake must be precisely located to fail Norris, Cherokee, Douglas, and Tellico Dams before a flood threat to the site would exist. This would also be the case with the failure of Norris and Tellico. Cherokee and Douglas Dams failures that could occur when the OBE is located midway between the dams which are just 15 miles apart.

The warning system utilizes TVA's RO flood forecast system to identify when flood conditions will be such that seismic failure of critical dams could cause a flood wave to approach elevation 728.0 ft at the plant site. In addition to the critical combinations, failure of a single major upstream dam will lead to an early warning. A Stage I warning is declared once failure of (1) Norris, Cherokee, Douglas, and Tellico Dams or (2) Norris and Tellico Dams or (3) Cherokee, Douglas and Tellico Dams has been confirmed.

If loss of or damage to an upstream dam is suspected based on monitoring by TVA's RO, efforts will be made by TVA to determine whether dam failure has occurred. If the critical case has occurred or it cannot be determined that it has not occurred, Stage I shutdown will be initiated. Once initiated, the flood preparation procedures will be carried to completion unless it is determined that the critical case has not occurred.

Communications between WBN, dams, power system control center, and TVA RO are accomplished by TVA-owned microwave networks, fiber-optics network, radio networks, and commercial and satellite telephone service. These systems are described in UFSAR 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 RO to determine if the desired warning time is available. If weather and reservoir conditions are such that the desired time can be provided, special warning procedures will be developed, if necessary, to ensure the time is available. This special case continues until the Plant Manager notifies TVA's RO that maintenance has been completed. If threatening storm conditions are forecast which might shorten the available time for special maintenance, the Plant Manager is notified by RO and steps taken to assure that the plant is placed in a safe shutdown mode.

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45. "SOCH Geometry Verification - Tellico Reservoir and Tellico/Ft. Loudoun Canal" CDQ000020080025 Revision 2

46. "SOCH Geometry Verification - Watts Bar Reservoir" CDQ000020080026 Revision 2

47. "SOCH Geometry Verification – Melton Hill Reservoir" CDQ000020080029 Revision 2

48. "SOCH Geometry Verification - Chickamauga Reservoir" CDQ000020080030 Revision 2

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50. "SOCH Geometry Verification - Guntersville Reservoir" CDQ000020080032 Revision 3

51. "SOCH Geometry Verification - Wheeler Reservoir" CDQ000020080033 Revision 0

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ENCLOSURE 3

FSAR Significant Change Roadmap

ENCLOSURE 3

FSAR Significant Change Roadmap

Contents by FSAR Section (Number of Changes)

2.4 - Hydrologic Engineering (3)

Three significant changes were made to this section:

1. The calculated probable maximum flood elevation was changed to 738.9 ft. This calculated PMF, combined with 0.3 ft additional margin, provides a design basis probable maximum flood elevation of 739.2 ft.
2. The discussion associated with wind waves and wave run up was deleted as it is covered in other locations
3. A reference to UFSAR Section 2.4.14 was added to direct the reader to a location that discusses plant protection during external flood events.

2.4.1 - Hydrological Description (0)

2.4.1.1 - Sites and Facilities (0)

No significant changes were made to this section.

2.4.1.2 – Hydrosphere (1)

One significant change was made to this section to reflect the correct number of non-TVA dams and current owners.

2.4.2 – Floods (0)

2.4.2.1 - Flood History (0)

No significant changes made

2.4.2.2 - Flood Design Considerations (3)

Two significant changes were made to this section:

1. Changed location of storm that produced maximum PMF from PMP critically centered on the watershed to the 7,980 square-mile Bulls Gap storm.
2. Increased the design basis flood level of the Intake Pumping Station from 741.0 ft. to 741.7 ft.

2.4.2.3 - Effects of Local Intense Precipitation (1)

Changed reference for local intense precipitation from PMF to PMP.

2.4.3 - Probable Maximum Flood (PMF) on Streams and Rivers (8)

Six significant changes were made to this section:

1. The storm designation that caused the maximum flood level at the plant was changed from the 'March PMP 21,400 square-mile storm' to the 'March PMP 7,980 square-mile Bull's Gap centered storm with hydrologic failure of low margin dams'.

2. Included a list of the 18 critical dams TVA evaluated the stability of at PMF headwater/tailwater conditions. This list includes:

   1) Apalachia;
   2) Blue Ridge;
   3) Boone;
   4) Chatuge;
   5) Cherokee;
   6) Chickamauga;
   7) Douglas;
   8) Fontana;
   9) Fort Loudoun;
   10) Fort Patrick Henry;
   11) Hiwassee;
   12) Melton Hill;
   13) Norris;
   14) Nottely;
   15) South Holston;
   16) Tellico;
   17) Watauga; and
   18) Watts Bar

3. Included a list of the nine dams in the tributary system that were not evaluated for stability and are postulated to fail during the PMF. This list includes:

   1) Ocoee 1;
   2) Ocoee 2;
   3) Ocoee 3;
   4) Chilhowee;
   5) Calderwood;
   6) Cheoah;
   7) Mission;
   8) John Sevier; and
   9) Wilbur

4. Included a list of four dams that were evaluated for stability but because of low margin were postulated to fail during the PMF. This lists includes:

   1) Apalachia (for the 21,400 square mile storm);

2) Boone;
3) Fort Patrick Henry; and
4) Melton Hill (for the 7,980 square mile storm)

5. Added the crest elevation of the West Saddle Dike at the Watts Bar Dam and changed the failure from 'breached' to 'postulated to fail'.

6. Changed the discussion of maximum discharge at the plant from the constituent parts to a total flow of 1,158,956 cfs.

2.4.3.1 - Probable Maximum Precipitation (PMP) (4)

Four significant changes were made to this section.

1. Deleted the discussion associated with only two seasons being evaluated.

2. Changed the storm that produced the PMF from the '21,400 square mile storm' to the '7,980 square-mile Bulls Gap centered storm'.

3. Changed the PMP rainfall produced in three days from '16.25 inches' to '16.17 inches'.

4. Changed the antecedent storm rainfall produced from '6.18 inches' to '6.00 inches'.

2.4.3.2 - Precipitation Losses (1)

One significant change was made to this section.

1. Changed the average precipitation loss for the watershed above the Chickamauga Dam from:

   1) 2.33 inches to 2.32 inches for the three-day antecedent storm
   2) 1.86 inches to 1.87 inches for the three-day main storm

2.4.3.3 - Runoff and Stream Course Model (7)

Eight significant changes were made to this section.

1. Changed the description of the model used from the Simulated Open Channel Hydraulics (SOCH) model to the USACE Hydrologic Engineering Center River Analysis System (HEC-RAS) model.

2. Changed the listing of the reservoirs where temporary flood barriers are installed from Cherokee, Watts Bar, Fort Loudoun, and Tellico to only Fort Loudoun.

3. Added Section 2.4.3.3.1, PMF Determination, which describes how the HEC-RAS model was used to determine the PMF elevations and discharges at WBN.

4. Added Section 2.4.3.3.2, Model Setup, which includes a discussion on, the extent of the model, development of a composite model, gate/outlet functionality, unsteady flow rule description, run iterations, and the use of inflow hydrographs.

5. Added Section 2.4.3.3.3, Main Stem Geometry, which describes how the geometry of the main stem portions including the Fort Loudoun, Tellico, Melton Hill, Watts Bar, Chickamauga, Nickajack, Guntersville, Wheeler, and Wilson reservoirs were validated.

6. Added Section 2.4.3.3.4, Tributary Geometry, which describes how the geometry of the tributary portions including the Apalachia Reservoir, Ocoee River, Toccoa River, Blue Ridge Reservoir, Boone Reservoir, Watauga River, Wilbur Reservoir, South Fork Holston River, Holston River, French Broad River, Nolichucky River, Little Tennessee River, Fort Patrick Henry Reservoir, Hiwassee River and Reservoir, Nottely River, and the Elk River, Fontana Reservoir, Tuckasegee River, Norris Reservoir, Powell River, Big Creek, and Cove Creek were created and/or validated.

7. Added Section 2.4.3.3.5, Calibration, which includes Section 2.4.3.3.5.1, Main Stem River, and Section 2.4.3.3.5.2, Tributary Calibration. These Sections describe how the model calibration was performed to adjust model parameters so that the model would accurately predict the outcome of a known historic event(s).

8. Moved discussions regarding SOCH modeling to new Appendix 2.4A.

2.4.3.4 - Probable Maximum Flood Flow (4)

Five significant changes were made to this section.

1. Increased the PMF discharge at WBN from 1,088,625 cfs to 1,158,956 cfs

2. Changed the storm that produced the PMF from the '21,400 square mile storm' to the '7,980 square-mile Bulls Gap centered storm'.

3. The concrete section analysis was changed to identify the procedure TVA uses to evaluate dams including the acceptance criteria. Using the procedure the following dams were determined to have low margin and are postulated to fail; Fort Patrick Henry (total failure), Boone (total failure), Melton Hill (total failure in the 7,980 Bulls Gap centered storm), Apalachia (total failure in the 21,400 downstream centered March storm).

4. The concrete section analysis was changed to include a listing of the dams/dikes where modifications were credited including Cherokee Dam post-tensioning, Watts Bar Dam non-overflow neck, Douglas Dam post-tensioning, Fort Loudoun Dam non-overflow, and Tellico Dam non-overflow neck.

5. A new subsection is added to provide a discussion on embankment dam sections global stability analysis, including the acceptance criteria used and the modifications credited in support of WBN Unit 2 licensing.

2.4.3.5 - Water Level Determinations (2)

Two significant changes were made to this section.

1. The calculated probable maximum flood elevation was changed to 738.9 ft. This calculated PMF, combined with 0.3 ft additional margin, provides a design basis probable maximum flood elevation of 739.2 ft.

2. Changed the storm that produced the PMF from the '21,400 square mile storm' to the '7,980 square-mile storm'

2.4.3.6 - Coincident Wind Wave Activity (3)

No significant changes were made to this section.

2.4.4 - Potential Dam Failures, Seismically Induced (0)

2.4.4.1 - Dam Failure Permutations (3)

Three significant changes were made to this section:

1. Changed the listing of the reservoirs where temporary flood barriers are installed from Cherokee, Watts Bar, Fort Loudoun, and Tellico to only Fort Loudoun.

2. Restored historical information regarding a safety factor of 1.52.

3. Identified that Brookfield Renewable Energy Partner purchased four ALCOA dams

2.4.4.2 - Unsteady Flow Analysis of Potential Dam Failures (0)

No significant changes were made to this section.

2.4.4.3 - Water Level at Plant Site (0)

No significant changes were made to this section.

2.4.5 - Probable Maximum Surge and Seiche Flooding (0)

No significant changes were made to this section.

2.4.6 - Probable Maximum Tsunami Flooding (0)

No significant changes were made to this section.

2.4.7 - Ice Effects (0)

No significant changes were made to this section.

2.4.8 - Cooling Water Canals and Reservoirs (0)

No significant changes were made to this section.

2.4.9 - Channel Diversions (0)

No significant changes were made to this section.

2.4.10 - Flooding Protection Requirements (0)

No significant changes were made to this section.

2.4.11 - Low Water Considerations (0)

No significant changes were made to this section.

2.4.11.1 - Low Flow in Rivers and Streams (0)

No significant changes were made to this section.

2.4.11.2 - Low Water Resulting From Surges, Seiches, or Tsunami (0)

No significant changes were made to this section.

2.4.11.3 - Historical Low Water (0)

No significant changes were made to this section.

2.4.11.4 - Future Control (0)

No significant changes were made to this section.

2.4.11.5 - Plant Requirements (0)

No significant changes were made to this section.

2.4.12 - Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents (0)

No significant changes were made to this section.

2.4.12.1 - Radioactive Liquid Wastes (0)

No significant changes were made to this section.

2.4.12.2 - Accidental Slug Releases to Surface Water (0)

No significant changes were made to this section.

2.4.12.3 - Effects on Ground Water(0)

No significant changes were made to this section.

2.4.13 - Groundwater (0)

No significant changes were made to this section.

2.4.13.1 - Description and On-Site Use (0)

No significant changes were made to this section.

2.4.13.2 - Sources (0)

No significant changes were made to this section.

2.4.13.3 - Accident Effects (0)

No significant changes were made to this section.

2.4.13.4 - Monitoring and Safeguard Requirements (0)

No significant changes were made to this section.

2.4.13.5 - Design Basis for Subsurface Hydrostatic Loading (0)

No significant changes were made to this section.

2.4.14 - Flooding Protection Requirements (0)

No significant changes were made to this section.

2.4.14.1 - Introduction (1)

One significant change was made to this section. Added a statement associated with the design basis flood (DBF) elevation stating it is based on a still water elevation of 739.2 ft and that the still water PMF elevation is 738.9 ft.

2.4.14.2 - Plant Operation During Floods Above Grade (0)

No significant changes were made to this section.

2.4.14.3 - Warning Scheme (0)

No significant changes were made to this section.

2.4.14.4 - Preparation for Flood Model (0)

No significant changes were made to this section.

2.4.14.5 - Equipment (0)

No significant changes were made to this section.

2.4.14.6 - Supplies (0)

No significant changes were made to this section.

2.4.14.7 - Plant Recovery (0)

No significant changes were made to this section.

2.4.14.8 - Warning Plan (0)

No significant changes were made to this section.

2.4.14.9 - Basis For Flood Protection Plan In Rainfall Floods (0)

No significant changes were made to this section.

2.4.14.10 - Basis for Flood Protection Plan in Seismic-Caused Dam Failures (0)

No significant changes were made to this section.

2.4.14.11 - Special Condition Allowance (0)

No significant changes were made to this section.

REFERENCES (17)

Added 17 new references.

ENCLOSURE 3

UFSAR Significant Change Roadmap
July 19, 2012 Submittal to Current

Major Change Road Map		
Major Change	UFSAR Sections	
PMF Level Reduction	2.4	Hydrologic Engineering
	2.4.2.2	Flood Design Considerations
	2.4.3	Probable Maximum Flood (PMF) on Streams and Rivers
	2.4.3.5	Water Level Determinations
Dam Ownership	2.4.1.2	Hydrosphere
PMF Storm	2.4.2.2	Flood Design Considerations
	2.4.3.1	Probable Maximum Precipitation
	2.4.3.4	Probable Maximum Flood Flow
	2.4.3.5	Water Level Determinations
Run-up/Set-up Elevations	2.4.2.2	Flood Design Considerations
	2.4.3.6	Coincident Wind Wave Activity
Critical Dams	2.4.3	Probable Maximum Flood (PMF) on Streams and Rivers
Dams Postulated to Fail	2.4.3	Probable Maximum Flood (PMF) on Streams and Rivers
	2.4.3.4	Probable Maximum Flood Flow
	2.4.3.5	Water Level Determinations
Low Margin Dams	2.4.3	Probable Maximum Flood (PMF) on Streams and Rivers

ENCLOSURE 3

UFSAR Significant Change Roadmap
July 19, 2012 Submittal to Current

Major Change Road Map		
Major Change	UFSAR Sections	
Discharge Volumes	2.4.3	Probable Maximum Flood (PMF) on Streams and Rivers
	2.4.3.4	Probable Maximum Flood Flow
Rain Fall	2.4.3.1	Probable Maximum Precipitation
	2.4.3.2	Precipitation Losses
SOCH – HEC-RAS Model Change	2.4.3.3	Runoff and Stream Course Model
Temporary Flood Barriers	2.4.4.1	Dam Failure Permutations
Safety Factors	2.4.3.4	Probable Maximum Flood Flow
	2.4.14.1	Introduction

ENCLOSURE 4
Tennessee Valley Authority
Watts Bar Nuclear Plant, Unit 2
Docket No. 50-391
List of Commitments

1. Enclosure 2 provides a clean version of Unit 2 FSAR Section 2.4 which will be included in Amendment 113 scheduled for October 2014.

2. TVA will take actions to ensure the stability of the following dams under probable maximum flood conditions, as credited in the hydraulic analysis described in TVA Letter dated September 30, 2014 to NRC (CNL-14-184), Enclosure 7:

    - Tellico Dam
    - Watts Bar Dam
    - Watts Bar West Saddle Dike
    - Fort Loudoun Dam
    - Cherokee Dam
    - Douglas Dam
    - Douglas Saddle Dams

    The actions, which may include the modifications described in TVA Letter dated September 30, 2014 (CNL-14-184) to NRC, or other appropriate actions to ensure dam stability consistent with TVA's River Operations acceptance criteria, will be completed prior to loading fuel into the WBN Unit 2 reactor vessel.

3. The Tennessee Valley Authority will complete modifications to install passive flood barriers to provide protection for equipment within the Watts Bar Nuclear Plant Unit 2 Intake Pumping Structure needed during an external flood to the design basis flood elevation, by April 30, 2015.