![TVA logo]

Tennessee Valley Authority, 1101 Market Street, Chattanooga, TN 37402

CNL-17-070

June 5, 2017

10 CFR 52, Subpart A

ATTN: Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Clinch River Nuclear Site
NRC Docket No. 52-047

Subject: Submittal of Supplemental Information Associated with Hydrologic Engineering in Support of the Clinch River Nuclear Site Early Site Permit Application

References: 1. Letter from TVA to NRC, CNL-16-081, "Application for Early Site Permit for Clinch River Nuclear Site," dated May 12, 2016

2. NRC Memorandum, "April 17 - 28, 2017, Audit of Clinch River Nuclear Site Early Permit Application - Hydrology and Health Physics Analyses," dated April 11, 2017

By letter dated May 12, 2016 (Reference 1), Tennessee Valley Authority (TVA) submitted an application for an early site permit for the Clinch River Nuclear (CRN) Site in Oak Ridge, TN. Between April 24, 2017 and April 27, 2017, the NRC conducted an audit of the hydrologic engineering information contained in the CRN Site Early Site Permit Application (ESPA), Part 2, "Site Safety Analysis Report (SSAR)," Section 2.4, "Hydrologic Engineering" (Reference 2). During the face-to-face portion of the NRC audit held at the TVA offices in Knoxville, TN, the NRC requested that, by June 5, 2017, TVA provide supplemental information to SSAR Section 2.4 as presented during the NRC audit.

The enclosure to this letter provides proposed SSAR markups as discussed during the NRC audit for information needs 1 through 10, 15 through 20, 22, and 32. The SSAR markups will be incorporated in a future revision of the early site permit application.

There are no new regulatory commitments associated with this submittal. If any additional information is needed, please contact Dan Stout at (423) 751-7642.

I declare under penalty of perjury that the foregoing is true and correct. Executed on this 5th day of June 2017.

Respectfully,

J. W. Shea

Digitally signed by J. W. Shea
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J. W. Shea
Vice President, Nuclear Licensing

Enclosure:

> Supplemental Information Regarding Site Safety Analysis Report Section 2.4, "Hydrologic Engineering"

cc (w/ Enclosure):

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cc (w/o Enclosure):

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ENCLOSURE

**Supplemental Information Regarding
Site Safety Analysis Report Section 2.4, "Hydrologic Engineering"**

By letter dated May 12, 2016 (Reference 1), Tennessee Valley Authority (TVA) submitted an application for an early site permit for the Clinch River Nuclear (CRN) Site in Oak Ridge, TN. On April 24, 2017 through April 27, 2017, the NRC conducted an audit of the hydrologic information contained in the CRN Site Early Site Permit Application (ESPA) (Reference 2). During the face-to-face portion of the NRC audit held at the TVA offices in Knoxville, TN, the NRC requested that TVA provide supplemental information associated with SSAR Section 2.4, "Hydrologic Engineering," to reflect the information that TVA provided during the NRC audit.

This enclosure provides the supplemental information as an update of portions of Site Safety Analysis Report (SSAR) Section 2.4 discussed during the audit. Specifically, this enclosure provides supplemental information associated with audit information needs 1 through 10, 15 through 20, 22, and 32. The SSAR markups included in Attachments 1 through 5 of this enclosure will be incorporated in a future revision of the ESPA.

References:

1. Letter from TVA to NRC, CNL-16-081, "Application for Early Site Permit for Clinch River Nuclear Site," dated May 12, 2016
2. NRC Memorandum from Mallecia Sutton to Allen Fetter, "Audit of Clinch River Nuclear Site Early Permit Application - Hydrology and Health Physics Analysis," dated April 11, 2017

Attachments:

1. Site Safety Analysis Report Subsection 2.4.1 Markups
2. Site Safety Analysis Report Subsection 2.4.2 Markups
3. Site Safety Analysis Report Subsection 2.4.3 Markups
4. Site Safety Analysis Report Subsection 2.4.4 Markups
5. Site Safety Analysis Report Subsection 2.4.13 Markups

Supplemental Information Associated with NRC Audit Information Needs:

Following the face-to-face portion of the NRC audit, TVA is providing the following supplemental information associated with the referenced audit Information Need:

Supplemental Information associated with NRC Information Need 1 and 32

> To clarify the naming of the "Oak Ridge Water Treatment Plant," SSAR Subsections 2.4.1.2.1, "Surface Water," and 2.4.13.2, "Receptors," are being revised. See the SSAR markup provided in Subsection 2.4.1.2.1 and 2.4.13.2 in Attachments 1 and 5, respectively.

Supplemental Information associated with NRC Information Need 2

Footnotes 1 and 2 of SSAR Table 2.4.2-1 are being revised to clarify how the flood elevations were developed. See the SSAR Table 2.4.2-1 markup as shown in Attachment 2.

Supplemental Information associated with NRC Information Need 3

To clarify the discussion of Flooding from Dams Breaches and Failures, the following SSAR Subsections are being revised:

a) in SSAR Subsection 2.4.2.2, subheading "Flooding from Rivers and Streams," the following sentence is being inserted after the third sentence: "Dam failures associated with this event are discussed in Subsection 2.4.3.5."
b) in SSAR Subsection 2.4.2.2, subheading "Flooding from Combined Effects," the following sentence is being inserted at the end of the first paragraph: "Dam failures associated with the combined effects of the PMF and wind are equivalent to the dam failures for PMF alone. Dam failures associated with the PMF alone are discussed in Subsection 2.4.3.5."
c) in SSAR Subsection 2.4.2.2, subheading "Flooding from Combined Effects," the current reference to SSAR Subsection 2.4.4 in the fifth paragraph is being revised to Subsection 2.4.4.2.1.

See the markup provided in SSAR Subsection 2.4.2.2 in Attachment 2.

Supplemental Information associated with NRC Information Need 4

To clarify the text discussion regarding a site drainage plan, Figure 2.4.1-4, "Representative Site Grading Plan," is being removed from the SSAR as indicated in Attachment 1.

Additional clarification discussing the basis for the Local Intense Precipitation (LIP) Probable Maximum Precipitation (PMP) temporal distribution is being added to SSAR Subsection 2.4.2.3.1, "Precipitation Distribution." See the markup provided in SSAR Subsection 2.4.2.3.1 in Attachment 2.

Supplemental Information associated with NRC Information Need 5

A discussion of the methods and calculations of the PMP development for the four types of events is being added to SSAR Subsection 2.4.3.2, "Probable Maximum Precipitation." Table 2.4.3-1, "Flood Events Analyzed," is being revised to add precipitation depths and velocities and to add clarifying footnotes. See the markup provided in SSAR Subsection 2.4.3.2 and SSAR Table 2.4.3-1 in Attachment 3.

Supplemental Information associated with NRC Information Need 6

The discussion in SSAR Subsection 2.4.3.3, "Precipitation Losses," second paragraph, in regard to adjusting the unit hydrographs to reflect nonlinearity of the runoff generation process under field conditions, as recommended by NUREG/CR-7046, is being relocated and inserted prior to the "National Inventory of Dams (NIS) Considerations" subheading of SSAR Section 2.4.3.4.1, "Runoff Model." See the markup provided in SSAR Subsections 2.4.3.3 and 2.4.3.4.1 in Attachment 3.

Supplemental Information associated with NRC Information Need 7

A discussion of how the Unit Hydrographs (UHs) were developed, including validation of the UH's is being added, under a subheading titled "Unit Hydrograph Development and Validation," to SSAR Subsection 2.4.3.4.1, "Runoff Model." Validated unit hydrograph figures and reference to those figures are also being added with this revision. In addition, a discussion of how the inflows were determined for the 1973/2003 storms used in the calibration is being added to SSAR Subsection 2.4.3.4.3, "Stream Course Model Geometry Development and Calibration." For these changes, see the markup provided in SSAR Subsection 2.4.3.4.1, subheading "Unit Hydrograph Development and Validation," addition of new Figures 2.4.3-17, Sheets 1 through 8, and Figures 2.4.3-18, 2.4.3-19, and 2.4.3-20, and SSAR Subsection 2.4.3.4.3, the eighth paragraph, in Attachment 3.

A clarification of how the storage volume from NID dams was used to develop inflow hydrographs is being added to SSAR Subsection 2.4.3.4.1, "Runoff Model," under the new subheading "National Inventory of Dams (NID) Considerations." See the markup provided in SSAR Subsection 2.4.3.4.1, under the new subheading "National Inventory of Dams (NIS) Considerations," in Attachment 3.

Supplemental Information associated with NRC Information Need 8

A discussion of the Highway 58 bridge sensitivity analysis and the results/conclusions is being added to SSAR Subsection 2.4.3.4.3, "Stream Course Model Geometry Development and Calibration." In addition, a figure depicting the Highway 58 bridge profile is being added to the SSAR and reference to the Figure is being included in the SSAR Subsection 2.4.3.4.3 discussion. See the markup provided in SSAR Subsection 2.4.3.4.3, fourth paragraph, and addition of new Figure 2.4.3-21, "Highway 58 Bridge Profile," in Attachment 3.

SSAR Subsection 2.4.3.4.3, "Stream Course Model Geometry Development and Calibration," and Subsection 2.4.3.4.4, "Design Storm Implementation," are being revised to add sufficient detail on the development of the HEC-RAS model, including a discussion of ineffective flow, impact of new fill on storage capacity, and new fill above PMF and away from the conveyance channel. In addition, figures depicting the reservoir storage curves and reference to the reservoir storage curves are being included in Subsection 2.4.3.4.3. See the markup provided in SSAR Subsections 2.4.3.4.3, first, second, fourth, and fifth paragraphs and 2.4.3.4.4, first and fifth paragraph, and new Figures 2.4.3-22, "Norris Reservoir Volume versus Elevation," 2.4.3-23, "Melton Hill Reservoir Volume versus Elevation," and 2.4.3-24, "Watts Bar Reservoir Volume versus Elevation," in Attachment 3.

Supplemental Information associated with NRC Information Need 9

A discussion describing the methods and results of the reservoir volume verification and figures showing the methods and results are being added to SSAR Subsection 2.4.3.4.3, "Stream Course Model Geometry Development and Calibration." See the markup provided in SSAR Subsection 2.4.3.4.3, first paragraph, third and fourth sentences and reference to Figures 2.4.4-1, Sheets 1 - 12, in first sentence in Attachment 3.

Reference to Figure 2.4.4-1, Sheets 1 - 12, is being added to SSAR Subsection 2.4.3.4.3, "Stream Course Model Geometry Development and Calibration." See the markup provided in SSAR Subsection 2.4.3.4.3, ninth paragraph, in Attachment 3.

Reference to new Figures 2.4.3-10 through 2.4.3-13, is being added to SSAR Subsection 2.4.3.4.3 to provide calibration points and calibration results for the Watts Bar and Melton Hill Reservoirs. Reference to new Figures 2.4.3-14 and 2.4.3-15 is being added to the SSAR to the calibration point and the calibration result for the Clinch River above Norris Dam. See the markup provided in SSAR Subsection 2.4.3.4.3, ninth, tenth, and eleventh paragraphs, and addition of new Figures 2.4.3-10 through 2.4.3-15 in Attachment 3.

Supplemental Information associated with NRC Information Need 10

A discussion to provide the operational curve of the Norris Dam as well as the various cases presented in the rating curve figures and their application to the HEC-RAS simulations is being added to SSAR Subsection 2.4.3.4.4, "Design Storm Implementation." In addition, SSAR Figure 2.4.3-4, Sheets 1 and 2 are being revised and Figure 2.4.3-7, Sheet 2 of 2 is being removed. See the markup provided in SSAR Subsection 2.4.3.4.4, fourth and fifth paragraphs, and the last four sentences of this Subsection, revised Figure 2.4.3-4, Sheets 1 and 2, and Figure 2.4.3-7 is shown removed, in Attachment 3.

Supplemental Information associated with NRC Information Need 15

The references used in the hydrologic dam failure analysis are being added to SSAR Subsection 2.4.3.5, "Probable Maximum Flood Flow," and to the list of references in SSAR Subsection 2.4.3.8, "References." These changes are provided in the markup in SSAR Subsections 2.4.3.5 and 2.4.3.8 in Attachment 3.

Supplemental Information associated with NRC Information Need 16

See the supplemental information associated with NRC Information Need 8, third and sixth paragraphs in SSAR Subsection 2.4.3.4.3 in Attachment 3.

Supplemental Information associated with NRC Information Need 17

A figure illustrating the PMF elevation in the Clinch River and estimation of the fetch length used in the wind wave calculations is being added to SSAR Subsection 2.4.3.7, "Coincident Wind Wave Activity." See the SSAR Subsection 2.4.3.7 markup and the addition of Figure 2.4.3-16, "CRN Site Critical Fetch Length," in Attachment 3.

Supplemental Information associated with NRC Information Need 18

A GIS map showing the location of all dams upstream of Watts Bar Dam in relation to the layout of the Tennessee River and its tributaries is being added to the SSAR. Existing Figure 2.4.1-5 is being removed and replaced with new Figure 2.4.1-5, "Tennessee River System," Sheets 1 and 2 of 2. A copy of the new Figure 2.4.1-5, Sheets 1 and 2 of 2 is provided in Attachment 1.

Supplemental Information associated with NRC Information Need 19

The seismic NID inflow hydrographs, Figure 2.4.4-5, "Seismic Inflow Hydrographs for 500-Yr June Flood Event - Norris Dam," Figure 2.4.4-6, "Seismic Inflow Hydrographs for 500-Yr June Flood Event - Melton Hill Dam," and Figure 2.4.4-7, "Seismic Inflow Hydrographs for 500-Yr June Flood Event - Watts Bar Dam," are being added to SSAR 2.4.4. Reference to the Figures is being added to SSAR Subsection 2.4.4.2.1, "Seismic Failure Analysis," at the end of the sixth paragraph under the subheading "Flood Routing." See the markup in SSAR Subsections 2.4.4 and the new Figures 2.4.4-5, 2.4.4-6, and 2.4.4-7, in Attachment 4.

Supplemental Information associated with NRC Information Need 20

A discussion of the methods and inflows for each of the assumed flood events is being added to SSAR Subsection 2.4.4.2.1, "Seismic Failure Analysis," Subheading "Flood Routing." New references are being added to SSAR Subsection 2.4.4.7, "References." See the SSAR Subsection 2.4.4.2.1 and 2.4.4.7 markups provided in Attachment 4.

Supplemental Information associated with NRC Information Need 22

A figure illustrating the PMF elevation in the Clinch River and the estimation of the fetch length used in the wind wave calculations is being added to the SSAR. Figure 2.4.3-16, "CRN Site Critical Fetch Length," is being added to SSAR Subsection 2.4.3. Reference to Figure 2.4.3-16 is being added to SSAR Subsection 2.4.4.5, "Coincident Wind Wave." See the addition of Figure 2.4.3-16 in Attachment 3 and SSAR Subsection 2.4.4.5 markup in Attachment 4.

Attachment 1
Site Safety Analysis Report Subsection 2.4.1 Markups

SSAR Subsections 2.4.1.2.1 is being revised as indicated. Strikethroughs indicate text to be deleted. Underlines indicate text to be added.

2.4.1.2.1   Surface Water

Site Location

The CRN Site is located on the north bank of the Clinch River in the upper reach of Watts Bar Reservoir between CRM 19 and CRM 14.5 (Reference 2.4.1-1). The drainage area at CRM 16.0 is 3382 sq mi. The nearest facility downstream of the CRN Site with ~~an active~~ a surface water withdrawal ~~permit~~ registration with the State of Tennessee is the Oak Ridge Bear Creek Plant, located downstream of the CRN Site as shown on Table 2.4.1-1 and Figure 2.4.1-1 (Location No. 45). This plant is also known as the City of Oak Ridge's West End Water Treatment plant (WTP) and the K-25 Water Treatment Plant. The Oak Ridge Bear Creek Plant ceased water production on September 30, 2014, and Oak Ridge Utilities which now owns the facility has no plans to resume production at the site ~~is the former Oak Ridge Water Treatment Plant~~. Surface water supplies withdrawn from the 350 mi stretch of the Clinch River between the headwaters near Tazewell, Virginia and the confluence with the Tennessee River at CRM 0 and Tennessee River Mile (TRM) 567.8 are shown in Figure 2.4.1-1 and listed in Table 2.4.1-1. (Reference 2.4.1-6).

Figure 2.4.1-4 is removed and page will be noted

"Figure Number 2.4.1-4 Not Used."

A1-2

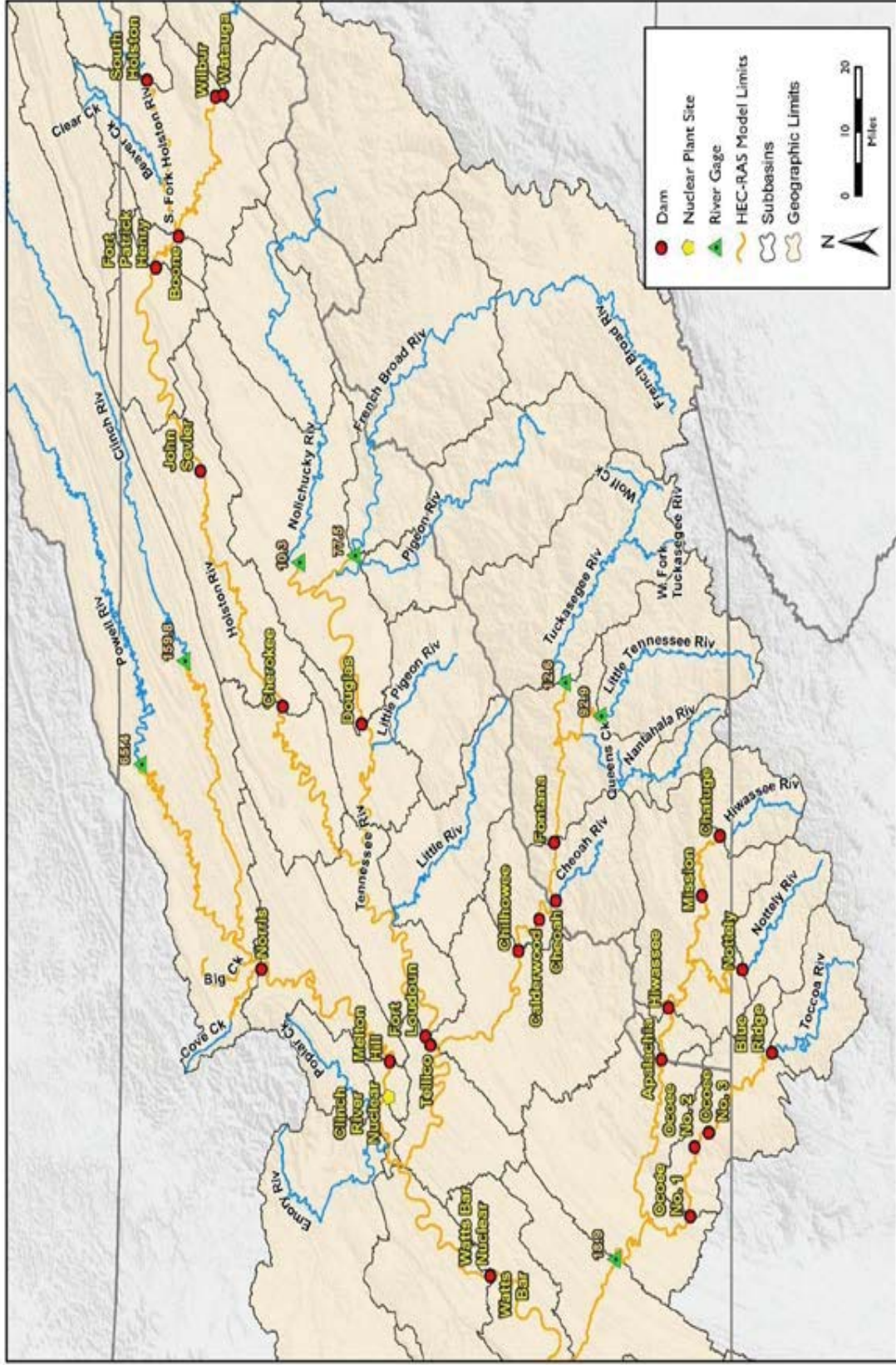**Existing Figure 2.4.1-5 is being replaced with new Figure 2.4.1-5, Sheets 1 and 2 of 2**

Figure 2.4.1-5 Tennessee River System (Sheet 1 of 2)
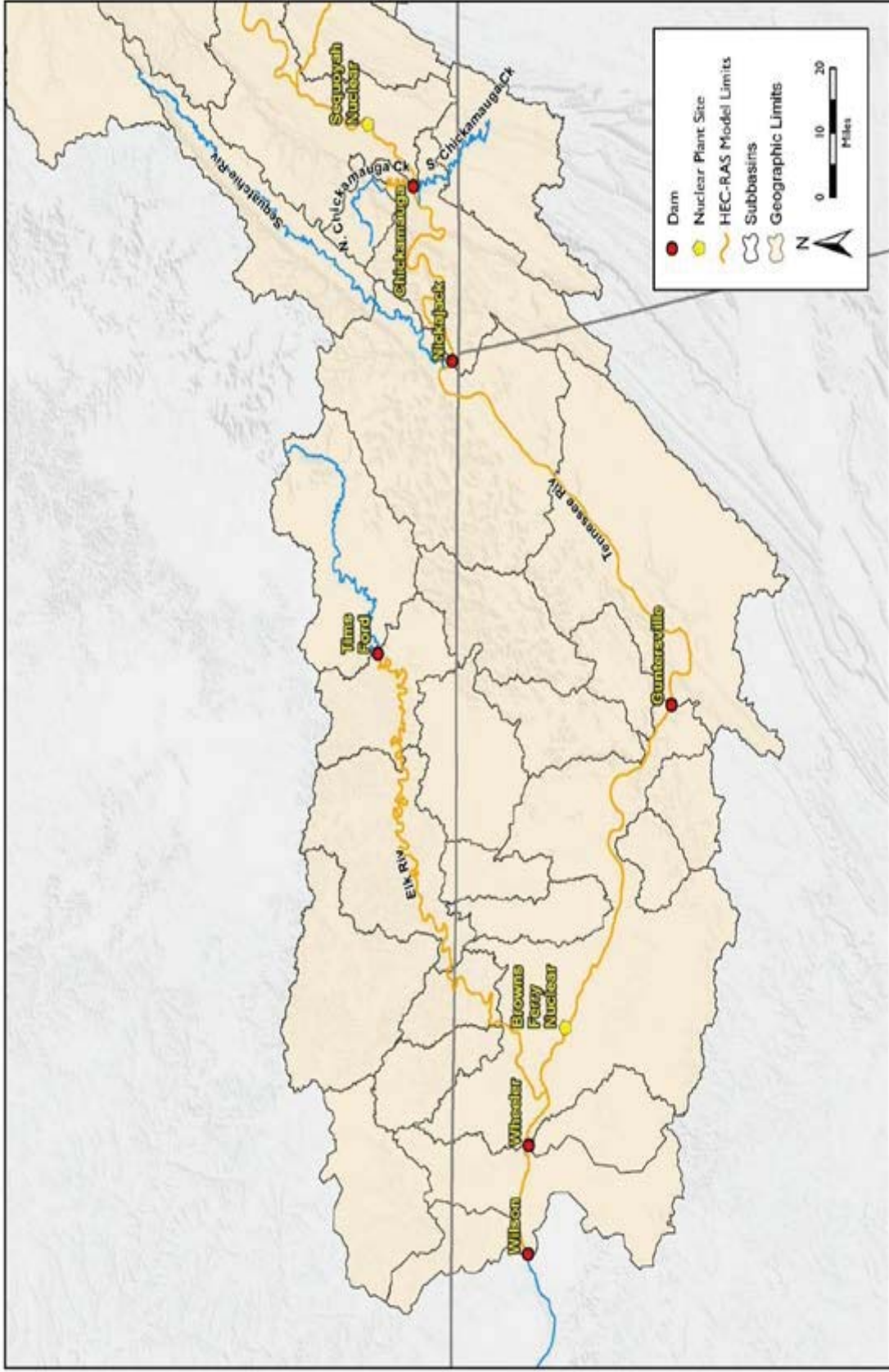
A1-3

Figure 2.4.1-5 Tennessee River System (Sheet 2 of 2)

A1-4

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Attachment 2
Site Safety Analysis Report Subsection 2.4.2 Markups

SSAR Subsection 2.4.2 is being revised as indicated. Strikethroughs indicate text to be deleted. Underlines indicate text to be added.

2.4.2.2   Flood Design Considerations

Flooding from Rivers and Streams

The condition producing the most critical flood level calculated at the CRN Site is the 7980 square mile (sq mi) Bulls Gap centered March storm event. This storm event produces a maximum flood level of ▮▮▮▮ ft NGVD29 with a peak discharge of 536,000 cubic feet per second (cfs) at the CRN Site. Dam failures associated with this event are discussed in Subsection 2.4.3.5. This elevation would result from the PMP critically centered on the watershed as described in Subsection 2.4.3. Consistent with Regulatory Guide 1.59, Watts Bar Dam was conservatively assumed to not fail even though the Watts Bar embankments are significantly overtopped.

. . .

Flooding f~~F~~rom Combined Effects

(SRI/CEII) Wind waves based on a calculated 2-year (yr) overwater wind speed of 33 miles per hour (mph) were assumed to occur coincident with the flood peak. This would create maximum wind waves up to ▮▮ ft high (trough to crest). When the effects of wind wave and runup were added, the maximum Clinch River design PMF water level was established to be at elevation ▮▮▮▮ ft NGVD29. The PMP and Flood Flow are discussed in Subsections 2.4.3.1 and 2.4.3.4. Dam failures associated with the combined effects of the PMF and wind are equivalent to the dam failures for PMF alone. Dam failures associated with the PMF alone are discussed in Subsection 2.4.3.5.

(SRI/CEII) The CRN Site and upstream reservoirs are located in the Southern Appalachian Tectonic Province and, therefore, subject to potential moderate earthquake forces with possible attendant failures (Reference 2.4.2-6). Upstream dams whose failure in a seismic event has the potential to cause flood problems at the CRN Site were investigated as described in Subsection 2.4.4.2.1. Studies to determine the potential failure of upstream dams from PMF conditions are described in Subsection 2.4.3. The half-10,000-yr Douglas centered seismic event with a coincident 500-yr flood produces a peak discharge of 162,000 cfs and a peak water surface elevation of ▮▮▮▮ ft NGVD29.

2.4.2.3.1   Precipitation Distribution

Temporal LIP distribution for the plant was determined from guidance presented in Hydro-Meteorological Report No. 56 and No. 52 (HMR-56 and HMR-52) (References 2.4.2-2 and 2.4.2-1, respectively). The guidelines set forth in HMR-56 and HMR-52 were followed to form the rainfall hyetograph of the 1-hour (hr), 1-sq mi PMP for CRN. PMP rainfall values in HMR-56 were chosen based on the terrain distribution roughness of the site, as shown in Figure 68 of Reference 2.4.2-2. The CRN Site falls in the rough zones in Figure 68. ~~Therefore, HMR-56 rough zone PMP rainfall values were used to calculate the CRN LIP.~~ The base LIP hyetograph for the CRN site is determined by adjusting the "rough terrain" precipitation depths in Table 6 of HMR-56 with the moisture index factor of 95.6 percent from HMR-56, Figure 20, representative of the CRN site location.

No specific guidance was provided in HMR-52 or HMR-56 to calculate the temporal distribution for rainfalls of one hour duration. A temporal distribution similar to that used for 72-hr storms with 6-hr increments (Section 2.3 HMR-52, Reference 2.4.2-1) was used to calculate the rainfall hyetograph for the 1-hr, 1-sq mi LIP.

Three temporal distributions were reviewed, with the peak 20 minutes of precipitation located either at the beginning (early peak), middle (middle peak), or end (late peak) of the 1-hr storm. With each 5-minute incremental precipitation depth from the adjusted 60-minute base LIP hyetograph labeled as D1 (initial 5-minute duration with largest incremental precipitation depth) to D12 (last 5-minute duration with the smallest incremental precipitation depth), the incremental LIP precipitation distribution for each case is defined consistent with HMR-52, Section 2.3 guidance (Reference 2.4.2-1):

1. Early distribution: D4, D1, D2, D3 (initial 20-minutes), D5, D6, D7, D8 (middle 20-minutes), D9, D10, D11, D12 (last 20-minutes)
2. Middle distribution: D8, D7, D6, D5 (initial 20-minutes), D4, D1, D2, D3 (middle 20-minutes), D9, D10, D11, D12 (last 20-minutes)
3. Late distribution: D12, D11, D10, D9 (initial 20-minutes), D8, D7, D6, D5 (middle 20-minutes), D4, D1, D2, D3 (last 20-minutes)

The cumulative rainfall values are summarized in Table 2.4.2-2. Additional analysis will be performed at COLA with consideration of LIP temporal distributions.

Table 2.4.2-1
Floods on the Clinch River Arm of Watts Bar Reservoir

Flood Event	Elevation at CRM 16.0 (feet NGVD29)	Elevation at CRM 18.0 (feet NGVD29)
1867[1]	762.3	764.5
1886[2]	759.0	767.8
1918[2]	755.8	763.0
1973[3]	748.4	748.5
2003[3]	748.4	748.7

[1] Estimated from historical flood profiles in a 1951 TVA historical record drawing. Flood elevation occurred prior to the current regulated state of the Clinch River arm of the Watts Bar Reservoir. ~~plots~~

[2] Estimated from a Clinch River flood report developed by TVA in 1959. Historical flood elevations were derived from various sources including flood markings from photographs and tree carvings found by the United States Army Corps of Engineers, and observations of river stages and gage data by the National Weather Bureau and United States Geological Survey. Flood elevations occurred prior to the current regulated state of the Clinch River arm of the Watts Bar Reservoir. (Reference 2.4.2-5)

[3] Estimated from HEC-RAS calibrations described in Section 2.4.3.4.3.

SSAR Subsection 2.4.3 is being revised as indicated. Strikethroughs indicate text to be deleted. Underlines indicate text to be added.

2.4.3.2 Probable Maximum Precipitation

The candidate storms having the potential to create maximum flood conditions at the CRN Site consist of four events: a PMP storm centered over the watershed upstream of the CRN Site; a PMP storm centered over the watershed upstream of Norris Dam; a PMP storm centered over the watershed upstream of the CRN Site and downstream of Norris Dam; and one additional PMP storm with the potential to maximize the flood levels on the Tennessee River system at the Watts Bar Reservoir. These PMP storms define depth-area-duration characteristics of rainfall and their seasonal variations and antecedent storm potentials. Because the watershed lies in the temperate zone, snowmelt is not a factor in generating maximum floods at the CRN Site (See page 97 of Reference 2.4.3-1).

The first event is a PMP storm centered over the 3382 sq mi watershed upstream of the CRN Site at CRM 16. The Norris and Melton Hill projects are located in this watershed and provide flood control for the downstream areas. The Hydrometeorological Branch of the NWS, in HMR-51 (Reference 2.4.3-2) and HMR-52 (Reference 2.4.3-3) as well as 1973 correspondence between TVA and the National Oceanic and Atmospheric Administration (NOAA), have provided guidance on defining this event. These publications outline the methods to use in the calculation and application of PMP storms for watersheds of 10 to 20,000 sq mi in size and are generalized for areas east of the 105th meridian.

The stepwise process followed to distribute the storm-area averaged PMF from HMR-51 over the 3382 sq mi watershed is described in HMR-52, Section 7. PMP depths for 10 sq mi to 20,000 sq mi basins and durations from 6 hours to 72 hours are scaled from Figures 18 through 47 in HMR 47. Using this PMP depth data, curve fits are used to define the area versus precipitation depth relationships for 6-hour, 12-hour, 24-hour, 48-hour, and 72-hour storm durations. With these relationships, precipitation depths are applied to standard isohyet area sizes defined in HMR 52, Section 7.1.A and precipitation depth versus storm duration curves are developed for each standard isohyet. Cumulative and incremental precipitation depths at 6-hour intervals up to 72 hours are then determined for four HMR-52 standard drainage areas smaller and four standard drainage areas larger than the 3382 sq mi watershed. The next step in the HMR-52 process is to determine the bounding storm size using the initial three 6-hour incremental precipitation depths and applying the adjustment factors provided in HMR-52, Tables 15 through 17. After determining the critical isohyetal pattern of rotation as described in HMR-52, Section 7.1.B and centering the isohyet pattern over the basin centroid, GIS is used to determine the area associated with each of the standard HMR isohyets on the watershed which, multiplied by the incremental rainfall depth, provides the precipitation volume. This process provides the volume associated with the four standard HMR-52 storms smaller and the four standard storms larger than the 3382 sq mi watershed. Of the eight standard storms, the 6500 sq mi storm area places the maximum precipitation volume on the 3382 sq mi watershed. Using the 6500 sq mi storm area depth-duration curve and applying the HMR-52 Tables 15-18 adjustment values, the controlling precipitation depth for each HMR-52 standard isohyet at 6-hour intervals up to 72 hours can be determined. GIS is then used to determine the incremental average precipitation depth applied to each sub-basin contributing to the 3382 sq mi watershed.

The second event is a PMP storm centered over the 2912 sq mi watershed upstream of Norris Dam. While the Norris project provides flood control for the downstream areas, this event was considered because of potentially higher Norris water surface elevations resulting in higher uncontrolled Norris discharges. The NWS HMR-56 report (Reference 2.4.3-4) provided guidance on defining this event. This publication outlines the methods to use in the calculation and application of PMP storms for watersheds less than 3000 sq mi in size and is specific to the Tennessee Valley. The development of the PMP for the 2912 sq mi watershed follows a process similar to the process defined above for the 3382 sq mi watershed.

The third event is a PMP storm centered over the 469 sq mi watershed upstream of CRM 16 and below Norris Dam. While the Melton Hill project is located in this area, it has a limited flood control storage volume making a PMF over this area essentially uncontrolled at the CRN Site. This storm was also defined using guidance from the NWS HMR-56 report (Reference 2.4.3-4). The development of the PMP for the 469 sq mi watershed follows a process similar to the process defined above for the 3382 sq mi watershed.

The fourth storm considered was selected as a candidate to determine maximum flood levels on Watts Bar Reservoir. Two storms, defined in the NWS HMR-41 report (Reference 2.4.3-1), were considered. One candidate storm event was a 21,400 sq mi PMP event whose defined spatial pattern was centered over the downstream portion of the Tennessee Valley watershed above Chattanooga, Tennessee. The second storm event was defined from an idealized elliptical pattern that was originally centered over the 7980 sq mi area above Chickamauga Dam and below the major tributary storage dams, but, with HMR-41 guidance, was subsequently allowed to shift upstream to be centered at Bulls Gap, Tennessee, with the intent of maximizing rainfall above Watts Bar Dam. This Bulls Gap centered storm produced a higher flood elevation above Watts Bar Dam than the 21,400 sq mi PMP storm event and thus was selected as the fourth storm.

The PMP development for the 7980 sq mi event was determined using GIS software to process PMP isohyets for the given storm centering. Centerings considered included a 7980 sq mi March and June rainfall pattern at Bulls Gap, Tennessee, as well as a 7980 sq mi March rainfall pattern downstream at Sweetwater, Tennessee. Isohyets were scanned from HMR 41, Figure 7-2. The scans were geo-referenced using ESRI ArcMap 9.2 GIS software and then vectorized via ArcScan. The resulting arc vectors were then adjusted for position and scale to achieve the best overall visual match between the outside watershed boundary and current GIS watershed boundary data as well as the latitude/longitude tic marks shown on the scans. The resulting patterns were then overlaid on the project sub-watersheds and reservoirs. GIS software was used to calculate the weighted average rainfall depth over each sub-watershed and reservoir. Depths were weighted by their respective areas. The per sub-basin PMP development for the 7980 sq mi event was determined using GIS software to process PMP isohyets for the given storm centering. The 72-hour rainfall depth for each sub-basin area above Watts Bar Dam (sub-basins 1 through 36) is provided in Table 2.4.3-2 for the 7980 sq mi March centering at Bulls Gap, Tennessee. For this controlling event, the average rainfall on the total drainage basin above Watts Bar Dam is 17.02 inches, as determined by the sum of the 72-hour rainfall volume on sub-basins 1 through 36, divided by the sum of the sub-basins area.

The 3380, 2912, and 469 sq mi PMP storms are modeled as nine-day events. A three-day antecedent storm was postulated to occur three days prior to the three-day PMP storm in each PMF determination. Rainfall depths equivalent to 30 percent of the main storm were used for the antecedent storms for the 3380, 2912, and 469 sq mi storms uniform areal distribution. These conditions are as recommended in HMR-56 report (Reference 2.4.3-4).

The 7980 sq mi PMP event is also modeled as a nine-day event with a similar three-day antecedent storm, three-day dry period, and three-day main storm pattern. Antecedent storm rainfall depths applied were equivalent to 40 percent of the main storm with a uniform areal distribution. The HMR-41 report (Reference 2.4.3-1) states that a subsequent rainfall is applicable for this storm. However, the peak elevation at the CRN Site during this PMF event occurs about 12 hours before the beginning of any subsequent rainfall, during a period when any subsequent rainfall induced increased flows could not compensate for the rate at which the upstream dams failure discharges are decreasing.

Temporal distribution patterns were adopted for all events based upon major observed storms transposable to the Tennessee Valley and distributions used by Federal agencies. The adopted distributions were within the limits stipulated in Chapter VII of HMR-41 (Reference 2.4.3-1) or Section 2.2.14 of HMR-56 (Reference 2.4.3-4) as applicable. These distributions placed the heaviest precipitation in approximately the middle of the ~~respective~~ antecedent and main storms. The twelve 6-hour rainfall increments of each 72-hour storm, were ordered from D1 (maximum depth) to D12 (smallest depth) and applied in each 72-hour duration in the following sequence: D12, D11, D10, D9 (first 24 hours), D2, D1, D3, D4 (middle 24 hours) and D5, D6, D7, D8 (last 24 hours). The adopted sequence closely conforms to the method used by the U.S. Army Corps of Engineers (USACE) (Reference 2.4.3-3). A typical distribution mass curve resulting from this approach is shown in Figure 2.4.3-2 and the controlling 7980 sq mi Bulls Gap centered storm temporal distribution is shown in Table 2.4.3-3.

As shown in Table 2.4.3-1, the PMP event producing the highest PMF water surface elevation at the CRN Site was determined to result from the 7980 sq mi Bulls Gap centered storm producing PMP on the watershed as defined in HMR-41 (Reference 2.4.3-1). The PMP storm having the largest seasonal precipitation occurs in March and would produce 17.0~~5~~2 inches of rainfall in three days on the watershed above Watts Bar Dam (Reference 2.4.3-1). 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.

2.4.3.3   Precipitation Losses

No precipitation losses were assumed. One-hundred percent of rainfall was assumed to be precipitation excess.

~~For PMF analysis, unit hydrographs were adjusted to reflect the nonlinearity of the runoff generation process under field conditions as recommended by NUREG/CR-7046. Peak discharge was increased by 20 percent and the time-to-peak was decreased by one-third. Unit hydrograph ordinates were then adjusted to preserve the unit hydrograph volume.~~

2.4.3.4   Runoff and Stream Course Models

2.4.3.4.1   Runoff Model

The runoff model used to determine flood hydrographs on the Clinch River arm of Watts Bar Reservoir at the CRN Site is divided into 65 sub~~areas~~-basins and includes the total 30,747 sq mi watershed above Wilson Dam.

Unit Hydrograph Development and Validation

In the 1960s through the early 1980s, the TVA Water Management Group developed hydrographs for sub-basins from direct rainfall inputs convoluted with unit hydrographs developed specifically for each sub-basin.

Using the process of "convolution", the direct runoff (stream flow minus base flow) hydrograph is determined from a series of M excess rainfall inputs of any depth and the K ordinates of the unit hydrograph. The N = K + M -1 ordinates of the direct runoff hydrograph are given by the discrete convolution equation, which states that the direct runoff Q_n at a given time n is obtained from the excess runoff P_m and the unit hydrograph ordinate U_{n-m+1} (where $U_i = 0$ for all i = n – m + 1 > K) as follows:

$$Q_n = \sum_{m=1}^{n \leq M} P_m U_{n-m+1}$$

The reverse process, called deconvolution, is used to derive the ordinates of the unit hydrograph (U), from excess rainfall (P) and direct runoff (Q) derived from observed data.

The TVA Water Management unit hydrographs, created from observed rainfall and stream flow, and reservoir headwater and discharge data, were validated by checking the unit hydrograph performance in reproducing recent floods.

The methodology used for unit hydrograph validation followed ANSI/ANS-2.8-1992. For the purpose of validating the unit hydrographs, the period of record from which the highest two or more floods were selected extended from 1997 through 2007. This period was targeted because high resolution, radar-based, hourly precipitation data are available for this period. Where suitable floods were not found within the 1997 to 2007 period, data back to 1985 was considered. Because the original TVA Water Management hydrographs were developed from floods that occurred between 1940 and1973, the use of recent rainfall and stream flow data considered the changes in watershed characteristics over the intervening years.

The hydrograph validation generally included the following steps:

1. Historical stream flow data were screened to identify significant floods that occurred subsequent to those used to develop the sub-basin unit hydrograph. The more recent floods are used in unit hydrograph validation.

2. The observed hydrograph data for the more recent floods were obtained and the flow series was transferred to the sub-basin outlets using established hydrologic procedures as necessary (e.g. reverse reservoir routing or stream flow routing and hydrograph separation) to develop the local basin hydrograph.

3. Base flow was separated from the local basin hydrograph to obtain the "observed" direct runoff hydrograph for the basin and the volume of direct runoff determined based on hydrograph ordinates.

4. Observed rainfall data for the selected floods were obtained and the basin average precipitation determined for the adopted time step.

5. The observed rainfall series was converted to an effective rainfall series using the TVA Antecedent Precipitation Index (API) method. This includes inputting the observed runoff volume obtained in Step 3 to ensure that the effective rainfall volume calculated equals the observed runoff volume.

6. Utilizing the TVA unit hydrograph and the effective rainfall series as input, HEC-HMS was run. The resulting simulated hydrograph was compared to the observed direct runoff hydrograph in terms of total volume as well as the timing and magnitude of peak discharge.

If observed flow at the outlet to a sub-basin was not available and the time distribution of local runoff could not be reliably estimated with simple lag routing, approximate methods were utilized. The total volume of local runoff was estimated as the difference in the volume of direct runoff calculated for the upstream and downstream observed hydrographs. This total volume was used to compute the effective rainfall hyetograph. The local flood hydrograph, generated by convolution of the excess runoff hyetograph with the TVA unit hydrograph developed for the sub-basin, was validated by comparing the simulated in- stream hydrograph with the observed hydrograph at the downstream basin outlet. ~~Runoff from Wilson sub-basins (66-69) was computed as if the entire Wilson subwatershed was the reservoir surface receiving constant rainfall for the entire event period equal to the highest single period rainfall. This is appropriate because this assumption was used only for the determining Wheeler Dam tailwater conditions. Above Wheeler Dam, sub-area unit hydrographs (UHs) and coefficients for any sub-basins requiring channel routing to reach model input locations were previously validated against the larger storms of record for that sub-basin.~~ Validated unit hydrographs, shown in Figure 2.4.3-17 Sheets 1 through 8, were used to compute model inflows from these areas. The watershed sub-basins are shown in Figure 2.4.3-1 and areas are included in Table 2.4.3-2.

For PMF analysis, unit hydrographs were adjusted to reflect the nonlinearity of the runoff generation process under field conditions as recommended by NUREG/CR-7046. Peak discharge was increased by 20 percent and the time-to-peak was decreased by one-third. Unit hydrograph ordinates were then adjusted to preserve the unit hydrograph volume.

National Inventory of Dams (NID) Considerations

Storage volumes from potentially critical projects (Reference 2.4.3-6) upstream of the model boundaries were identified and accounted for in the inflow hydrograph development. These additional volumes used the National Inventory of Dams (NID) to develop the additional inflow volumes to be applied. The USACE maintains the NID, which provides characteristics for each dam (location, height, and volume). The guidance for assessment of flooding hazards due to dam failure (Section 1.3.1 of Reference 2.4.3-6) requires a screening process to identify all dams that are potentially critical. In order to identify the number of structures upstream of the stream-course model limits, the NID was queried for the Tennessee Valley watershed above Wheeler Dam, identifying approximately 700 dams for inclusion in the analysis.

The NID data does not identify project stabilities, design basis capacities, spillway capacities or likely failure mode, so failures of NID projects during a PMP event are considered possible. Because of the large number of NID projects, the application of the additional inflow volume from the NID projects was simplified. Complete failure of all NID identified projects outside the model was conservatively assumed. Project failures were postulated to occur over time with some failures due to overtopping late in the antecedent event and the remaining projects failing before the end of the main storm rainfall. Considering the nine day PMF event (three day antecedent, three day dry period and three day main storm), the resulting failure volume was postulated to produce a rectangular, constant inflow hydrograph ~~Rectangular-shaped hydrographs were used at existing inflow locations to account for the volume of upstream small dams failing at varied times during the PMP event. These hydrographs were~~ distributed across 6 days, from one day after the peak antecedent precipitation to one day after the peak main storm precipitation (Figures 2.4.3-18, 19, and 20). Volumes were added to model inflows, translated (as needed), and distributed for input to the stream-course model.

2.4.3.4.2   Stream Course Model Extent

An unsteady flow model of the greater Tennessee River System was developed in the USACE Hydrologic Engineering Centers River Analysis System (HEC-RAS) to perform the unsteady flow routing of the Tennessee River System in a continuous simulation from upstream boundaries of Chatuge Dam on the Hiwassee River, Blue Ridge Dam on the Ocoee River, Nottely Dam on the Nottely River, River Mile 92.9 on the Little Tennessee River, River Mile 12.6 on the Tuckasegee River, River Mile 77.5 on the French Broad River, River Mile 10.3 on the Nolichucky River, South Holston Dam on the South Fork Holston River, Watauga Dam on the Watauga River, River Mile 159.8 on the Clinch River, River Mile 65.4 on the Powell River, Tims Ford Dam on the Elk River, and three small tributaries to the downstream boundary at Wilson Dam tailwater. Rainfall occurring in sub-basins upstream of the upstream boundaries of the unsteady flow model was computed and routed or translated downstream to the model boundaries where it was input as inflow hydrographs. The HEC-RAS unsteady flow model extends far enough upstream to allow PMF inflows to be input directly into the model and then hydraulically routed downstream. The western extent of the model, Wilson Dam, is approximately 270 Tennessee River Miles (TRM) southwest of Watts Bar Dam. However, dams and reservoirs modeled below the Chickamauga Dam, immediately downstream of Watts Bar Dam, have little impact on the predicted water elevations at the CRN Site.

2.4.3.4.3   Stream Course Model <u>Geometry Development</u> and Calibration

Main stem reservoir elevation-storage relationships (Figure 2.4.4-1, Sheets 1-12) were developed using historical TVA reservoir level-storage information and sediment range surveys. Reservoir areas were measured on composite maps consisting of USACE survey maps, TVA land maps, USGS topographic maps, and TVA navigation maps. GIS-based surface areas were used to compare historic data to more current data and to extend the elevation-storage relationships to projected flood elevations. An assessment of the acceptability of the elevation-storage curves was accomplished by comparing various sources of information (TVA dam project data, TVA level storage tables, steady flow simulation data and GIS-based surface area data) to determine consistency. Review of the graphical results of the assessment indicates consistent reservoir storage results and acceptability for use in the HEC-RAS simulation models. Tributary storage relationships were developed from historic TVA below-water-surface profiles supplemented with GIS data above the water surface.

Geometry profiles and effective flow areas of the main stem HEC-RAS model to the upstream boundaries at Norris, Cherokee, Douglas and Chilhowee dams and the Charleston gage on the Hiwassee River were derived from TVA historical hydrology model cross section data, USACE hydrographic survey data, topography of the water surface using DTM data and aerial photos. Where available, the historical information was validated for use based on comparisons to TVA silt range data, USACE survey data, USGS Quadrangle maps, and DTM. Cross-sections were adjusted as necessary to ensure that the topographic area was above the Probable Maximum Flood elevation.

After validation for use, cross-sections were oriented and located on USGS Quadrangle maps with contours every 20 feet. Cross-sections on the Tennessee River were generally spaced about 2 miles apart. Each cross-section was reviewed to ensure the location accurately represented the effective flow area for each reach of the reservoirs. In some cases, this review resulted in new cross sections being added or existing cross sections adjusted to remove ineffective flow areas.

The Highway 58 bridge (Figure 2.4.3-21) near Clinch River Mile (CRM) 14 was not included in the HEC-RAS geometry because the impact was considered to not be significant. A sensitivity analysis was performed by inserting additional cross-sections at the Highway 58 bridge location based upon bridge drawing obtained from the Tennessee Department of Transportation and surrounding topography. The sensitivity analysis indicated a negligible impact (less than 0.1 ft) at the CRN Site.

For most of the HEC-RAS model tributaries above the main stem boundaries, cross-section data were developed from historic cross-section data and verified as acceptable by comparison to cross-sections extracted from a GIS Triangulated Irregular Network (TIN). Cross-sections were then augmented to account for reach storage as needed. Cross-sections were oriented and located at approximately 1 to 2 mile intervals to represent the reach or channel restrictions.

Because cross-sections are typically located at constricted locations, the reach storage was evaluated and cross-sections adjusted with off-channel ineffective flow areas to more closely replicate reservoir elevation-storage relationships. Utilizing the TIN file of each reservoir or tributary, the volume between each cross-section was computed in GIS and a volume versus elevation curve was produced (Figures 2.4.3-22, 23, and 24). Because the volumes computed in GIS extended down only to the water surface, additional volume versus elevation data, presented in reservoir storage tables, were used for below the water surface. The computed volume between sections was then added to each section as an augmented ineffective flow area. Any new onsite fill would be added above the PMF elevation and would not affect the storage capacity in the reservoirs.

The developed HEC-RAS model geometry and input parameters (Manning's n values, etc.) were verified against observed historical floods. The main river reservoir models above Wheeler were verified against the March 1973 and May 2003 floods which were the largest large-scale floods of record since completion of the dams. The tributary reservoir models were verified against large available floods as well as 500-yr flood profiles.

Inflow development of the historic floods were included as a part of the unit hydrograph validation process. Observed rainfall data was obtained for the historic floods and used to calculate the basin average rainfall for each sub-basin. Excess precipitation was computed from the observed rainfall using the TVA's API loss methodology. Calculated effective rainfall volume was checked against the observed excess volume and losses were adjusted to ensure volume is preserved. The final excess precipitation is then computed, and an effective rainfall series for each historic flood event was generated. The computed effective rainfall series were input into USACE's HEC-HMS software to convolute the sub-basin unit hydrographs and generate direct runoff hydrographs. Observed streamflow data were obtained from gages at sub-basin outlets and compared to the computed direct runoff hydrographs in terms of total volume, timing, and magnitude of peak discharge. Once validated, the direct runoff hydrographs for each historic flood event were input in the HEC-RAS model as local inflows for use in the calibration process.

The Clinch River portion of the model was divided into three individual models. The lower portion of the Clinch River from the confluence with the Tennessee River to Melton Hill Dam at ~~Clinch River Mile (CRM)~~ 23.1 was verified as part of the Watts Bar Reservoir model to the March 1973 and May 2003 flood events. The verification process was a multi-step process that first included a steady-state flat-pool storage comparison to verify that the volume contained in the HEC-RAS model is representative of the known reservoir ~~volume~~ elevation-storage relationships shown in Figure 2.4.4-1, Sheets 1-12. The model segments of the Watts Bar Reservoir including the Lower Clinch River were then combined into a single model and run under unsteady-flow conditions to replicate the 1973 and 2003 floods events. The boundary conditions were the recorded discharges for Fort Loudoun Dam and Melton Hill Dam (upstream

boundary conditions) and the recorded headwater elevations for Watts Bar Dam (downstream boundary conditions) for both the 1973 and 2003 flood events. Tellico Dam recorded discharges were also an upstream boundary for the 2003 flood event, but Tellico Dam was not constructed at the time of the 1973 event. As a result, discharges used for the 1973 event upstream boundary at Tellico were computed inflows from the Little Tennessee River. Local inflow hydrographs were input to account for local inflows. For the 1973 flood, calculated flood elevations were compared to the observed elevations at three locations and calculated discharges were compared to observed discharges at Watts Bar Dam. For the 2003 flood, calculated flood elevations were compared to the observed elevations at three locations and calculated discharges were compared to observed discharges at Watts Bar Dam. To improve how well the HEC-RAS model reproduced the observed elevations, the Manning's n values for each of the model segments were evaluated and adjusted as needed. The model was then rerun and the results again compared to the observed elevations. After adjusting the Manning's n values, the model reproduced the historical floods with good agreement at the gage locations for the two events, so the verification was considered complete (see Figures 2.4.3-10 and 2.4.3-11). The model results were approximately equivalent to the 1973 flood at the Melton Hill Dam tailwater and reproduced the 2003 flood within one foot of the peak headwater elevation. The modeled peak flood elevations were conservatively higher than the observed elevations.

The portion of the Clinch River from Melton Hill Dam at CRM 23.1 to Norris Dam at CRM 79.8 was also verified by the March 1973 and May 2003 flood events. The verification process was a multi-step process that first included a steady-state flat-pool storage comparison to verify that the volume contained in the HEC-RAS model is representative of the known reservoir volume. The model of Melton Hill Reservoir was then run under unsteady-flow conditions to replicate the 1973 and 2003 flood events. The boundary conditions were the recorded discharges for Norris Dam (upstream boundary conditions) and the recorded elevations for Melton Hill Dam (downstream boundary condition). Local inflow hydrographs were input to account for local inflows. For the 1973 flood, calculated flood elevations were compared to the observed elevations at two locations and calculated discharge was compared to observed discharges at Melton Hill Dam. For the 2003 flood, calculated flood elevations were compared to the observed elevations at one location and calculated discharge was compared to observed discharge at Melton Hill Dam. To improve how well the HEC-RAS model reproduced the observed elevations, the Manning's n values for each of the model segments were evaluated and adjusted as needed. The model was then rerun and the results again compared to the observed elevations. After adjusting the Manning's n values, the model reproduced the historical floods with good agreement at the gage locations for the two events, so the verification was considered complete (see Figures 2.4.3-12 and 2.4.3-13). The model reproduced the peak elevation at the observed locations of the 1973 flood within half a foot and reproduced the peak elevation of the 2003 flood within one and a half feet.

The furthest upstream portion of the Clinch River from Norris Dam at CRM 79.8 to CRM 153.6 and its tributaries (the Powell River from its confluence with the Clinch to Powell River Mile (PRM) 62.0; Big Creek from its confluence with the Clinch to Big Creek River Mile 11.8; and Cove Creek from its confluence with the Clinch to Cove Creek River Mile 12.2) were verified by the March 2002 and February 2003 floods and historical FEMA flood profiles. The verification process was a multi-step process that first included a steady-state flat-pool storage comparison to verify that the volume contained in the HEC-RAS model is representative of the known reservoir volume. In addition to the total reservoir volume, the distribution of storage from upstream to downstream within the reservoir is accurately maintained. The upstream model portions of the Clinch and Powell Rivers were run under steady-flow conditions and compared to the 100-yr and 500-yr FEMA flood profiles. To improve how well the HEC-RAS model reproduced the flood profiles, the Manning's n values for each of the model segments were evaluated and adjusted as needed. The model was then rerun and the results again compared to the FEMA flood profiles. The model of the upstream portions of the Clinch and Powell Rivers

closely reproduced the FEMA flood profiles. The model segments of the Norris Reservoir model including the Clinch River, Powell River, Big Creek and Cove Creek were then combined into a single model and run under unsteady-flow conditions to replicate the 2002 and 2003 flood events. The upstream boundaries of the model were CRM 153.6, PRM 62.0, Big Creek River Mile 11.8 and Cove Creek River Mile 12.2. The discharges used as the upstream flow boundary conditions were computed by dividing observed inflows at Norris Dam by drainage areas for each boundary. The downstream boundary conditions were the observed stage and discharge hydrographs at Norris Dam. Local inflow hydrographs were also computed based on drainage area. When Norris dam discharges were used as the downstream boundary for the 2002 and 2003 flood events the computed Norris headwater elevation hydrographs reproduced the historical floods within one foot, so the verification was considered complete (see Figures 2.4.3-14 and 2.4.3-15). No additional Manning's n value changes were required. The model reproduced the peak elevation of the two historical floods within one foot at the Norris Dam headwater. The modeled peak flood elevations at the Norris Dam headwater were conservatively higher than the observed elevations.

2.4.3.4.4   Design Storm Implementation

Reservoir operating guidelines are implemented as prescribed operating ranges of reservoir levels throughout the year. The development of reservoir specific guidelines, or flood operational guides, are is based on original project allocations and subsequent modifications, many years of historical flows, flood season conditions and experience with project and reservoir system operations. The reservoir specific operating guides employed in the HEC-RAS model are based on the 2004 River Operations flood risk evaluation study and the resulting changes in reservoir operating policy as approved by TVA.

Seasonal operational guides provide normal pool starting elevations throughout the year. Median, normal pool 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 HEC-RAS model used unsteady flow rules at each dam for the purpose of prescribing discharges based on either flood operational guides or dam rating curves. Prior to all outlet gates being fully open, the primary guide curve portion of the flood operational guides are applicable for attempting to regulate the downstream impacts of a flood event via prescribed discharges at given headwater elevations. As the flood recedes the recovery curve portion of the flood operational guide prescribes discharges at given headwater elevations with the goal of recovering reservoir flood storage in preparation for the next potential storm event.

Figure 2.4.3-4 demonstrates the current operational guide for mid-March at Norris Dam. At Point (Pt) A (starting point) on the curve, the turbine discharge is 4500 cfs and discharge is controlled by the spillway gates, turbine outflow, and sluices. As flood waters enter the reservoir and increase the headwater to 1005 feet, turbine operation ceases and sluices and spillway gates are closed (Pt B to C) (Figure 2.4.3-4). As flood waters continue to enter the reservoir with the discharge outlets closed, the reservoir elevation reaches 1032 feet (Pt D). At elevation 1032 feet, spillway gates begin to open and sluiceway operation is adjusted to allow an increasing discharge until the headwater elevation reaches 1034 feet (Pt E). At this elevation, gates continue to be raised to increase flow and sluices and turbines are adjusted to maintain 1034 feet headwater elevation (Pt F) until discharge points are at fully capacity and flow transitions to the dam rating curve. As additional flood waters enter the reservoir, the reservoir headwater elevation response is as indicated by Case 1 of the Norris Dam rating curve, Figure 2.4.3-7. As the flood waters recede, the operational guides follow the flood accession curve in reverse order to Pt E on Figure 2.4.3-4 at a discharge of 24,000 cfs. At this point, spillway gates and sluices are operated to decrease reservoir elevation until elevation 1015 feet

is attained while discharge flow is maintained at 24,000 cfs. Reservoir elevation is further reduced to 1006 feet at a 16,000 cfs discharge, followed by a recovery to elevation 1004 feet at a 9000 cfs and then a recovered flow rate of 4500 cfs using gate and sluice operation.

~~In addition,~~ Seasonal variability is incorporated into the flood operational guides and implemented in the unsteady flow rules. Once outlet capacity has been exceeded, discharges are calculated using the dam rating curves. Dam rating curves, developed for the key dams above Watts Bar Dam, provide the relationship between the surface water elevation of the reservoirs and the discharge past the dam structure. Using the configuration of the concrete and embankment dams with the potential discharge outlets defined by TVA drawings, discharges at varying reservoir elevations were determined using standard weir flow equations. Tailwater and submergence effects were considered in the discharge determination. Discharge coefficients in weir flow equations were based on USACE standards or TVA or industry model test experiments. Alternative dam rating curves were developed as needed for potential dam failure considerations. The dam rating curves ~~are~~ resulted in sets of equations implemented in the unsteady flow rules to define total dam discharge as a function of headwater elevation, tailwater elevation, and outlet configuration (normally all gates open). If, as during a PMF event, headwater exceeds the normal operating range, the dam rating curves determine flow over other components such as non-overflow sections, navigation locks, tops of open spillway gates, tops of spillway piers, saddle dams, rim leaks, and most postulated dam breaches. For any dam breach whose base was postulated to reach the bottom of the stream channel, internal HEC-RAS computations were used instead of weir equations calculating discharge using unsteady flow rules. If the operating deck elevation is not exceeded by the floodwater surface elevation and there are no postulated dam breaches, operations return to the flood operational guides during the flood recession. Plots of the flood operational guides ~~and dam rating curves~~ for the three dams that control the water flow at the CRN Site (Norris Dam, Melton Hill Dam and Watts Bar Dam) are provided in Figures 2.4.3-4 through 2.4.3-69. Dam rating curves are provided in Figures 2.4.3-7 through 2.4.3-9 for Norris, Melton Hill and Watts Bar Dams. These dam rating curves show multiple cases addressing various dam configurations (including potential failures) as well as turbines turned on and off. All or portions of each curve were utilized for CRN Site simulations except Cases 3 through 6 for Watts Bar Dam. Cases 3 through 6, various dam failure configurations, were not used in CRN Site simulations because Watts Bar Dam was conservatively assumed to remain stable for all load cases.

2.4.3.5 Probable Maximum Flood Flow

The maximum discharge at the CRN Site resulting from the 7980 sq mi, Bulls Gap centered, March PMP event was determined to be 536,000 cfs. The maximum discharge resulting from the 3382 sq mi event was determined to be slightly higher, 544,000 cfs. However, the 7980 sq mi, Bulls Gap event is the controlling PMF event because of the higher elevation. The PMF discharge hydrograph is shown in Figure 2.4.3-3.

(SRI/CEII) The PMF event would overtop and breach ██ These are the only dams that would fail, and they were assumed to fail instantaneously and either totally or as prescribed by the Von Thun and Gillette method (References 2.4.3-7 to 2.4.3-9). ██████ Dam remained stable and ████████ Dam was assumed not to breach to provide bounding backwater conditions at the CRN Site. The analysis of dam failures is described in Subsection 2.4.4.

March reservoir levels were used at the start of the antecedent storm for the 7980 sq mi, Bulls Gap centered, March PMP event which yielded the largest seasonal precipitation (Reference 2.4.3-1). March reservoir levels represent winter pool levels. June reservoir levels were used at the start of the antecedent storm for the other three PMP events (the 3382 sq mi, the 2912 sq mi and the 469 sq mi events). June reservoir levels represent summer pool levels which are maintained as the highest normal pool levels of the year.

(SRI/CEII) The influence of the TVA reservoir system on the PMF was computed using operating procedures prescribed for floods. In addition to spillway flow, these permit turbine and sluice discharge in tributary reservoirs and turbine discharge at mainstream reservoirs until head differentials become too small because of tailwater rise in large flood flows. Flood gates were considered to be operable during the flood. Prescribed operating procedures have little influence on maximum flood discharge during a PMF event because spillway capacities and uncontrolled conditions are reached early in the main storm flood. Additionally, a sensitivity simulation was performed assuming reduced gate operability at ▮▮▮ Dam (all three gates remained closed) and ▮▮▮▮▮▮ Dam (20 percent reduction in available gates). This simulation resulted in overtopping failures of ▮▮▮ and ▮▮▮▮▮ Dams which produced an increase in elevation at the CRN Site of ▮▮ ft above the elevation produced by a PMF simulation without failures of ▮▮▮ and ▮▮▮▮▮ Dams. However, the increased elevation remains ▮▮ ft below the bounding design basis flood elevation of ▮▮▮ ft. Additionally, the possibility of all gates at Norris Dam being inoperable is not realistic because:

(SRI/CEII)
- TVA monitors gates daily for operation and the maintenance program for gates assures high reliability.
- TVA has the means and resources to resolve gate issues if needed to respond to flood events.
- The gates at ▮▮▮ Dam are drum gates which are reliable and do not rely on a crane for operation.

2.4.3.6   Water Level Determinations

(SRI/CEII) The controlling PMF would produce elevation ▮▮▮▮ ft National Geodetic Vertical Datum of 1929 (NGVD29) at the CRN Site. The bounding design basis elevation is established as ▮▮▮ ft NGVD29 to provide margin to the calculated value. The elevation hydrograph for the site is shown in Figure 2.4.3-3 and represents a point just upstream of the intake. Elevations were computed concurrently with the discharges for the site using the unsteady flow model.

2.4.3.7   Coincident Wind Wave Activity

Wind waves are likely when the controlling PMF crests at the CRN Site. The flood would be near its crest for one day beginning approximately two days after cessation of the PMP. The day of occurrence would likely be in the month of March.

(SRI/CEII) Wind waves to be associated with the PMF crest were computed using procedures of the USACE Coast Engineering Manual (Reference 2.4.3-5). Wind data from 2000 to the 2014 were collected at Huntsville, Alabama; Chattanooga, Knoxville, and Tri-Cities, Tennessee; and Asheville, North Carolina. The raw 2-minute average wind data were used to calculate the maximum 20-minute average wind speed for each year at each data collection site and the 2-yr wind speed was determined. The CRN Site overland wind speed of 28 mph was adjusted for overwater conditions, resulting in an overwater wind speed of 33 mph. The effective fetch found for the CRN Site from available GIS terrain data was 4.25 mi, based on the site grade elevation of 821 feet, and results in the critical site fetch length (Figure 2.4.3-16). For a calculated 33 mph overwater 2-yr wind, the total wave height of ▮▮ ft from crest to trough was calculated, which includes wave runup (▮▮ ft) and wave setup (▮▮ ft), resulting in a maximum elevation of ▮▮▮▮ ft NGVD29. CRN Site grade is 821.4 ft NGVD29 (821 ft North American Vertical Datum of 1988 [NAVD88]), ▮▮▮ ft higher than the maximum calculated water surface elevation with wind wave height. Because of the available margin, the coincident wind wave activity does not have an effect on flooding at the site.

(SRI/CEII) Additionally, wind waves may occur at ▮▮▮▮▮▮▮▮▮▮▮ Dams concurrent with the PMF crests at these dams. As discussed previously, ▮▮▮▮▮▮ earth embankments are assumed to be overtopped and to fail in the PMF. Adequate freeboard is available for the ▮▮▮▮ Dam embankments to prevent overtopping during the PMF.

2.4.3.8 References

2.4.3-1.   Schwarz, Francis K., Probable Maximum and TVA Precipitation over the Tennessee River Basin above Chattanooga, Hydrometeorological Report No. 41, Hydrometeorological Section, Office of Hydrology, U.S. Weather Bureau, U.S. Department of Commerce, Washington, D.C., June 1965.

2.4.3-2.   Schreiner, Louis C. and John T. Riedel, Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No. 51, Hydrometeorological Branch, Office of Hydrology, National Weather Service, U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA), U.S. Department of the Army, Corps of Engineers, Washington, June 1978.

2.4.3-3.   Hansen, E.M., L.C. Shreiner, and J.F. Miller, Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, NOAA Hydrometeorological Report No. 52, Hydrometeorological Branch, Office of Hydrology, U.S. Department of Commerce, NOAA, U.S. Department of the Army, Corps of Engineers, Washington, D.C., August 1982.

2.4.3-4.   Zurndorfer, E.A., F.K. Schwarz, E.M. Hansen, D.D. Fenn, and J.F. Miller, Probable Maximum and TVA Precipitation Estimates with Areal Distribution for Tennessee River Drainages Less Than 3,000 Mi2 in Area, Hydrometeorological Report No. 56, Hydrometeorological Section, Office of Hydrology, National Weather Service, U.S. Department of Commerce, NOAA, Silver Spring, Maryland, October 1986.

2.4.3-5.   USACE, Coast Engineering Manual, EM1110-2-1100, Pts. 2 and 6.

2.4.3-6.   United States Nuclear Regulatory Commission, Guidance for Assessment of Flooding Hazards Due to Dam Failure, Interim Staff Guidance, Report JLD-ISG-2013-01, ADAMS Accession No. ML13151A153, Rev. 0, July 29, 2013.

2.4.3-7   Von Thun, J Lawrence and David R. Gillette, Guidance on Breach Parameters, unpublished internal document, U.S. Bureau of Reclamation, March 13, 1990 (W50170329001)

2.4.3-8   Wahl, Tony, Prediction of Embankment Dam Breach Parameters: A literature Review and Needs Assessment, Dam Safety Office, Water Resources Research Laboratory, July 1998

2.4.3-9   BWSC, Breach Size Evaluation and Recommendation for NTTF Recommendation 2.1: Flooding, Technical Memorandum, December 19, 2012 (W50150129001)

(SRI/CEII)

Table 2.4.3-1
Flood Events Analyzed

Clinch River Event	Significant Failures Above Watts Bar Dam	Precipitation Depth Above CRN (in.)	Precipitation Depth Above Watts Bar Dam (in.)	Peak at CRN Site		
				Elevation (ft NGVD29)	Flow (cfs)	Velocity (fps)
2912 sq mi, centered above Norris Dam, June storm event	▮	19.20	7.89	▮	543,000	18.10
3382 sq mi, centered above CRN, June storm event		19.40	8.57	▮	544,000	16.42
469 sq mi, centered between CRN and Norris Dam, June storm event		8.73	3.72	▮	200,000	10.55
7980 sq mi, Bull's Gap centered, March PMF with 100% runoff and peaked/lagged unit hydrographs storm event		16.68	17.02	▮	536,000	7.79
Half-10,000-Yr Douglas Centered Seismic Event During A 500–Yr June Flood Event		NA²	NA²	▮	162,000	6.80
		NA³	NA³	▮	579,000¹	18.11

▮ Dam Sunny Day Failure

1. Water surface elevation at the CRN site is controlled by the backwater of Watts Bar Dam. As a result, the event producing the highest flow at the CRN Site does not result in the highest water surface elevation.
2. 500-yr flow based on estimate using gaged inflows.
3. Sunny day flows based on project storage only.

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Table 2.4.3-3
Temporal Rainfall Distribution for the 7980 Sq Mi PMP

Day of event	Hours since start	Distribution	Precipitation Depth above Watts Bar Dam (in.)	Source
	150	20% of 2ndDay	0.69	Table 7-2, HMR-41
7	156	23% of 2ndDay	0.80	Table 7-2, HMR-41
	162	27% of 2ndDay	0.94	Table 7-2, HMR-41
	168	30% of 2ndDay	1.04	Table 7-2, HMR-41
	174	2nd6hr	2.80	Table 7-2, HMR-41
8	180	1st6hr	5.39	Table 7-2, HMR-41
	186	3rd6hr	1.94	Table 7-2, HMR-41
	192	4th6hr	1.39	Table 7-2, HMR-41
	198	28% of 3rdDay	0.57	Table 7-2, HMR-41
9	204	26% of 3rdDay	0.53	Table 7-2, HMR-41
	210	23% of 3rdDay	0.47	Table 7-2, HMR-41
	216	23% of 3rdDay	0.47	Table 7-2, HMR-41

Figure 2.4.3-4 Sheets 1 and 2 are modified and replaced with new Figure 2.4.3-4 Sheets 1 and 2

Figure 2.4.3-4. Flood Operational Guide – Norris Dam (Sheet 1 of 2)

Legend:
- Primary Guide Curve (PGC) w/ Turbine
- Storage Recovery Curve (RC) w/o Turbine
- Antecedent Storm Transition to RC
- Main Storm Transition to PGC

If main storm transition elevation occurs in this range (1030 to 1034) then stay on RC to Pt E

HW elevation 12 hours after beginning of main storm go to PCG for operation (2)

HW elevation 24 hours after end of antecedent Storm rainfall (1)

Three feet above initial HW elevation at beginning of antecedent storm

Decision points
(1) - Decision #1 Antecedent transition to recovery (storm dependent, season dependent) must be calculated from initial run of TRBROUTE
(2) - Decision #2 Main storm transition (storm dependent & season dependent) must be calculated by TRBROUTE based on (1) above

Notes:
1. All transition points (dashed lines) are shown for illustration purpose only. Transitions are storm dependent.
2. The possibility exists that the antecedent storm exceeds point E elevation prior to transition.

Mid-March

Pt	A	B	C	D	E	F	a1	a2	a3	a4	a5	a6
Elevation (ft)	1001	1004	1004	1032	1034	1034	1004	1006	1006	1015	1015	1034
Discharge (cfs)	4,500	4,500	0	0	24,000	*	9,000	9,000	16,000	16,000	24,000	24,000

* - See dam rating curve for elevation/discharge at point F

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Transitions (1) and (2) occuring between Pt A and E go to MidMarch Curve for Details

If Main storm transition elevation occurs in this range (1030 to 1034) then stay on RC to Pt E

Decision points
(1) - Decision #1 Antecedent transition to recovery (storm dependent, season dependent) must be calculated from initial run of TRBROUTE
(2) - Decision # 2 Main storm transition (storm dependent & season dependent) must be calculated by TRBROUTE based on (1) above

Two feet above initial HW elevation at beginning of antecedent storm

— Summer Guide Curve
— Summer Recovery Curve (RC)

June 1

Pt	A	B	C	D	E	F	a1	a2	a3	a4	a5	a6
Elevation (ft)	1020	1022	1022	1032	1034	1034	1020	1023	1023	1027	1027	1034
Discharge (cfs)	4,500	4,500	0	0	24,000	*	9,500	9,500	16,000	16,000	24,000	24,000

* - See dam rating curve for elevation/discharge at point F

Figure 2.4.3-4. Flood Operational Guide –Norris Dam (Sheet 2 of 2)

A3-18
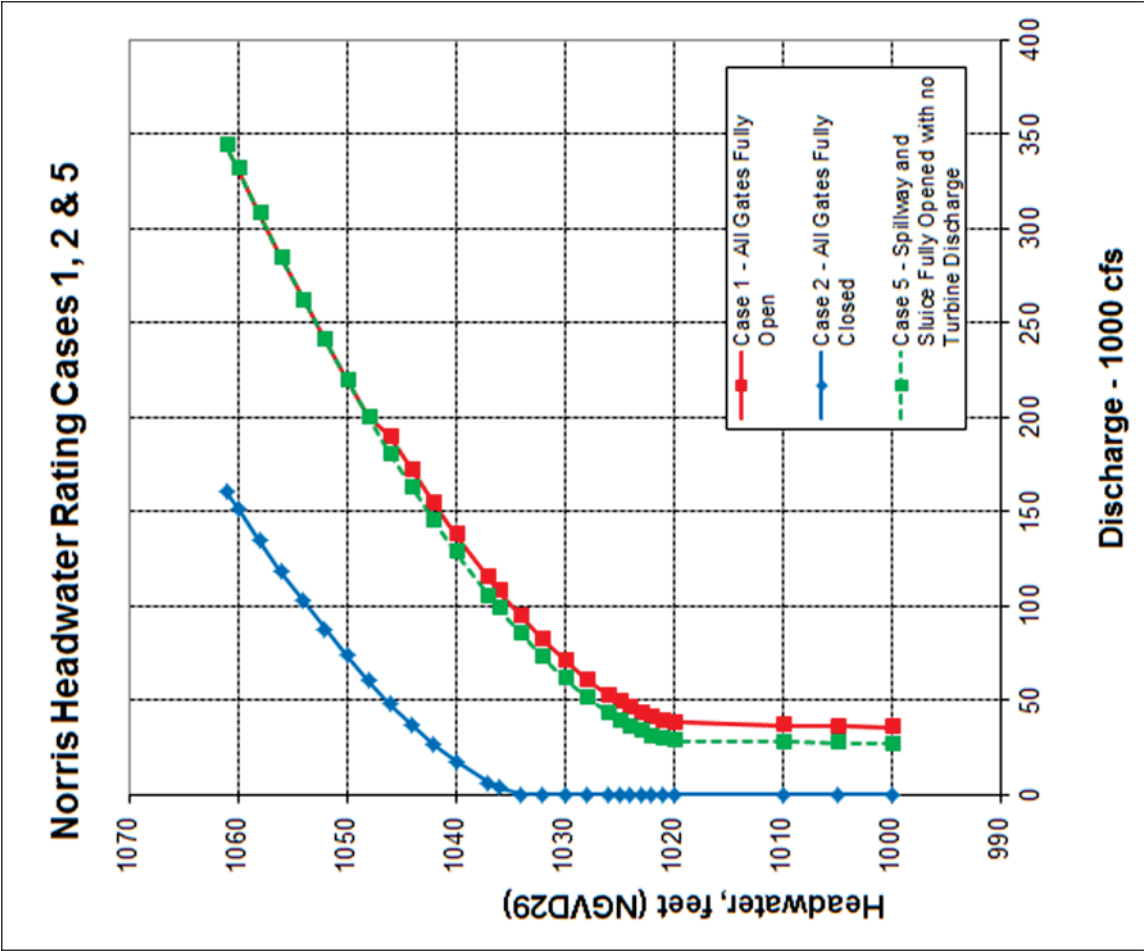
Figure 2.4.3-7. Dam Rating Curve – Norris Dam (Sheet 1 of 2)

A3-19

Figure 2.4.3-7. Sheet 2 of 2 is removed

Figures 2.4.3-10 to 2.4.3-24 are added
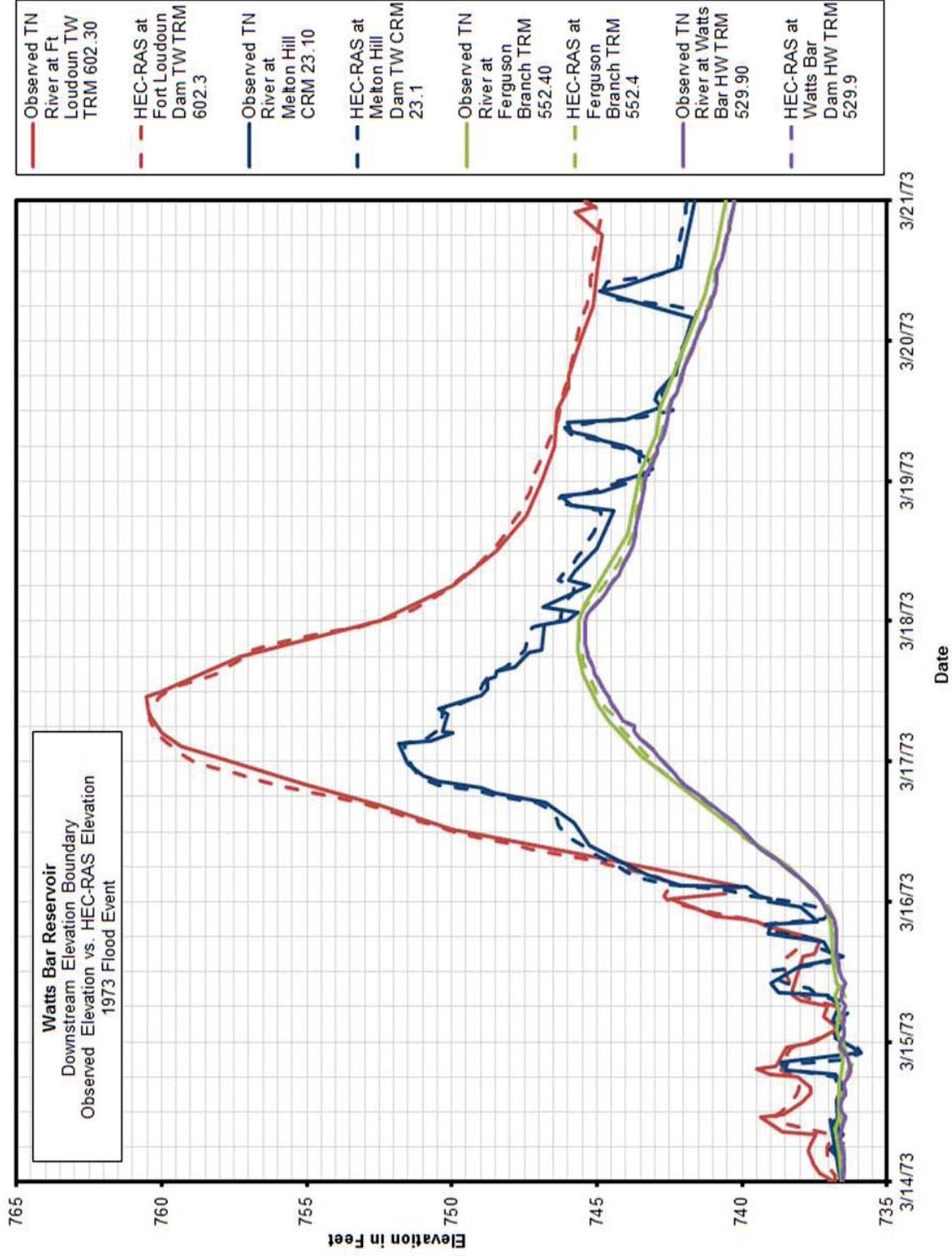
Figure 2.4.3-10. Watts Bar Reservoir Calibration Results - Elevation: 1973 Flood Event

Legend:
- Observed TN River at Ft Loudoun TW TRM 602.30
- HEC-RAS at Fort Loudoun Dam TW TRM 602.3
- Observed TN River at Melton Hill CRM 23.10
- HEC-RAS at Melton Hill Dam TW CRM 23.1
- Observed TN River at Ferguson Branch TRM 552.40
- HEC-RAS at Ferguson Branch TRM 552.4
- Observed TN River at Watts Bar HW TRM 529.90
- HEC-RAS at Watts Bar Dam HW TRM 529.9

Watts Bar Reservoir
Downstream Elevation Boundary
Observed Elevation vs. HEC-RAS Elevation
1973 Flood Event

Elevation in Feet
Date

Figure 2.4.3-11. Watts Bar Reservoir Calibration Results - Elevation: 2003 Flood Eve

Legend:

- Observed TN River at Ft Loudoun TW TRM 602.30
- HEC-RAS at Fort Loudoun Dam TW TRM 602.3
- Observed TN River at Melton Hill CRM 23.10
- HEC-RAS at Melton Hill Dam TW CRM 23.1
- Observed TN River nr Kingston, TN TRM 568.1
- HEC-RAS at Kingston, TN TRM 568.1
- Observed TN River at Watts Bar HW TRM 529.90
- HEC-RAS at Watts Bar Dam HW TRM 529.9

Watts Bar Reservoir
Downstream Elevation Boundary
Observed Elevation vs. HEC-RAS Elevation
2003 Flood Event

A3-23

Figure 2.4.3-12. Melton Hill Reservoir Calibration Results - Elevation: 1973 Flood Event
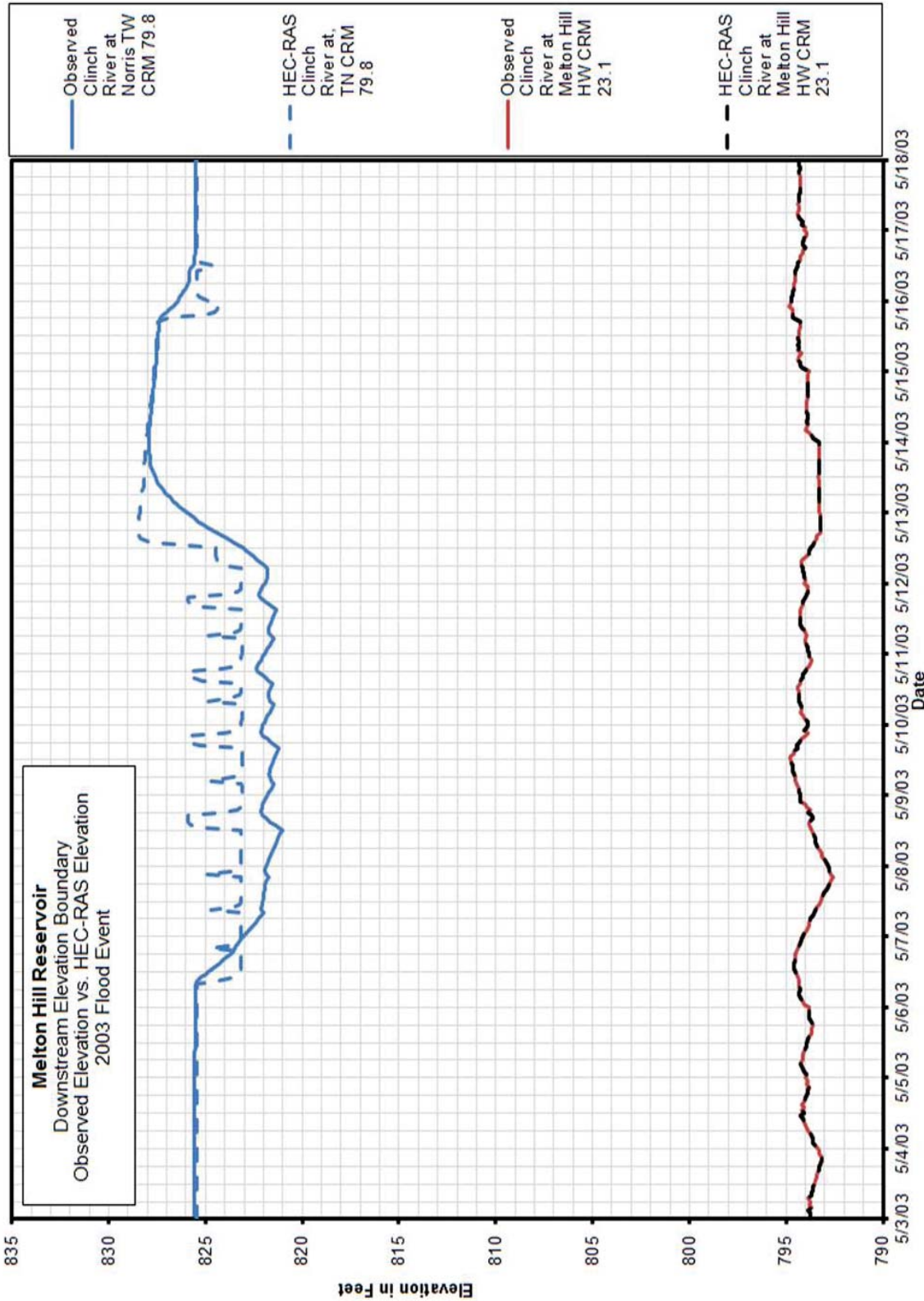
Figure 2.4.3-13. Melton Hill Reservoir Calibration Results - Elevation: 2003 Flood Event

A3-25
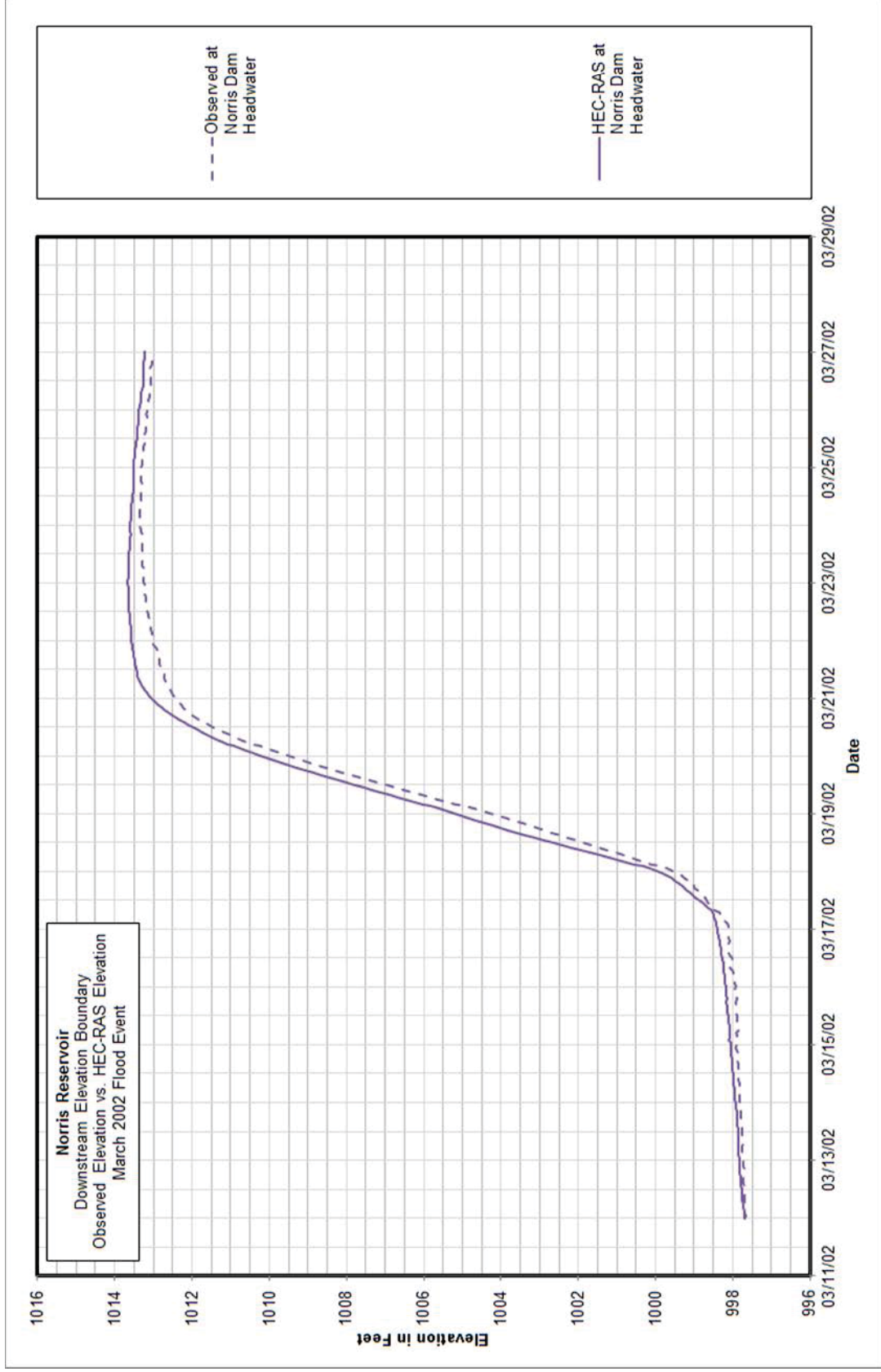
Figure 2.4.3-14. Norris Reservoir Calibration Results - Elevation: 2002 Flood Event

A3-26
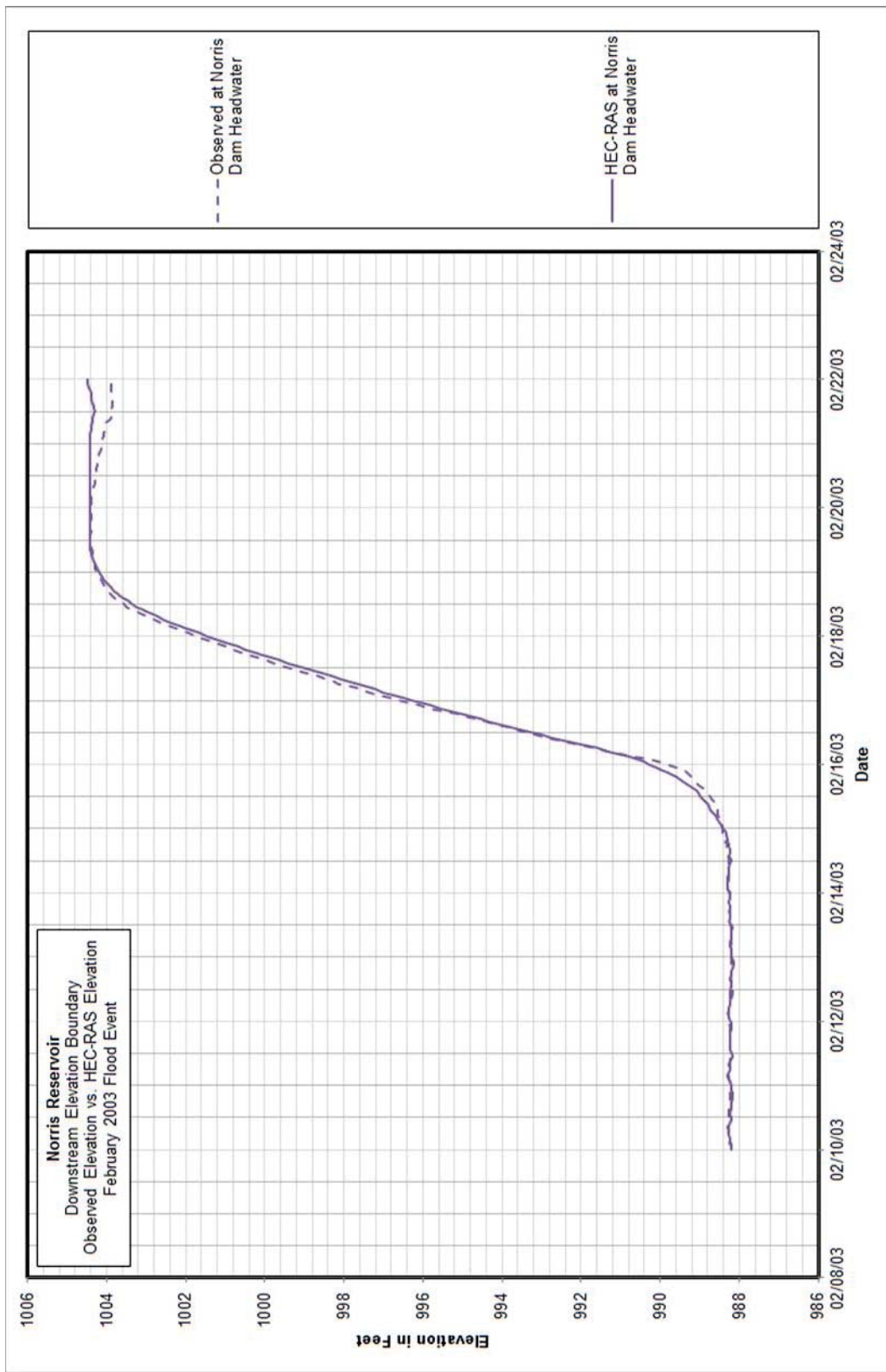
Figure 2.4.3-15. Norris Reservoir Calibration Results - Elevation: 2003 Flood Event

Figure 2.4.3-16. CRN Site Critical Fetch Length

Critical Site Fetch Length
821' Contour

N

0   2,000   4,000
Feet

A3-28

Figure 2.4.3-17. Unit Hydrographs, Sub-basins 1-6 (Sheet 1 of 8)

Legend:
— Sub-basin 1 - French Broad River at Asheville; 944.4 SQ.MI.; 6-Hour Duration
– – Sub-basin 2 - French Broad River, Newport to Asheville; 913.1 SQ.Mi.; 6-Hour Duration
– – – Sub-basin 3 - Pigeon River at Newport; 667.1 SQ.MI.; 6-Hour Duration
–·– Sub-basin 4 - Nolichucky River at Embreeville; 804.8 SQ.MI.; 4-Hour Duration
–··– Sub-basin 5 - Nolichucky Local; 378.7 SQ.MI.; 6-Hour Duration
······· Sub-basin 6 - Douglas Dam Local; 835.0 SQ.MI.; 6-Hour Duration

A3-29

Figure 2.4.3-17. Unit Hydrographs, Sub-basins 9-15 (Sheet 2 of 8)

Sub-basin 9 - South Holston Dam; 703.2 SQ.MI.; 6-Hour Duration
Sub-basin 10 - Watauga Dam; 468.2 SQ.MI.; 4-Hour Duration
Sub-basin 11 - Boone Local; 667.7 SQ.MI.; 6-Hour Duration
Sub-basin 12 - Fort Patrick Henry Dam; 62.8 SQ.MI.; 6-Hour Duration
Sub-basin 13 - North Fork Holston River Near Gate City; 668.9 SQ.MI.; 6-Hour Duration
Sub-basin 14 & 15 - Cherokee Local; 854.6 SQ.MI.; 6-Hour Duration

A3-30

Figure 2.4.3-17. Unit Hydrographs, Sub-basins 26, 27, 34 & 35 (Sheet 3 of 8)

Sub-basin 26 - Clinch River at Norris Dam; 2,918.2 SQ.MI.; 6-Hour Duration

Sub-basin 27 - Melton Hill Local; 431.9 SQ.MI.; 6-Hour

Sub-basin 34 - Poplar Creek at Mouth; 135.2 SQ.MI.; 2-Hour Duration

Sub-basin 35 - Emory River at Mouth; 868.8 SQ.MI.; 4-Hour Duration

A3-31
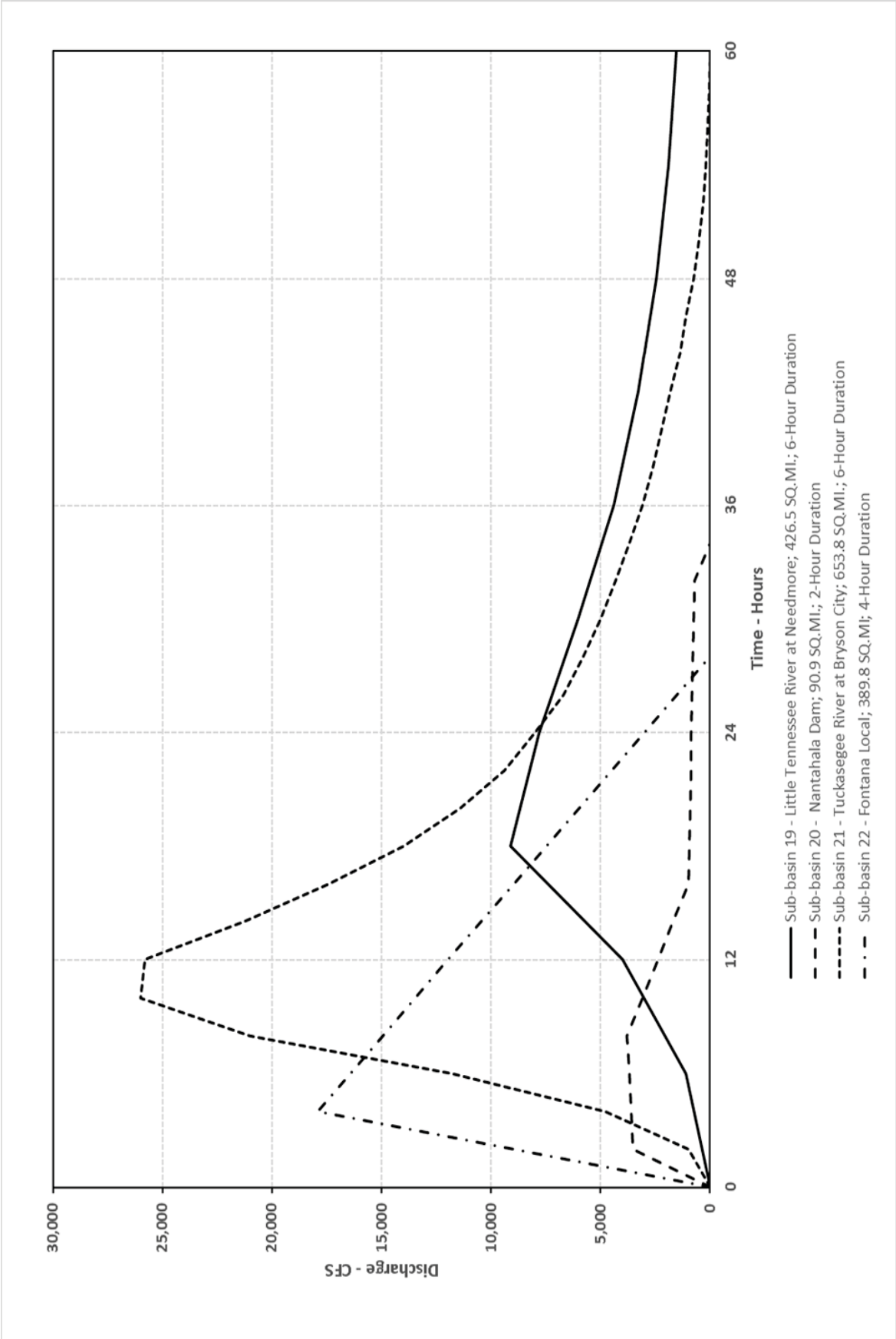
Figure 2.4.3-17. Unit Hydrographs, Sub-basins 19-22 (Sheet 4 of 8)

Sub-basin 19 - Little Tennessee River at Needmore; 426.5 SQ.MI.; 6-Hour Duration
Sub-basin 20 - Nantahala Dam; 90.9 SQ.MI.; 2-Hour Duration
Sub-basin 21 - Tuckasegee River at Bryson City; 653.8 SQ.MI.; 6-Hour Duration
Sub-basin 22 - Fontana Local; 389.8 SQ.MI; 4-Hour Duration
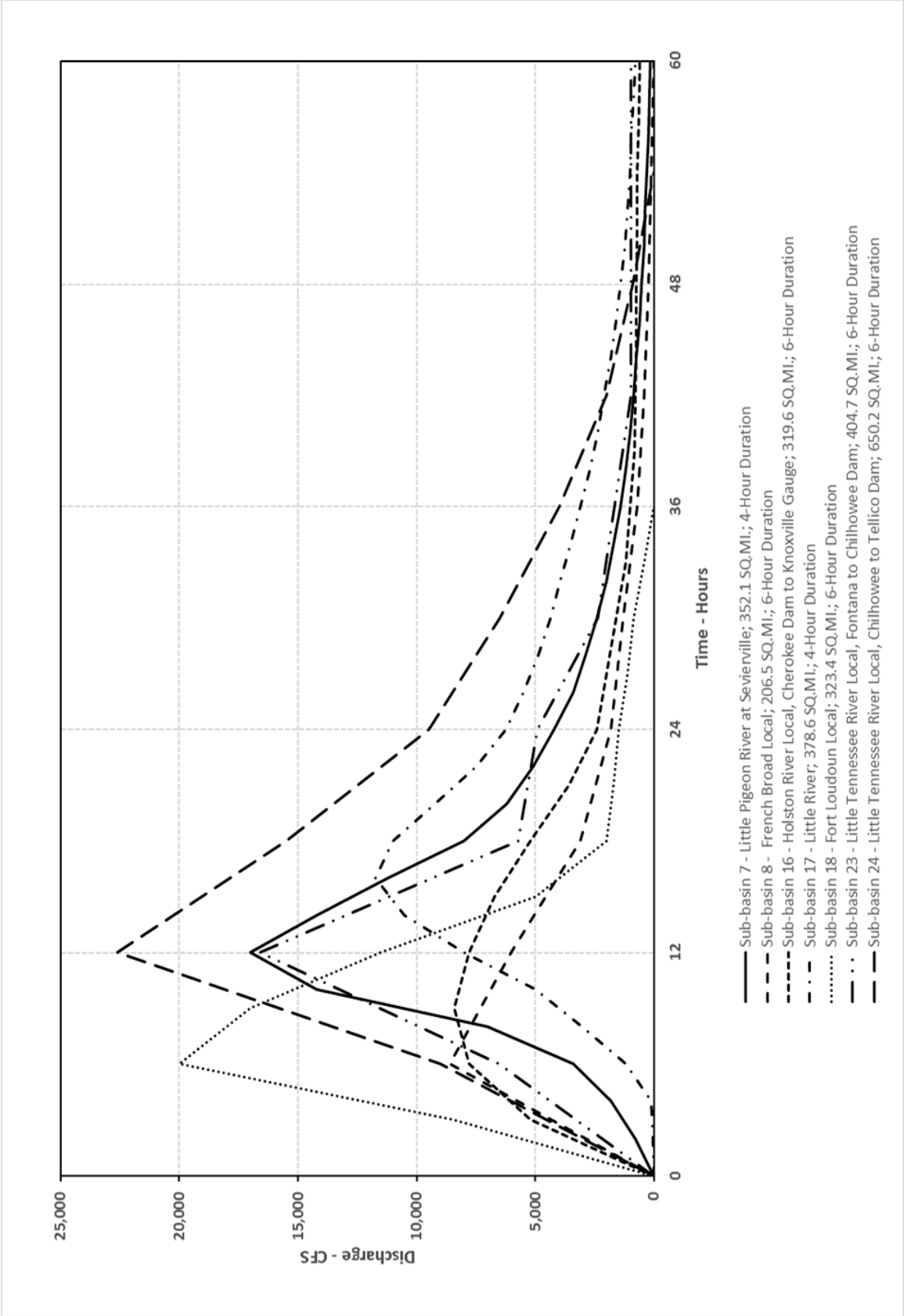
Figure 2.4.3-17. Unit Hydrographs, Sub-basins 7, 8, 16-18, & 23-24 (Sheet 5 of 8)

Sub-basin 7 - Little Pigeon River at Sevierville; 352.1 SQ.MI.; 4-Hour Duration

Sub-basin 8 - French Broad Local; 206.5 SQ.MI.; 6-Hour Duration

Sub-basin 16 - Holston River Local, Cherokee Dam to Knoxville Gauge; 319.6 SQ.MI.; 6-Hour Duration

Sub-basin 17 - Little River; 378.6 SQ.MI.; 4-Hour Duration

Sub-basin 18 - Fort Loudoun Local; 323.4 SQ.MI.; 6-Hour Duration

Sub-basin 23 - Little Tennessee River Local, Fontana to Chilhowee Dam; 404.7 SQ.MI.; 6-Hour Duration

Sub-basin 24 - Little Tennessee River Local, Chilhowee to Tellico Dam; 650.2 SQ.MI.; 6-Hour Duration

A3-33

Figure 2.4.3-17. Unit Hydrographs, Sub-basins 25, 33, 36, & 37 (Sheet 6 of 8)

Legend:
- Sub-basin 25 - Watts Bar Local Above Clinch River; 295.3 SQ.MI.; 6-Hour Duration
- Sub-basin 33 - Clinch River Local Above Mile 16; 37.2 SQ.MI.; 2-Hour Duration
- Sub-basin 36 - Clinch River Local, Mouth to Mile 16; 29.3 SQ.MI.; 2-Hour Duration
- Sub-basin 37 - Watts Bar Local Below Clinch River; 408.4 SQ.MI.; 6-Hour Duration

Axis labels: Discharge - CFS (vertical); Time - Hours (horizontal)

A3-34

Figure 2.4.3-17. Unit Hydrographs, Sub-basins 38-40, & 42 (Sheet 7 of 8)

Legend:
- Sub-basin 38 - Chatuge Dam; 189.1 SQ.MI.; 1-Hour Duration
- Sub-basin 39 - Nottely Dam; 214.3 SQ.MI.; 1-Hour Duration
- Sub-basin 40 - Hiwassee River Local; 565.1 SQ.MI.; 6-Hour Duration
- Sub-basin 42 - Blue Ridge Dam; 231.6 SQ.MI.; 2-Hour Duration

A3-35

Figure 2.4.3-17 Unit Hydrographs, Sub-basins 41, 43, 44A, 44B, & 45 (Sheet 8 of 8)

Legend:
— Sub-basin 41 - Apalachia Local; 49.8 SQ.MI.; 1-Hour Duration
······ Sub-basin 43 - Ocoee No. 1 Local; 362.6 SQ.MI.; 6-Hour Duration
- - - Sub-basin 44A - Hiwassee River From Charleston to Apalachia and Ocoee No. 1; 696.6 SQ.MI.; 6-Hour Duration
-··- Sub-basin 44B - Hiwassee River From Mouth to Charleston; 396.0 SQ.MI.; 6-Hour Duration
— — Sub-basin 45 - Chickamauga Local; 792.1 SQ.MI.; 6-Hour Duration
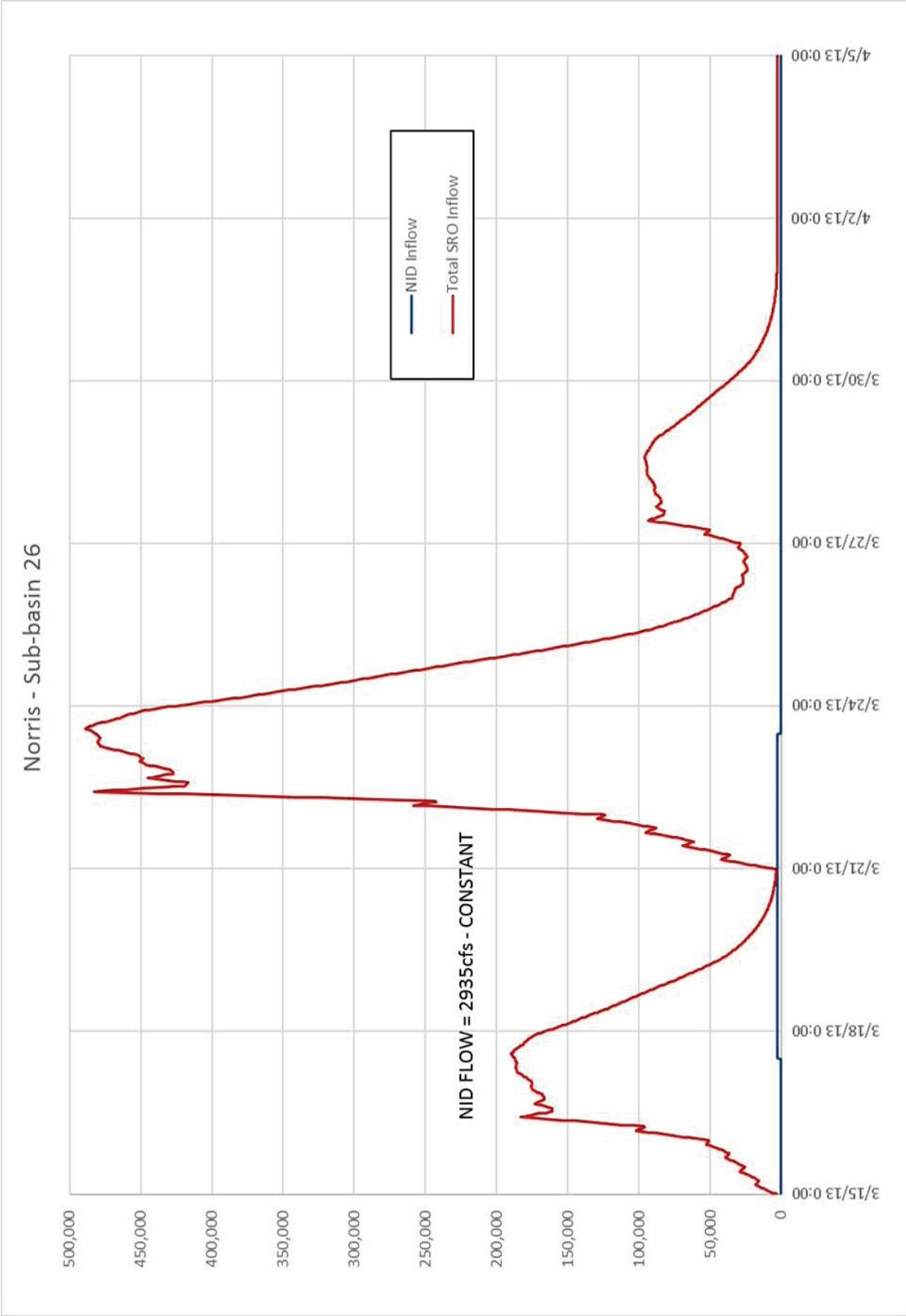
Figure 2.4.3-18. NID Inflow Hydrograph, Norris Reservoir
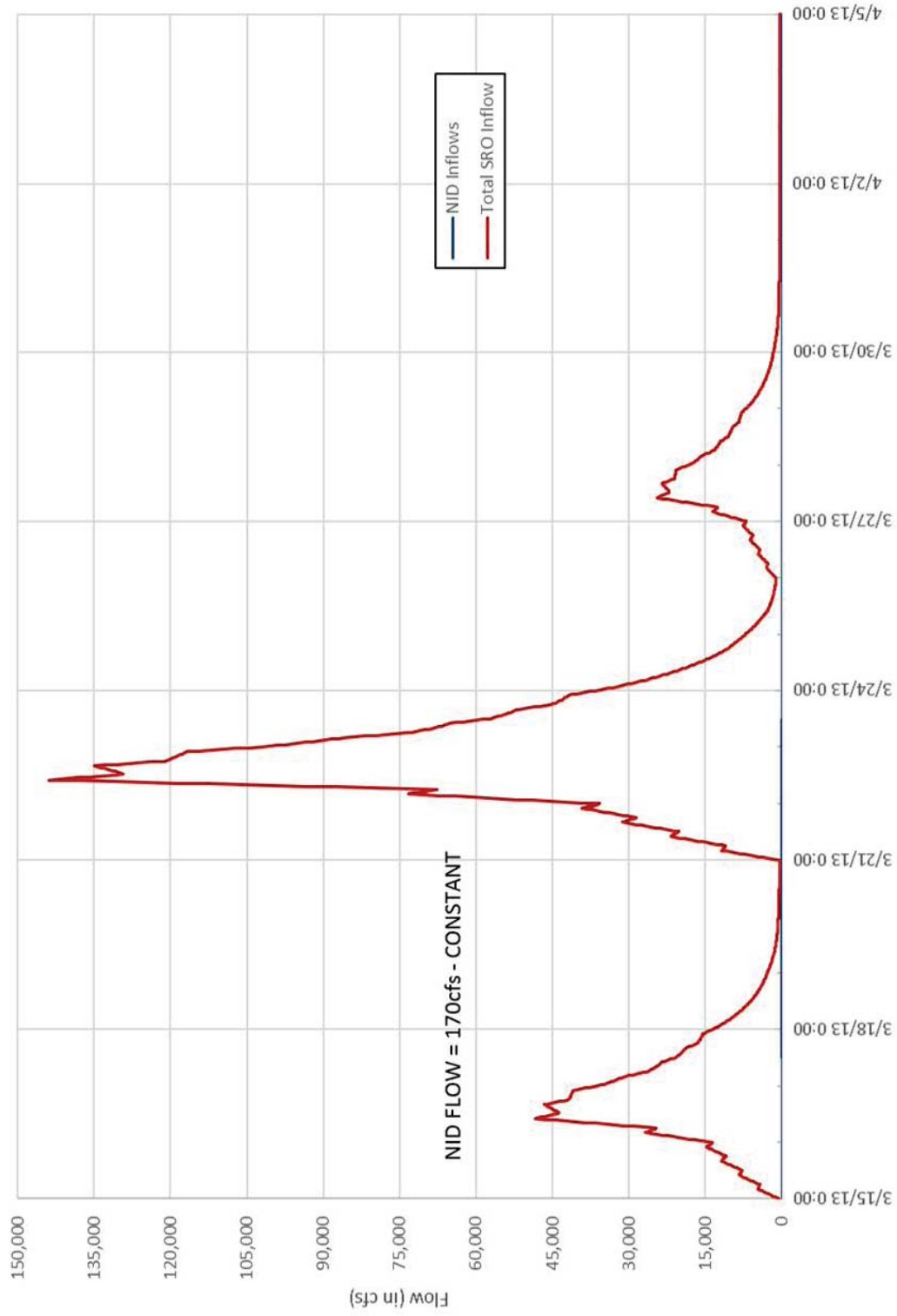
Figure 2.4.3-19. Melton Hill Reservoir
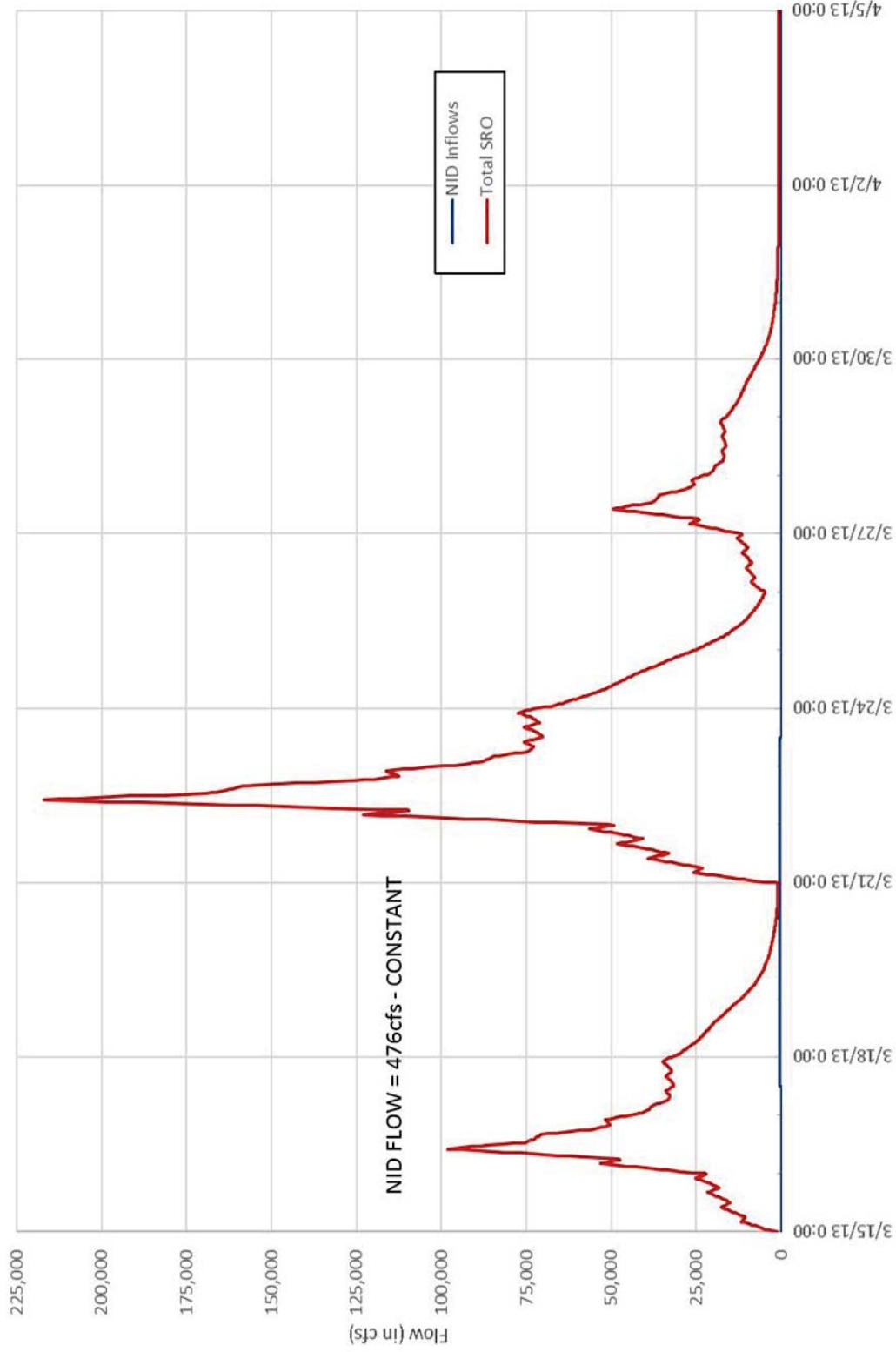
Figure 2.4.3-20. Watts Bar Reservoir
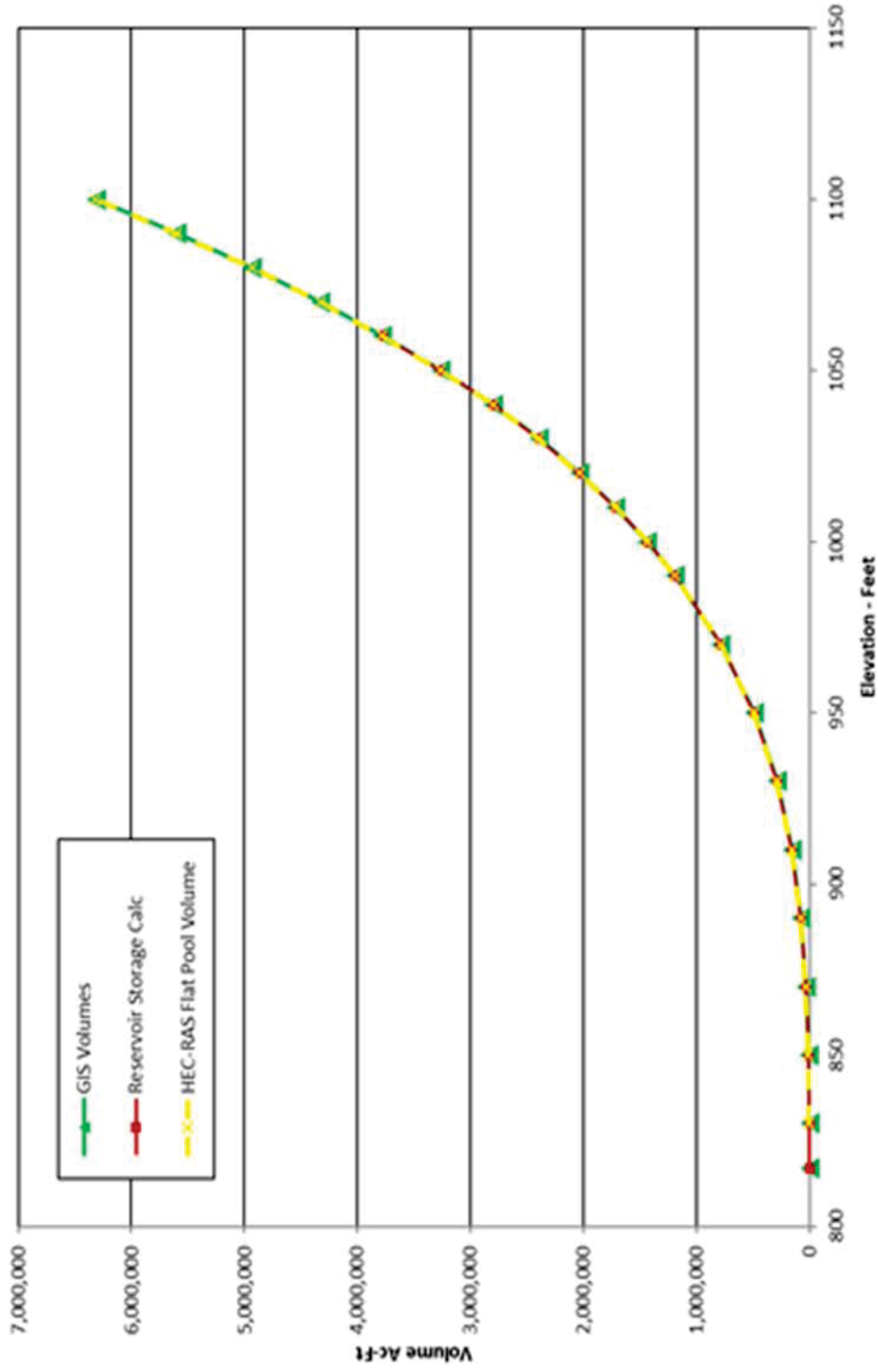
Figure 2.4.3-21. Highway 58 Bridge Profile

Figure 2.4.3-22. Norris Reservoir Volume versus Elevation
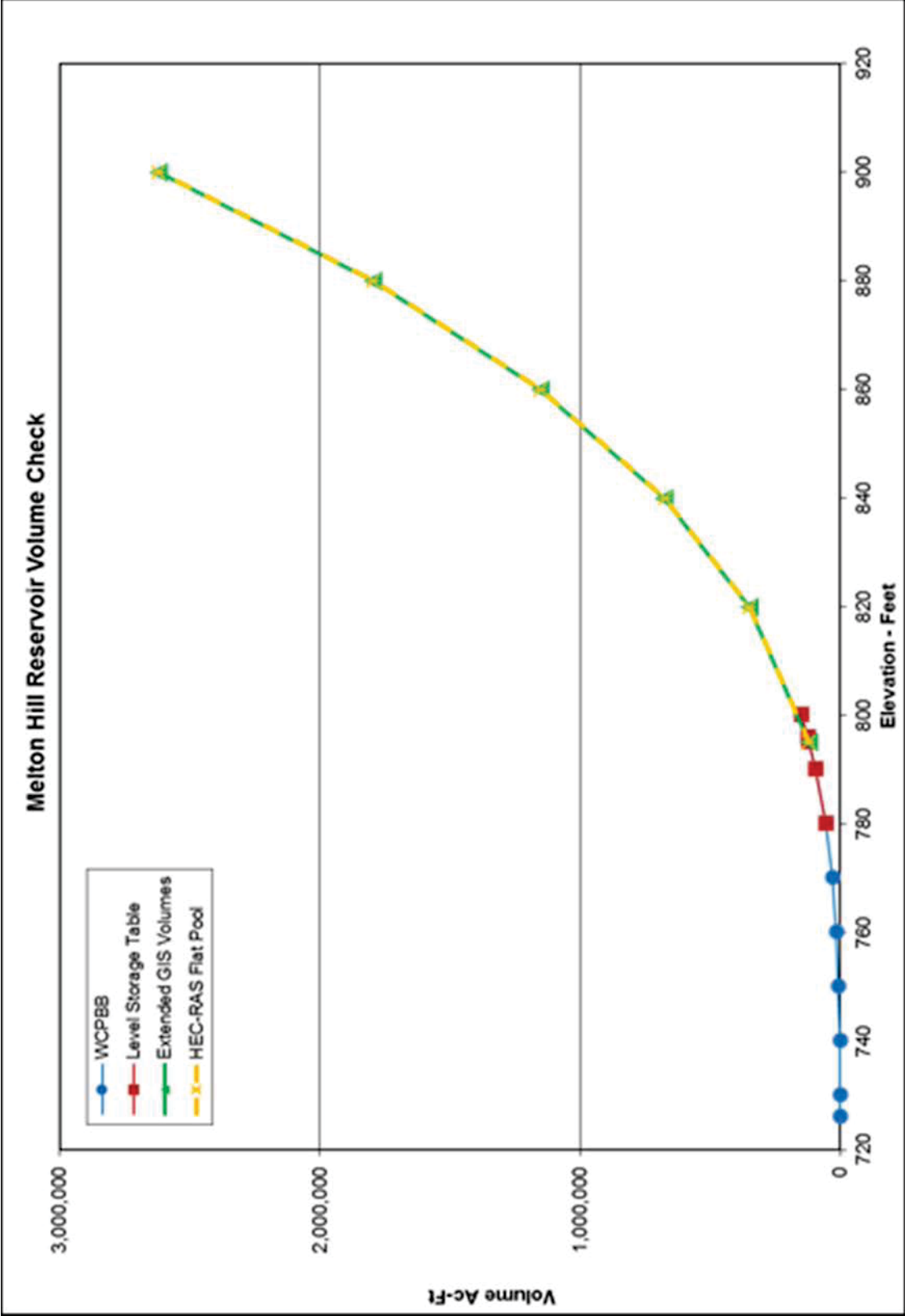
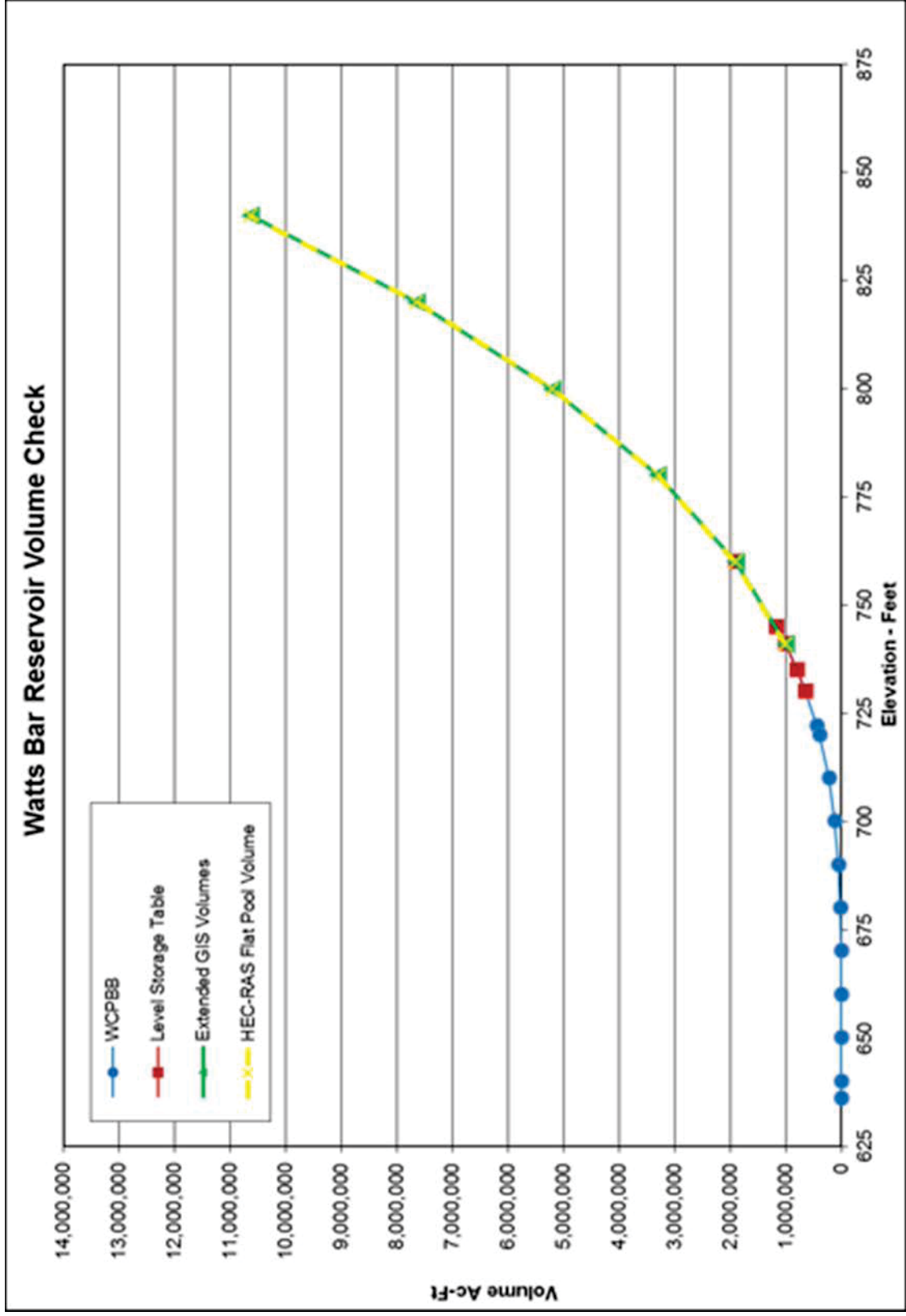**Figure 2.4.3-23. Melton Hill Reservoir Volume versus Elevation**

A3-42

Figure 2.4.3-24. Watts Bar Reservoir Volume versus Elevation

Attachment 4
Site Safety Analysis Report Subsection 2.4.4 Markups

SSAR Subsection 2.4.4 is being revised as indicated. Strikethroughs indicate text to be deleted. Underlines indicate text to be added.

2.4.4.2.1   Seismic Failure Analysis

Seismic Dam Failure Combination

(SRI/CEII)   The half-10,000-yr Douglas centered seismic event in combination with a 500-yr June flood event includes seismic failures of ██████████████████████████████████ Dams as well as seismic failures on downstream tributaries including ███████████████████ ████████ Dams. The 10,000-yr Fort Loudoun centered seismic event in combination with a 25-yr June flood event includes seismic failures of ██████████████████████████████ ████████ Dams as well as seismic failures on downstream tributaries including ████████████ █████████████████████████████████ were not analyzed for these seismic events and were assumed to fail in these combinations. It was assumed that ████████ Dam would not fail in order to maximize the water surface elevation upstream at the CRN Site. █████ Dam, upstream of the CRN Site, was evaluated for stability for the █████ Dam site-specific 10,000-yr and half-10,000-yr seismic events. █████ Dam was determined to be stable post-seismically at the normal maximum pool; therefore, a scenario that included a seismically induced failure of ████████ Dam was not warranted.

Flood Routing

~~The runoff model described in Subsection 2.4.3.4 was used to evaluate the potentially critical seismic events involving dam failures above the plant. Reservoir operating procedures used were those applicable to the season and flood inflows.~~

<u>Flood inflow hydrographs were developed by using watershed gaged data to scale prototypical inflow hydrographs to meet estimated 25- and 500-year volume targets.</u>

<u>Guidance for development of probabilistic point rainfall estimates is published in Reference 2.4.4-10. Reference 2.4.4-10, Section 5, indicates point rainfall estimate data represents rainfall frequency at a point approximately 0.5-miles square and is not directly applicable for larger areas. Reference 2.4.4-10 states that point estimates may be applied to larger areas after adjustment through the use of Areal Reduction Factors (ARFs) for areas up to 400 sq mi. Watersheds impacting the Clinch River Nuclear (CRN) Site are 17,310-sq mi above Watts Bar Dam and 3382-sq mi above the CRN Site. Because these areas are significantly beyond the published limits for ARFs, the application of ARF adjusted point rainfall based on Reference 2.4.4-10 was judged not suitable. Therefore, an alternate methodology for production of scaled inflow hydrographs was developed to meet the requirements. This methodology uses historical gaged data across the watershed above Watts Bar Dam aggregated into annual maximum series for 1- to 5-day durations to estimate 25- and 500-year frequency stream flows.</u>

TVA has maintained Estimated Local Flow (ELF) data at gaged points in the Tennessee River watershed since 1903. These data represent inflows at the referenced gage point and are independent of river regulation. The daily data from 1903 through 2013 were compiled into 'X-day values representing the corresponding durational flows (in cfs per 'X' days) for incremental daily durations of 1 to 5 days. The daily average for the 'X'-days were centered on each date for the odd durations and even durations were calculated using the average based on the leading center day. The series data were checked for conflict between same 'X'-day duration water years to identify and eliminate any overlapping events at the end of one water year and the beginning of the subsequent year. Conflicts were resolved by keeping the larger of the two series values and selecting the next highest non-overlapping annual value for the lower value water year. The 'X'-day data sets were arranged by water year (October 1 – September 30) and the annual maximum values for each duration for each water year were identified.

Following the guidance of Reference 2.4.4-8, an annual duration series (yearly 'X'-day maximum) was developed for each 'X'-day duration data set. A log-Pearson Type III distribution was applied to the resulting annual series following the methodologies described in References 2.4.4-8 and 2.4.4-10. Correction for data skew and elimination of low and high outliers were performed on the final distribution. A 10 percent significance level K value was used for the outlier check per guidance in Reference 2.4.4-8, Appendix 4. The resulting distributions provide both the 25- and the 500-year 'X'-day durational average streamflows. Because the resulting streamflows represent average flows over the respective duration, the estimates were used as streamflow volumes (i.e. durational streamflow x respective duration). The durational volumes above the Watts Bar project watershed were then selected as the target values for adjustment of the prototype inflow hydrographs.

The prototype inflow hydrographs are a representative storm event using published National Weather Service Atlas 14 data. A 25-year point rainfall at the centroid of the watershed above Chickamauga Dam was selected as the prototype rainfall for the watershed. A uniform rainfall areal distribution was applied over all sub-basins with a temporal distribution placing the peak rainfall according to a World Curve approach for a 24-hour event (Reference 2.4.4-9). Rainfall was applied with losses using the NRCS curve number methodology with validated curve numbers for the season, and baseflows applied were June average monthly values. Runoff transformation was accomplished by manual spreadsheet convolution using validated sub-basin unit hydrographs (UHs). Resulting inflow hydrograph data were multiplied by scaling factors applied to all sub-basins to achieve the target volumes for the 25-yr and 500-yr events at each daily duration from 1 to 5 days. Adjustment ratios at the maxima were varied iteratively to achieve an acceptable difference in volume between the targets and the final summed hydrographs for the 1- through 4-day values. The 5-day volume target was included in order to maintain an acceptable slope between the 4th and 5th day maxima which made the adjustment to meet the 4-day volume more reasonable. However, the 5-day maximum ratio tended to be very high since the applied rainfall was a 4- day event with losses. This 5-day ratio generated hydrograph ordinates that were considered to be artifacts. However, the volumes met the target values and were judged reasonable. Additionally, time steps more than 1 day after the 4-day peak ordinate applied a recession constant of 10 percent per day to the ratio values to smooth the falling limb and minimize ratio generated artifacts. The final hydrograph ordinates were summed and volumes calculated to confirm that the target volumes had been met or exceeded. The adjusted surface runoff values were limited to be no smaller than the constant baseflow.

During postulated single and multiple project failure events, the concurrent failure of National Inventory of Dams (NID) identified projects, as discussed in Subsection 2.4.3.4.1, outside the model is considered possible. The NID volumes are located across the sub-basins with conveyances having differing sinuosity, length, slope, cross-sectional and roughness characteristics. As a result, the postulated failure waves are expected to pass through a variety of supercritical, critical and subcritical flow regimes as they traverse the respective reaches, starting at the failure location and ending at the respective model input points. The resulting translation reduces the peak flows and spreads the time base of the volume input. A simplified calculation approach was used to account for the NID volumes under these failure conditions. A time to peak of 20 minutes was assumed for the failure hydrographs. A Froehlich approach was used to postulate the individual failure hydrograph peak flows. The individual hydrographs were then combined into a composite triangular hydrograph based on distance of the NID projects from the model, and the peaks were adjusted to preserve volume ensuring that the entire NID volume was included in the failure flows (Figures 2.4.4-5, 2.4.4-6, and 2.4.4-7).

The runoff model described in Subsection 2.4.3.4 was used to evaluate the potentially critical seismic events involving dam failures above the plant. Reservoir operating procedures used were those applicable to the season and flood inflows.

Based on a review of the flood elevations at the Watts Bar Dam in the half-10,000-yr seismic event compared to the 10,000-yr seismic event, the half-10,000-yr seismic event was determined to be controlling. The seismic dam failure combination producing the most critical elevation at the CRN Site is the half-10,000-yr Douglas centered seismic event during a 500-yr June flood event.

2.4.4.5   Coincident Wind Wave

(SRI/CEII) Wind waves to be associated with the PMF crest were computed using procedures of the USACE Coast Engineering Manual (Reference 2.4.4-6). Wind data from 2000 to the 2014 were collected at Huntsville, Alabama; Chattanooga, Knoxville, and Tri-Cities, Tennessee; and Asheville, North Carolina. The raw 2-minute average wind data were used to calculate the maximum 20-minute average wind speed for each year at each data collection site and the 2-yr wind speed was determined. The CRN Site overland wind speed of 28 mph was adjusted for overwater conditions resulting in an overwater wind speed of 33 mph. The effective fetch found for the CRN Site from available GIS terrain data was 4.25 mi, based on the site grade elevation of 821 feet North American Vertical Datum of 1988 (NAVD88), which results in the critical site fetch length (Figure 2.4.3-16). For a calculated 33 mph overwater 2-yr wind, the total wave height of ▮ ft from crest to trough was calculated, which includes wave runup ▮ ft) and wave setup (▮ ft). When applied to the maximum water surface elevation discussed in Subsection 2.4.4.4, coincident wind wave results in a maximum water surface elevation of ▮ ft NGVD29. CRN Site grade is 821 ft ~~North American Vertical Datum of 1988 (NAVD88)~~ (821.4 ft NGVD29), ▮ ft higher than the maximum calculated water surface elevation with wind wave height. Because of the available margin, the coincident wind wave activity does not have an effect on flooding at the site.

2.4.4.7   References

2.4.4-1.   NRC, "Guidance for Assessment of Flooding Hazards Due to Dam Failure, Interim Staff Guidance," Report JLD-ISG-2013-01, ADAMS Accession No. ML13151A153, Rev. 0, July 29, 2013.

2.4.4-2.   ANSI/ANS-2.8-1992, "Determining Design Basis Flooding at Power Reactor Sites," July 1992.

2.4.4-3.   TVA, Division of Water Management, Geological Services Branch, "Southern Appalachian Tectonic Study," Seay, William M., (ML082170494), Knoxville, Tennessee, January 1979.

2.4.4-4.   TVA, SRME - Design and Evaluation of New and Existing River System Dams, SRME-SPP-27.001.

2.4.4-5   Von Thun, J. Lawrence, and David R. Gillette, Guidance on Breach Parameters, internal memorandum, U.S. Bureau of Reclamation, March 13, 1990.

2.4.4-6.   U.S. Army Corps of Engineers, Coast Engineering Manual, EM1110-2-1100, Pts. 2 and 6.

2.4.4-7.   BWSC, Inc., Clinch River Nuclear Plant Sediment Transport Position Paper, Rev. 0, January 20, 2015.

2.4.4-8.   Guidelines for Determining Flood Flow Frequency, Bulletin #17B of the Hydrology Subcommittee, Interagency Advisory Committee on Water Data, Office of Water Data Coordination, Geological Survey, U.S. Department of the Interior, Revised September 1981 with March 1982 Editorial Corrections

2.4.4-9.   Moore, James N. and Ray C. Riley, "Comparison of Temporal Rainfall Distributions for Near Probable Maximum Precipitation Storm Events for Dam Design", National Water Management Center, Natural Resources Conservation Service (NRCS), Little Rock, Arkansas

2.4.4-10.   Hovey, Peter and Thomas DeFiore, "Using Modern Computing Tools to Fit the Pearson Type III Distribution to Aviation Loads Data", Report # DOT/FAA/AR-03/62, Office of Aviation Research, Federal Aviation Administration, U.S. Department of Transportation, Washington, D.C., September 2003.

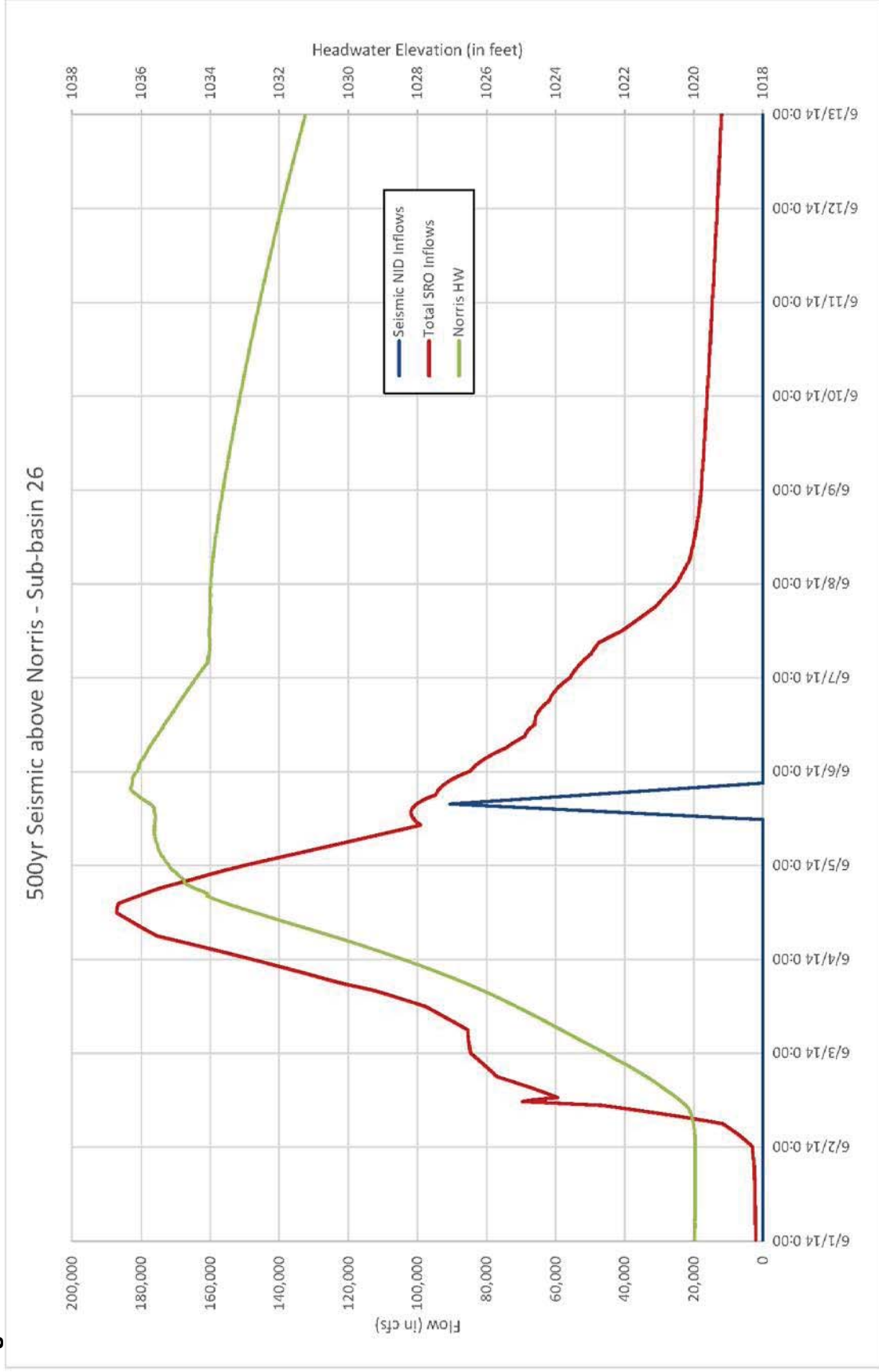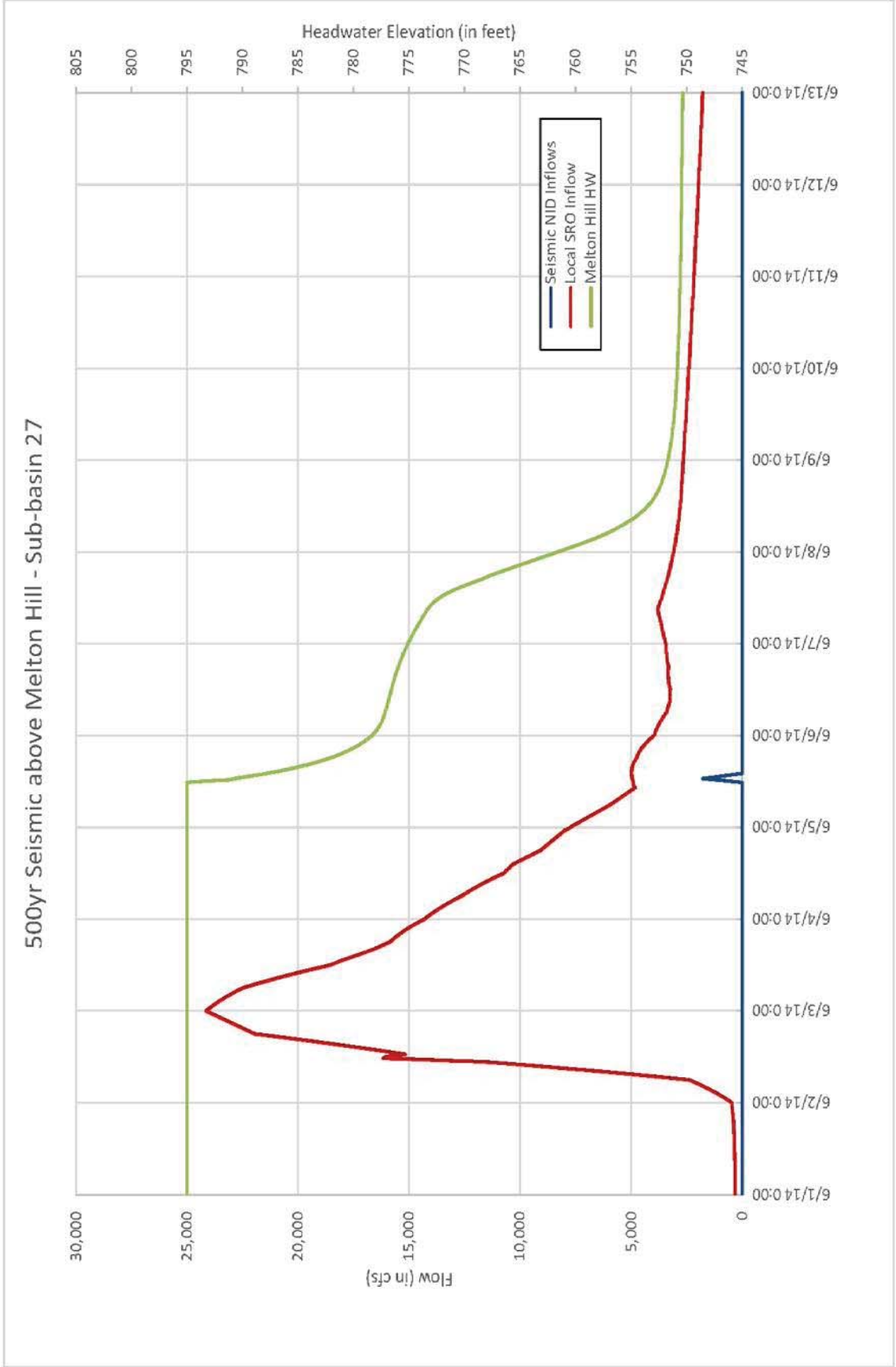**Figure 2.4.4-5. Seismic Inflow Hydrographs for 500-Yr June Flood Event – Norris Dam**

Figures 2.4.4-5 to 2.4.4-7 are added

Figure 2.4.4-6. Seismic Inflow Hydrographs for 500-Yr June Flood Event – Melton Hill Dam
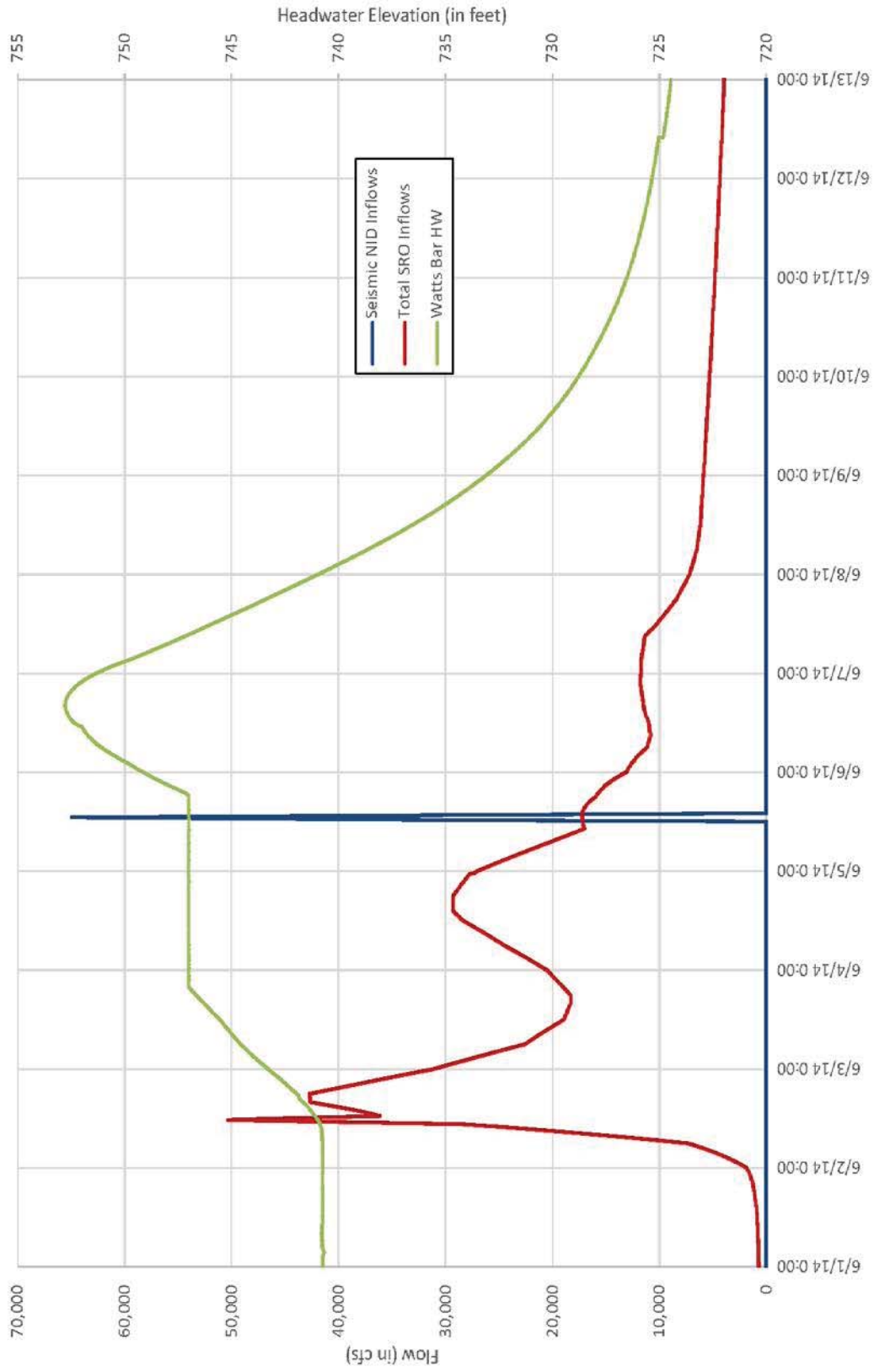
Figure 2.4.4-7. Seismic Inflow Hydrographs for 500-Yr June Flood Event – Watts Bar Dam

A4-7

Attachment 5
Site Safety Analysis Report Subsection 2.4.13 Markups

SSAR Section 2.4.13 is being revised as indicated. Strikethroughs indicate text to be deleted. Underlines indicate text to be added.

2.4.13.2   Receptors

NUREG-0800, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition, Section 2.4.13, and BTP 11-6 require the consideration of radiation exposure to members of the public at points beyond the site boundary where the Applicant has no administrative control. Radiation doses are then calculated based on various consumption pathways on an annual basis as defined in Regulatory Guide 1.109, Calculation of Annual Doses to Man from Routine Releases of Reactor Effluents for the Purpose of Evaluating Compliance with 10 CFR Part 50, Appendix I. The nearest site boundary beyond which the Tennessee Valley Authority (TVA) has no administrative control is the right bank (looking downstream) of the Clinch River arm of Watts Bar Reservoir (herein, referred to as the Reservoir).

The nearest surface water intake is the <u>Oak Ridge Bear Creek Plant</u> ~~City of Oak Ridge's West End Water Treatment plant (WTP)~~, located downstream of the CRN Site as shown on <u>Table</u> ~~Figure~~ 2.4.1-1 <u>and Figure 2.4.1-1</u> (Location No. 45). <u>This plant is also known as the City of Oak Ridge's West End Water Treatment plant (WTP) and the K-25 Water Treatment Plant. The Oak Ridge Bear Creek Plant ceased water production</u> ~~is WTP was idled~~ on September 30, 2014<u>, and Oak Ridge Utilities which now owns the facility has no plans to resume production at the site.</u> Further downstream of the CRN Site, the closest surface water intakes in the Watts Bar Reservoir are near Kingston, TN. The Kingston Fossil Plant uses the Emory River/Watts Bar Reservoir as a source of thermoelectric cooling water, and the Kingston Water System uses the Tennessee River/Watts Bar Reservoir for public drinking water supply.