SUBSECTION 2.4.3 TABLE OF CONTENTS

SUBSECTION 2.4.3 LIST OF TABLES

SUBSECTION 2.4.3 LIST OF FIGURES

2.4.3 Probable Maximum Flood (PMF) on Streams and Rivers

The design of nuclear power plants includes protection from the adverse effects of flooding. To assist in determining the potential for adverse flooding effects, the U.S. Nuclear Regulatory Commission (NRC) provides guidance for estimating design basis floods in Regulatory Guide 1.59, *Design Basis Floods for Nuclear Power Plants*, and NUREG/CR-7046, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America*. Either one or an appropriate combination of several hydrometeorological, geoseimic, or structural-failure phenomena causes a design basis flood which results in a hazard to structures, systems, and components (SSCs) important to the safety of a nuclear power plant. Due to the watershed size and operation of existing flood control structures on the Clinch and Tennessee River systems, the controlling event must be determined from several candidate PMP events. The events analyzed are summarized in Table 2.4.3-1.

Determination of design basis flood levels includes considering the most severe flood conditions that may be reasonably predicted to occur at a site as a result of severe hydrometeorological conditions. The National Weather Service (NWS) in Hydro-Meteorological Report 41 (HMR-41—Reference 2.4.3-1), Hydro-Meteorological Report 51 (HMR-51—Reference 2.4.3-2), Hydro-Meteorological Report 52 (HMR-52—Reference 2.4.3-3), and Hydro-Meteorological Report 56 (HMR-56—Reference 2.4.3-4) have defined, for TVA, PMP events for the Tennessee Valley. The storms defined by the NWS references provided either specific spatial rainfall patterns that reflected orographic effects or idealized elliptical isohyetal patterns with preferred orientation and orographic multipliers.

2.4.3.1 Watershed Characteristics

In order to accomplish their Federally mandated, integrated operation of the TVA reservoir system, TVA has developed runoff and stream course hydrologic models of the Tennessee River watershed including the watershed above the CRN Site. These models are used in design basis flood level analysis for sites in the Tennessee River System above Wilson Dam. The 30,747 sq mi watershed above Wilson Dam has been divided into 65 smaller sub-basin areas based on topography and gage locations. Sub-basins above Wilson Dam are depicted in Figure 2.4.3-1. Sub-basin areas are included in Table 2.4.3-2. The sub-basin hydrological models require PMP rainfall data estimations as inputs to calculate model inflows.

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 approximately in the middle of the 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 17.02 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.

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-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:

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

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 and 1973, 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. Validated unit hydrographs, shown in Figure 2.4.3-17, 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 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, 2.4.3-19, and 2.4.3-20). 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 Geometry Development and Calibration

Main stem reservoir elevation-storage relationships (Figure 2.4.4-1) 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, 2.4.3-23, and 2.4.3-24). 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 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 elevation-storage relationships shown in Figure 2.4.4-1.

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, 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.

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 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 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 2.4.3-6. 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 [redacted] 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, 2.4.3-8, and 2.4.3-9). [redacted] Dam remained stable and [redacted] 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 [redacted] Dam (all three gates remained closed) and [redacted] Dam (20 percent reduction in available gates). This simulation resulted in overtopping failures of [redacted] and [redacted] Dams which produced an increase in elevation at the CRN Site of [redacted] ft above the elevation produced by a PMF simulation without failures of [redacted] and [redacted] Dams. However, the increased elevation remains [redacted] ft below the bounding design basis flood elevation of [redacted] ft. Additionally, the possibility of all gates at Norris Dam being inoperable is not realistic because:

- 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 [redacted] 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 [redacted] ft National Geodetic Vertical Datum of 1929 (NGVD29) at the CRN Site. The bounding design basis elevation is established as [redacted] 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 [redacted] ft from crest to trough was calculated, which includes wave runup [redacted] ft) and wave setup [redacted] ft), resulting in a maximum elevation of [redacted] ft NGVD29. CRN Site grade is 821.4 ft NGVD29 (821 ft North American Vertical Datum of 1988

[NAVD88]), [redacted] 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 Melton Hill and Norris Dams concurrent with the PMF crests at these dams. As discussed previously, [redacted] earth embankments are assumed to be overtopped and to fail in the PMF. Adequate freeboard is available for the Norris 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 Mi^2 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).

Table 2.4.3-1
Flood Events Analyzed

(SRI/CEII)

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	[redacted]	19.20	7.89	[redacted]	543,000	18.10
3382 sq mi, centered above CRN, June storm event	[redacted]	19.40	8.57	[redacted]	544,000	16.42
469 sq mi, centered between CRN and Norris Dam, June storm event	[redacted]	8.73	3.72	[redacted]	200,000	10.55
7980 sq mi, Bull's Gap centered, March PMF with 100% runoff and peaked/lagged unit hydrographs storm event	[redacted]	16.68	17.02	[redacted] (a)	536,000	7.79
Half-10,000-Yr Douglas Centered Seismic Event During A 500–Yr June Flood Event	[redacted]	NA (b)	NA (b)	[redacted]	162,000	6.80
[redacted] Dam Sunny Day Failure	[redacted]	NA (c)	NA (c)	[redacted]	579,000 (a)	18.11

Notes:

CRN = Clinch River Nuclear

PMF = Probable Maximum Flood

(a) 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.

(b) 500-yr flow based on estimate using gaged inflows.

(c) Sunny day flows based on project storage only.

Table 2.4.3-2 (Sheet 1 of 2)
Sub-Basins, Areas and Rainfall Depth

Sub-Basin Label	Sub-Basin Name	Area (sq mi)	72 hr Rainfall Depth (inches)
1	French Broad River at Asheville	944.4	10.90
2	French Broad River, Newport to Asheville	913.1	16.47
3	Pigeon River at Newport	667.1	15.50
4	Nolichucky River at Embreeville	804.9	15.47
5	Nolichucky local, Embreeville to Nolichucky Dam	378.7	21.13
6	Douglas Dam local	835.0	26.68
7	Little Pigeon River at Sevierville	352.1	20.16
8	French Broad River local	206.5	23.98
9	South Holston Dam	703.3	16.83
10	Watauga Dam	468.2	16.17
11	Boone local	667.7	19.57
12	Fort Patrick Henry	62.8	23.32
13	North Fork Holston River near Gate City	668.9	17.55
14-15	Total Cherokee	854.6	24.31
16	Holston River local, Cherokee Dam to Knoxville gage	319.6	21.60
17	Little River at mouth	378.6	20.05
18	Fort Loudoun local	323.4	20.03
19	Little Tennessee River at Needmore	436.5	11.60
20	Nantahala	90.9	11.76
21	Tuckasegee River at Bryson City	653.8	13.47
22	Fontana local	389.8	14.75
23	Little Tennessee River local, Fontana Dam to Chilhowee Dam	404.7	15.33
24	Little Tennessee River local, Chilhowee Dam to Tellico Dam	650.2	15.92
25	Watts Bar local above Clinch River	295.3	15.85
26	Clinch River at Norris Dam	2912.8	16.48
27	Melton Hill local	431.9	18.02
28	Not Used		
29			
30			
31			
32			
33	Clinch River local above Mile 16	37.2	16.62
34	Poplar Creek at mouth	135.2	16.16
35	Emory River at mouth	868.8	12.25
36	Clinch River local, mouth to Mile 16	29.3	15.58
37	Watts Bar local below Clinch River	408.4	13.10
38	Chatuge Dam	189.1	10.61

Table 2.4.3-2 (Sheet 2 of 2)
Sub-Basins, Areas and Rainfall Depth

Sub-Basin Label	Sub-Basin Name	Area (sq mi)	72 hr Rainfall Depth (inches)
39	Nottely Dam	214.3	10.25
40	Hiwassee River local below Chatuge and Nottely	565.1	12.13
41	Apalachia local	49.8	12.47
42	Blue Ridge Dam	231.6	9.45
43	Ocoee No. 1 local, Ocoee No. 1 to Blue Ridge Dam	362.6	11.12
44A	Hiwassee River local, Charleston gage to Apalachia and Ocoee No. 1 Dams	686.6	12.83
44B	Hiwassee River local, mouth to Charleston gage at Mile 18.9	396.0	12.02
45	Chickamauga local	792.1	11.44
46	South Chickamauga Creek near Chattanooga	428.1	8.55
47A	Nickajack local below North Chickamauga Creek @ gage	545.7	8.24
47B	North Chickamauga Creek @ gage	98.3	9.54
48	Sequatchie River at Whitwell	400.0	9.84
49	Guntersville North local	1044.1	7.20
50	Guntersville South local	1154.9	5.71
51	Paint Rock Creek near Woodville	321.0	5.98
52	Paint Rock local	138.1	5.41
53	Flint River near Chase	343.0	5.19
54	Flint River local	224.9	5.30
55	Cotaco Creek at Florette	136.2	4.42
56	Cotaco Creek local	101.1	4.33
57	Limestone Creek near Athens	121.3	4.47
58	Limestone Creek local	157.4	4.22
59	Tims Ford Dam	533.3	6.62
60	Elk River Local, Tims Ford to Fayetteville	293.4	5.63
61	Elk River Local, Fayetteville to Prospect	490.2	4.57
62	Richland Creek at mouth	488.0	3.56
63	Sugar Creek at mouth	177.0	3.15
64	Elk River Local, Mile 16.5 to Prospect Gage	145.1	3.66
65	Wheeler local	1476.8	3.87
66	Big Nance Creek at mouth	197.1	2.95
67	Shoal Creek at Iron City Gage	347.7	2.25
68	Shoal Creek local	145.0	2.04
69	Wilson local	459.0	2.53

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 2nd Day	0.69	Table 7-2, HMR-41
7	156	23% of 2nd Day	0.80	Table 7-2, HMR-41
	162	27% of 2nd Day	0.94	Table 7-2, HMR-41
	168	30% of 2nd Day	1.04	Table 7-2, HMR-41
	174	2nd 6-hr	2.80	Table 7-2, HMR-41
8	180	1st 6-hr	5.39	Table 7-2, HMR-41
	186	3rd 6-hr	1.94	Table 7-2, HMR-41
	192	4th 6-hr	1.39	Table 7-2, HMR-41
	198	28% of 3rd Day	0.57	Table 7-2, HMR-41
9	204	26% of 3rd Day	0.53	Table 7-2, HMR-41
	210	23% of 3rd Day	0.47	Table 7-2, HMR-41
	216	23% of 3rd Day	0.47	Table 7-2, HMR-41

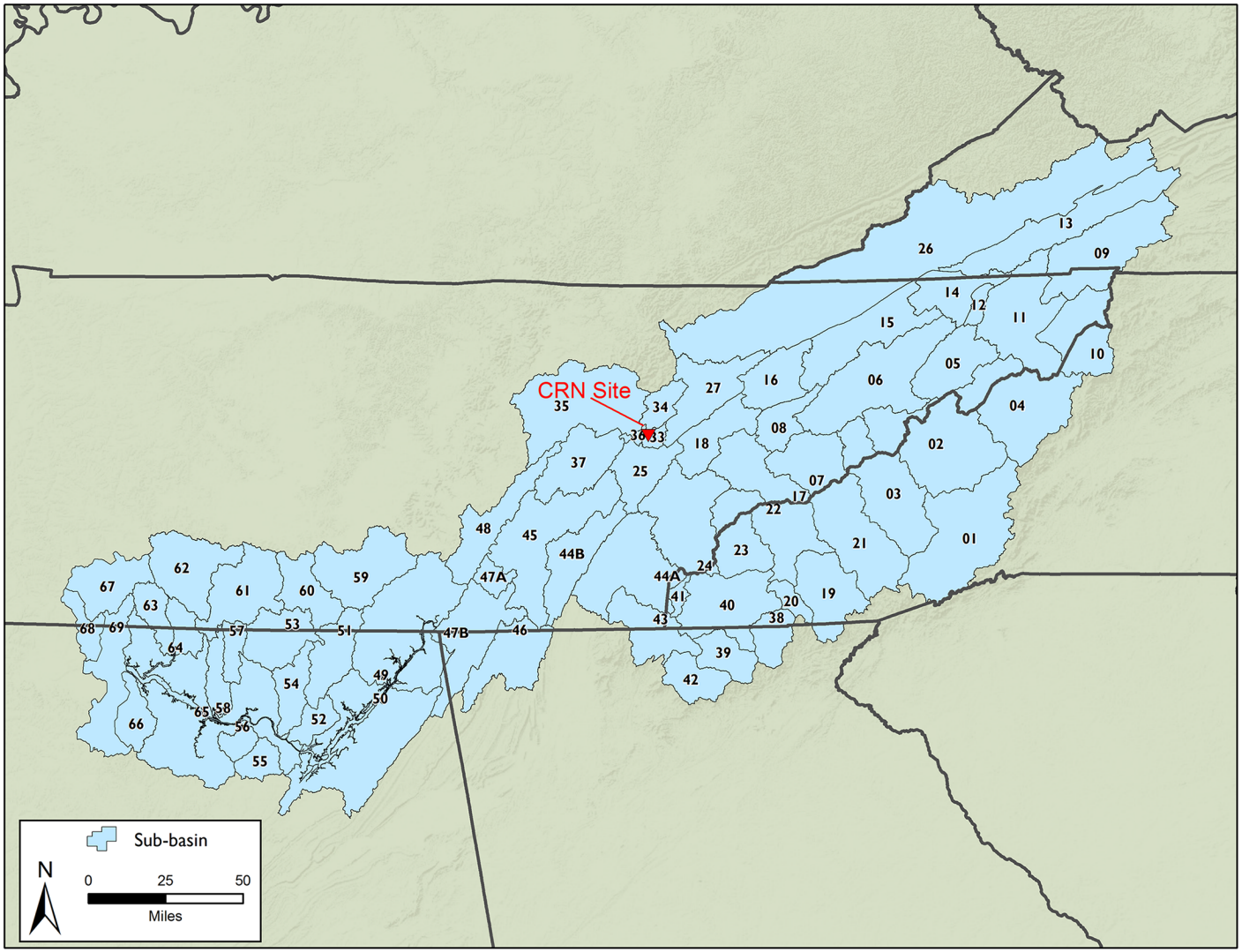


Figure 2.4.3-1. Tennessee River System Watershed Sub-Basins

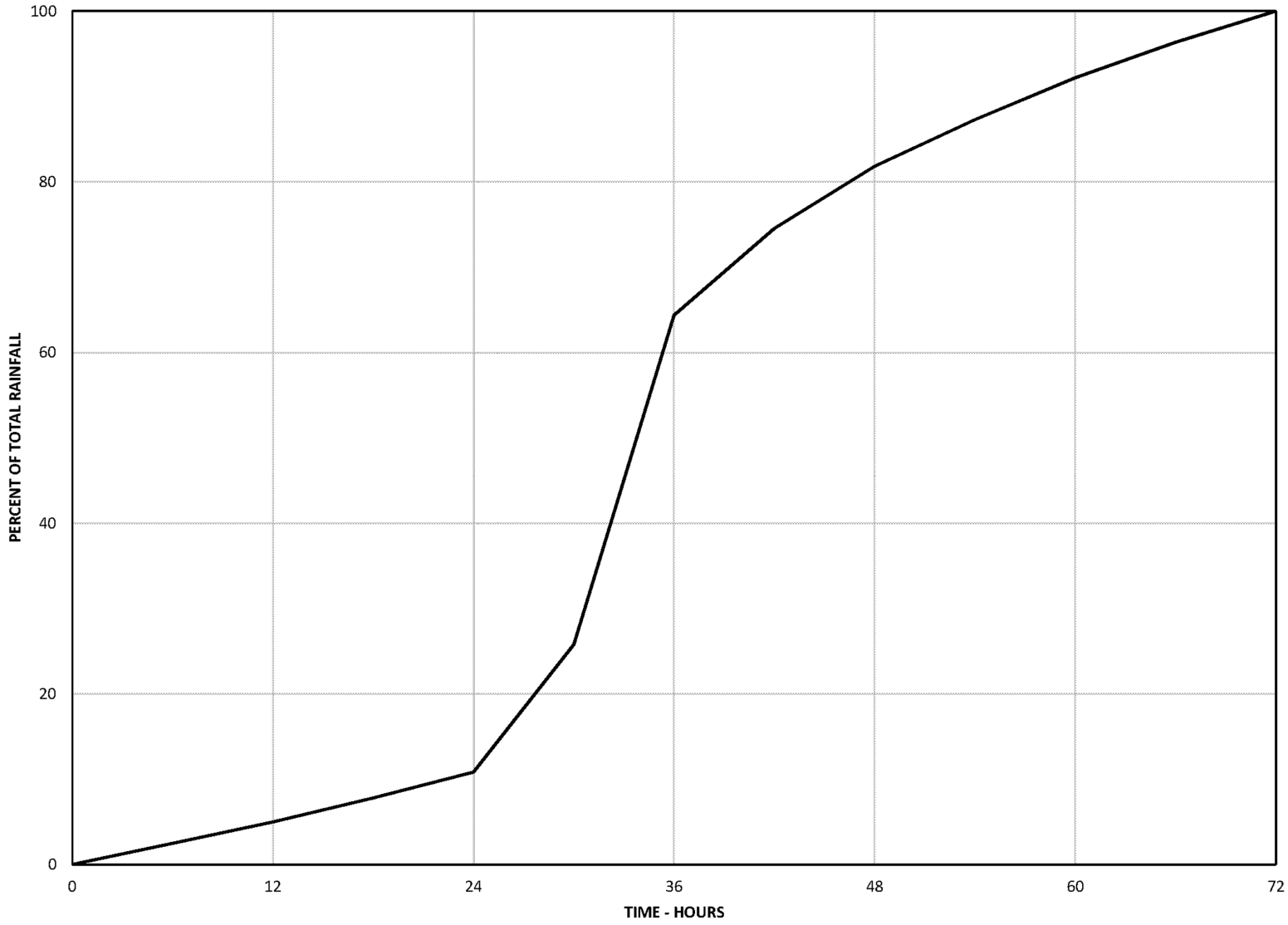
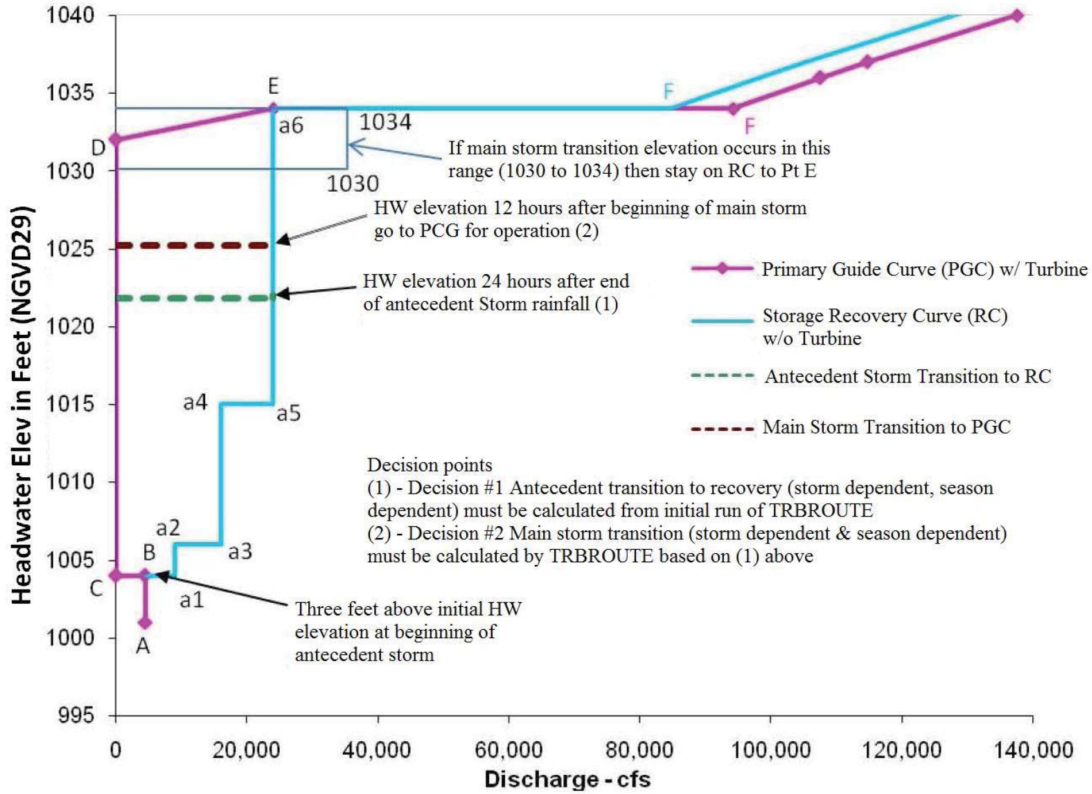


Figure 2.4.3-2. Rainfall Time Distribution – Typical Mass Curve

(SRI/CEII)

[redacted]

Figure 2.4.3-3. PMF Elevation and Discharge Hydrograph at Clinch River Nuclear Site



Notes:

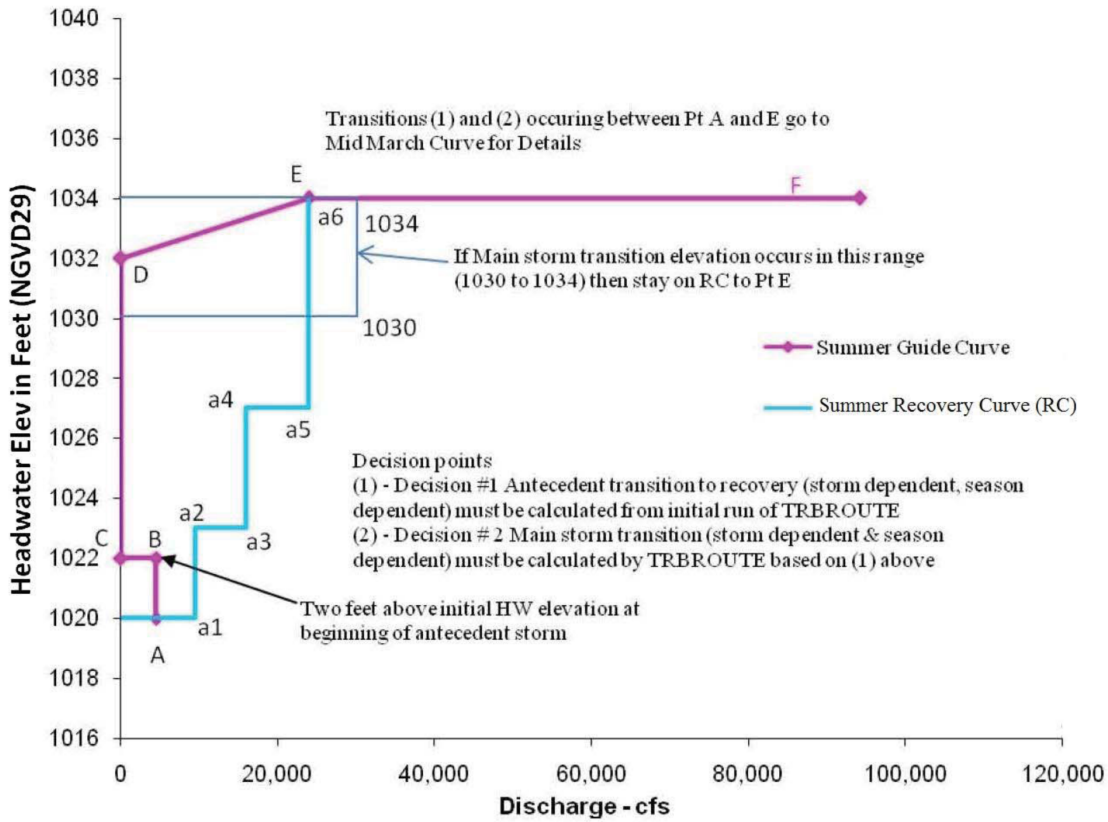
1. All transition points (dashed lines) are shown for illustration purpose only. Transitions are storm dependent.
2. The possibility exists that the antecedent storm exceeds point E elevation prior to transition.

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

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

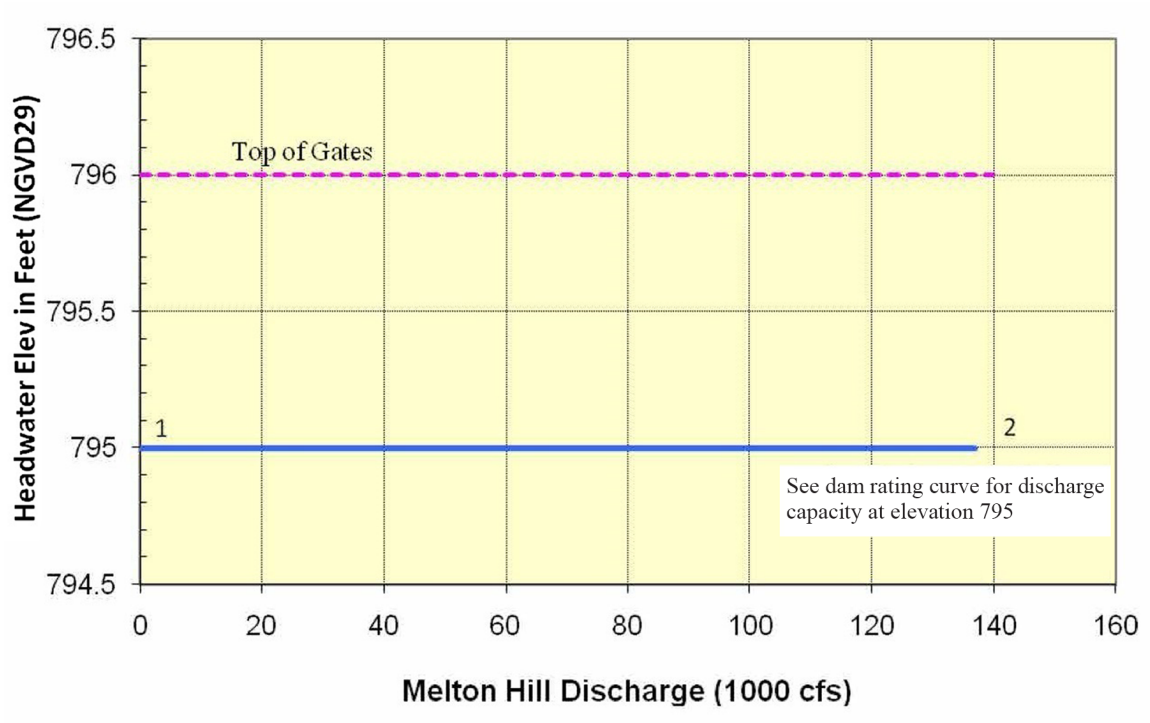


June 1

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. (Sheet 2 of 2) Flood Operational Guide – Norris Dam



Mid-March and June 1 Guide		
Pt	1	2
Elevation (Ft)	795	795
Discharge (1000 cfs)	0	*

*See dam rating curve for discharge capacity at elevation 795

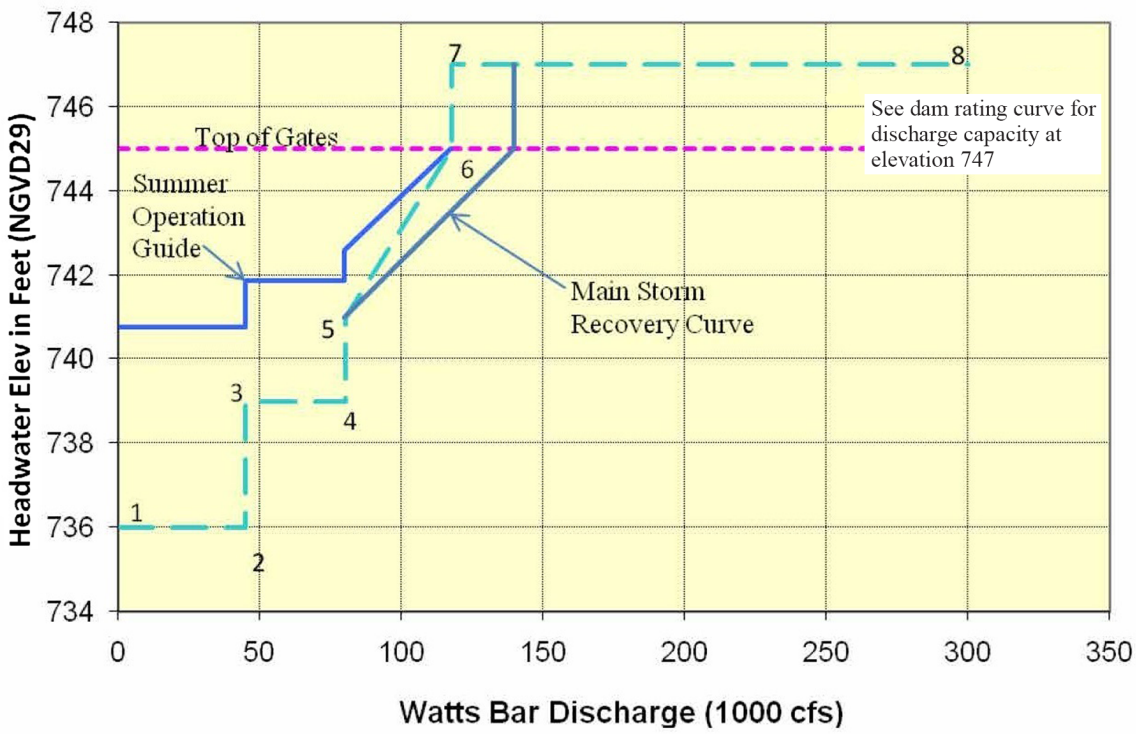
Notes:

Melton Hill Flood Operational Guide

The sequence for Melton Hill will follow the Flood Guide shown on this figure. As a flood develops the operation at Melon Hill will follow the numbers shown on this figure as defined below:

- Hold elevation 795 until Melton Hill discharge capacity is reached.

Figure 2.4.3-5. Flood Operational Guide – Melton Hill Dam



Mid-March Guide								
Pt	1	2	3	4	5	6	7	8
Elevation (Ft)	736	736	739	739	741	745	747	747
Discharge (1000 cfs)	0	45	45	80	80	117.5	117.5	*

*See dam rating curve for discharge capacity at elevation 795

June 1 Guide								
Pt	1	2	3	4	5	6	7	8
Elevation (Ft)	740.75	740.75	741.85	741.85	742.58	745	747	747
Discharge (1000 cfs)	0	45	45	80	80	117.5	117.5	*

Figure 2.4.3-6. Flood Operational Guide – Watts Bar Dam

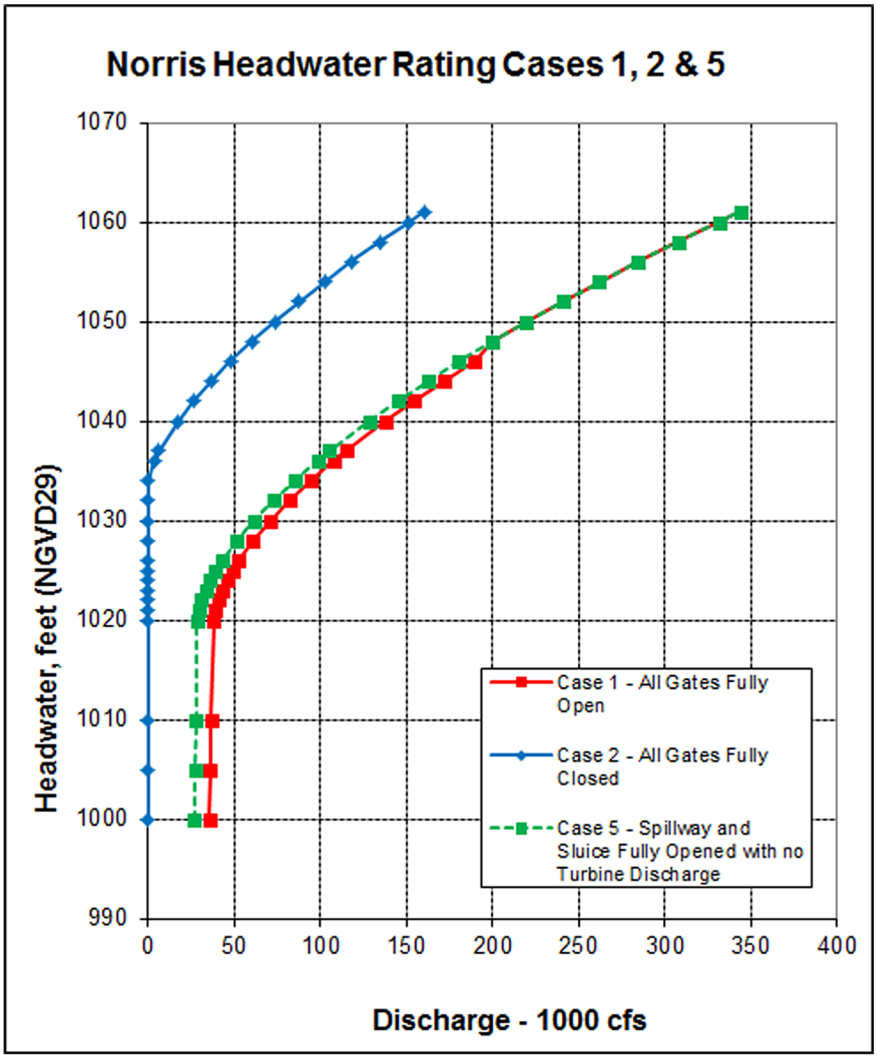


Figure 2.4.3-7. Dam Rating Curve – Norris Dam

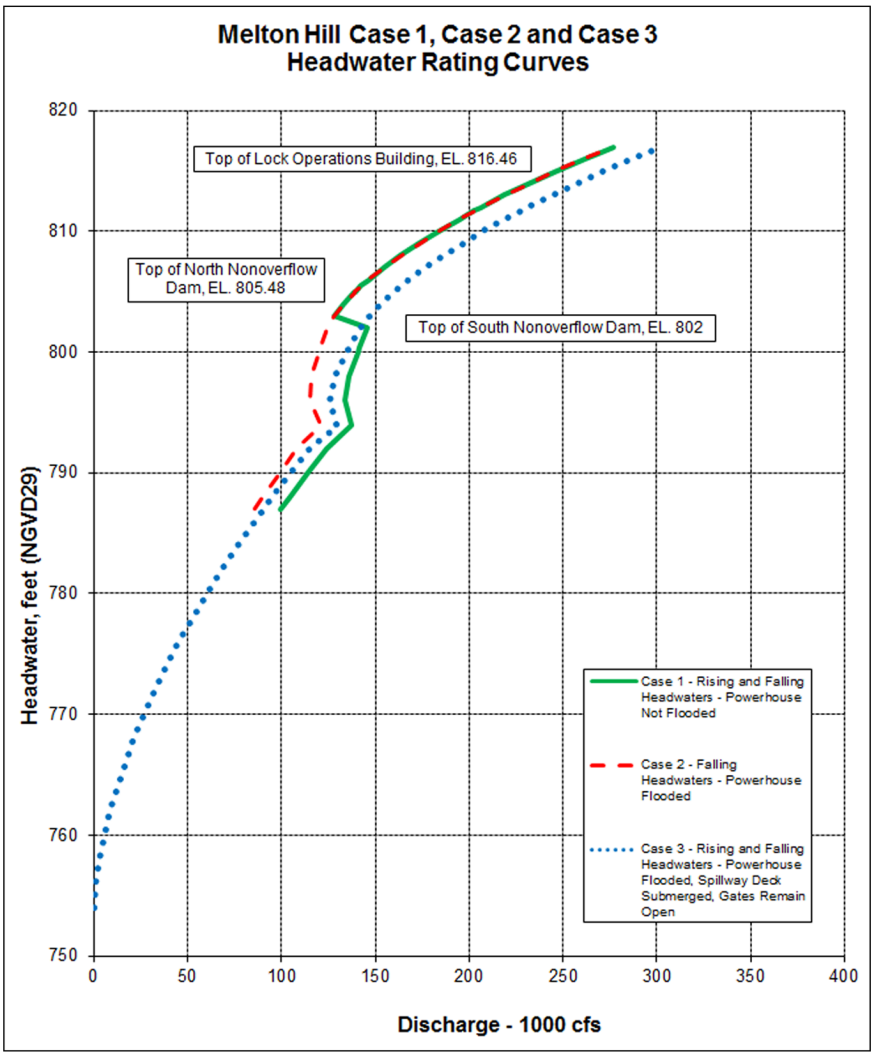


Figure 2.4.3-8. Dam Rating Curve – Melton Hill Dam

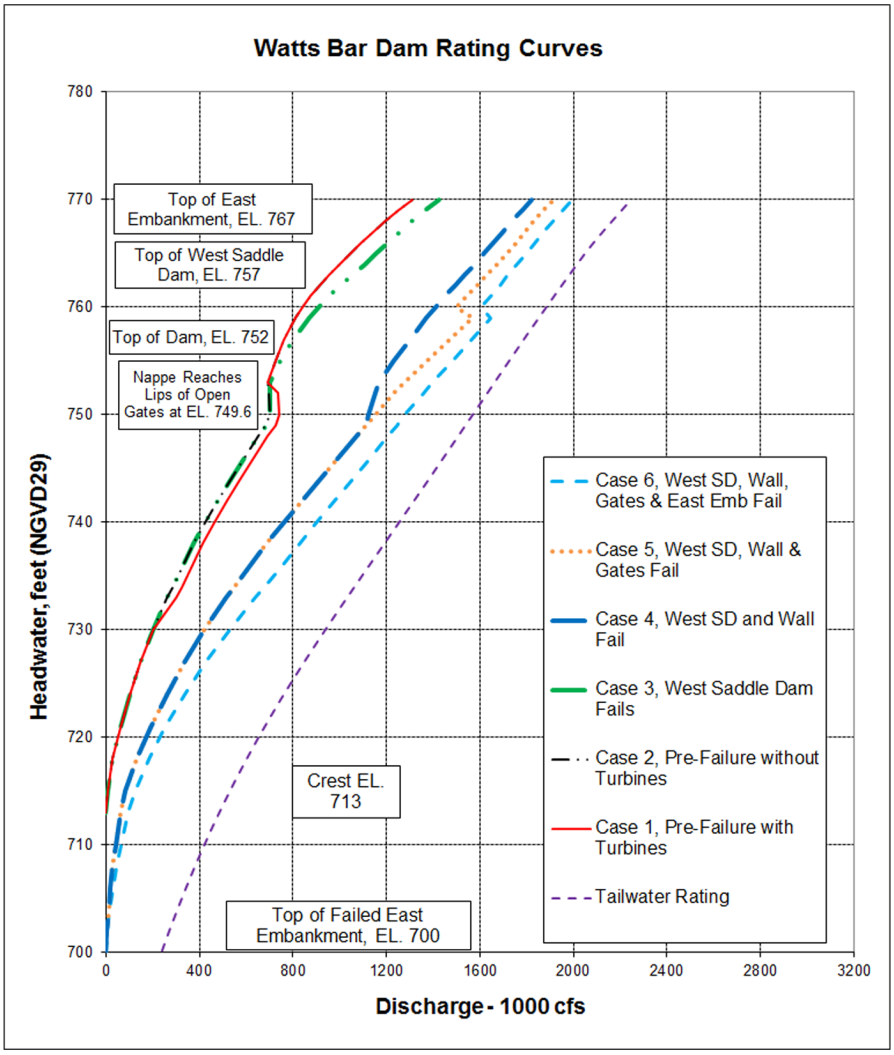


Figure 2.4.3-9. Dam Rating Curve – Watts Bar Dam

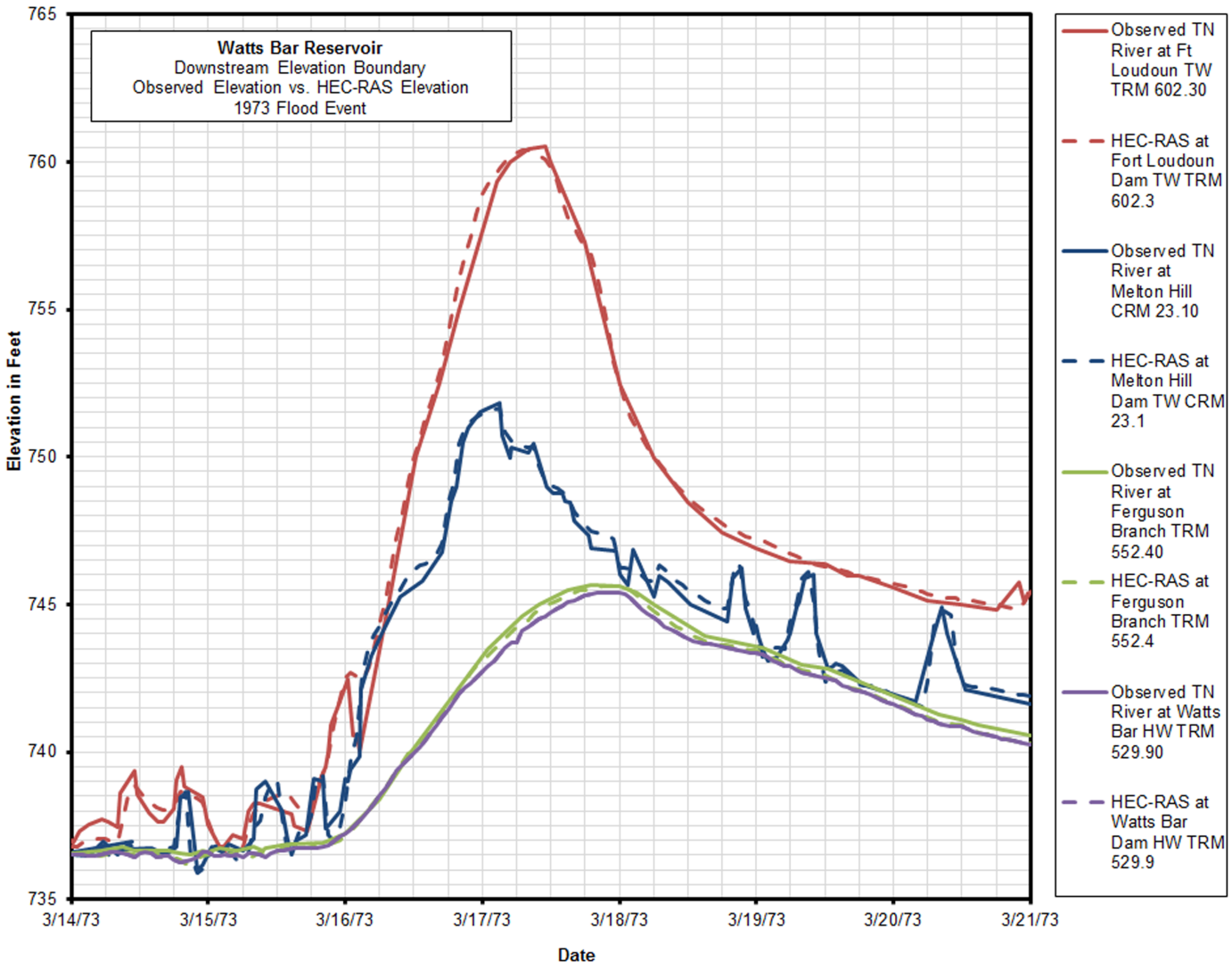


Figure 2.4.3-10. Watts Bar Reservoir Calibration Results – Elevation: 1973 Flood Event

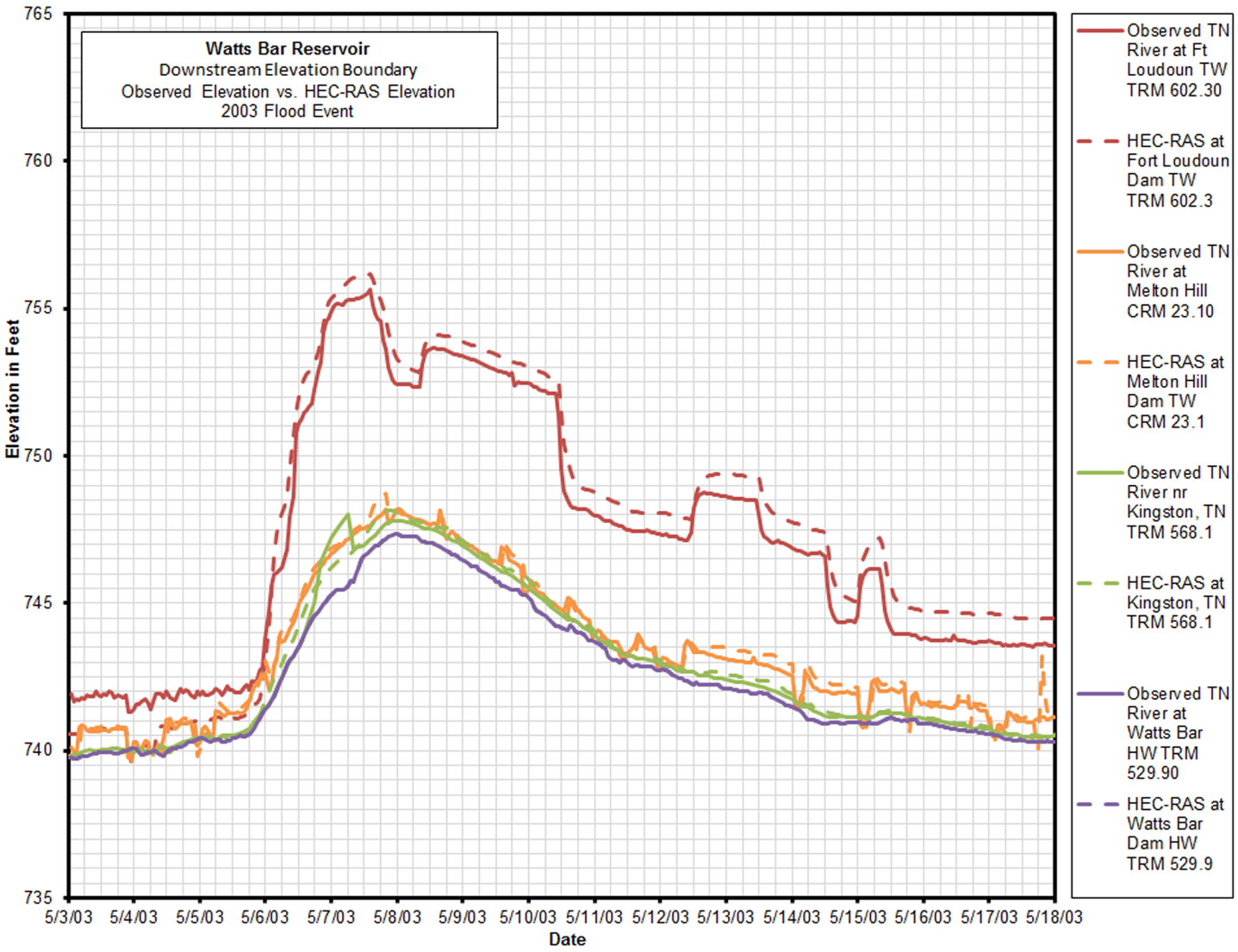


Figure 2.4.3-11. Watts Bar Reservoir Calibration Results – Elevation: 2003 Flood Event

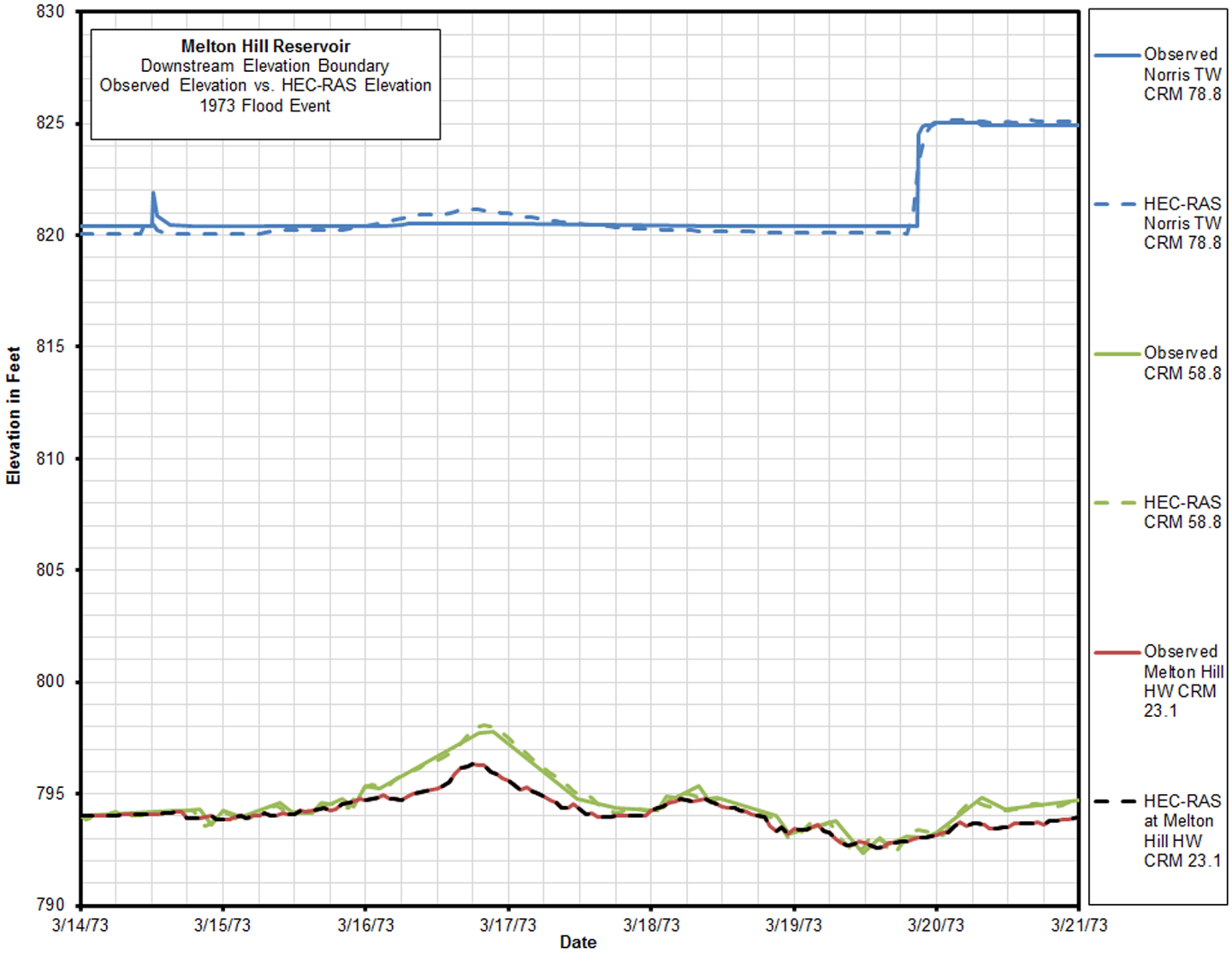


Figure 2.4.3-12. Melton Hill Reservoir Calibration Results – Elevation: 1973 Flood Event

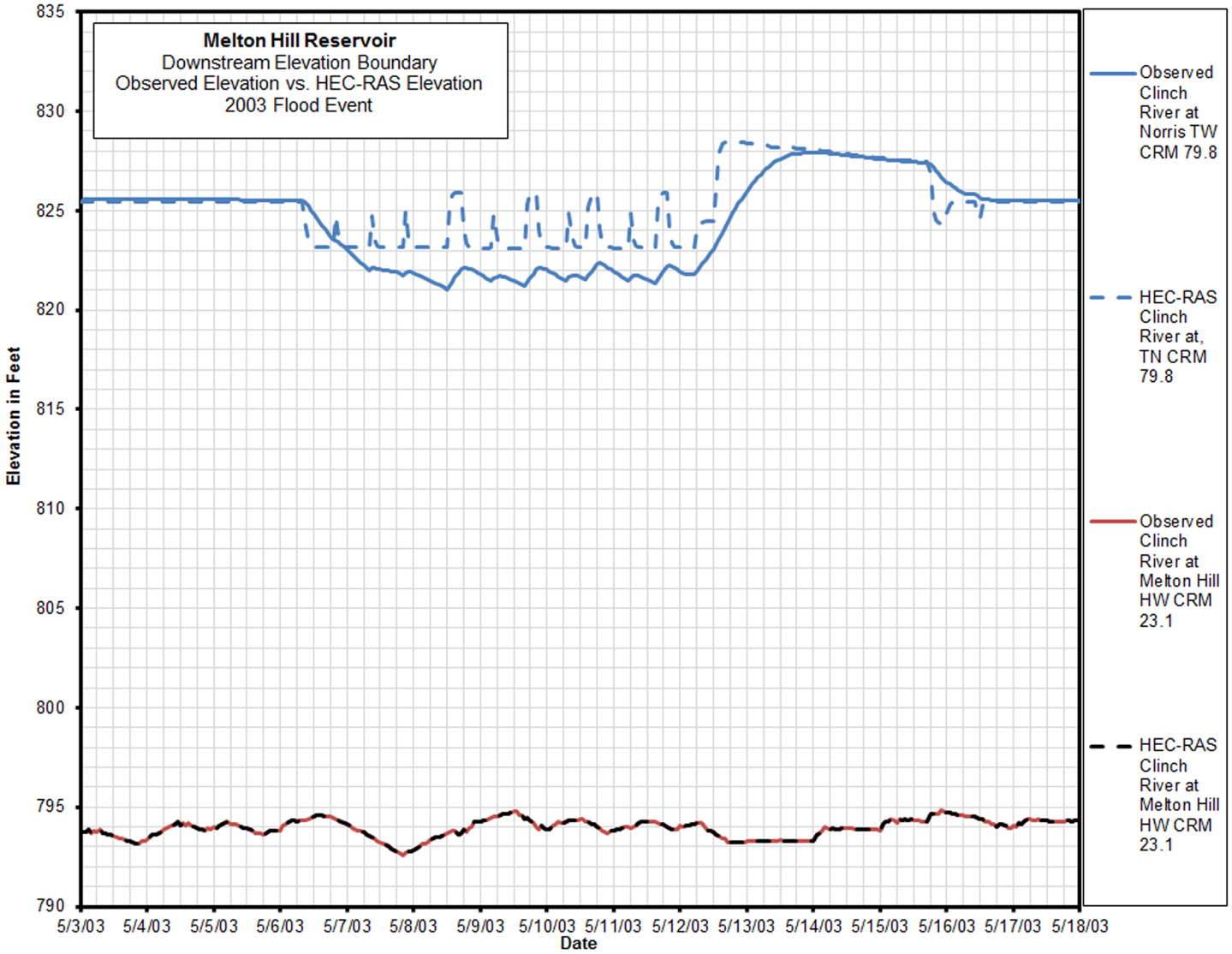


Figure 2.4.3-13. Melton Hill Reservoir Calibration Results – Elevation: 2003 Flood Event

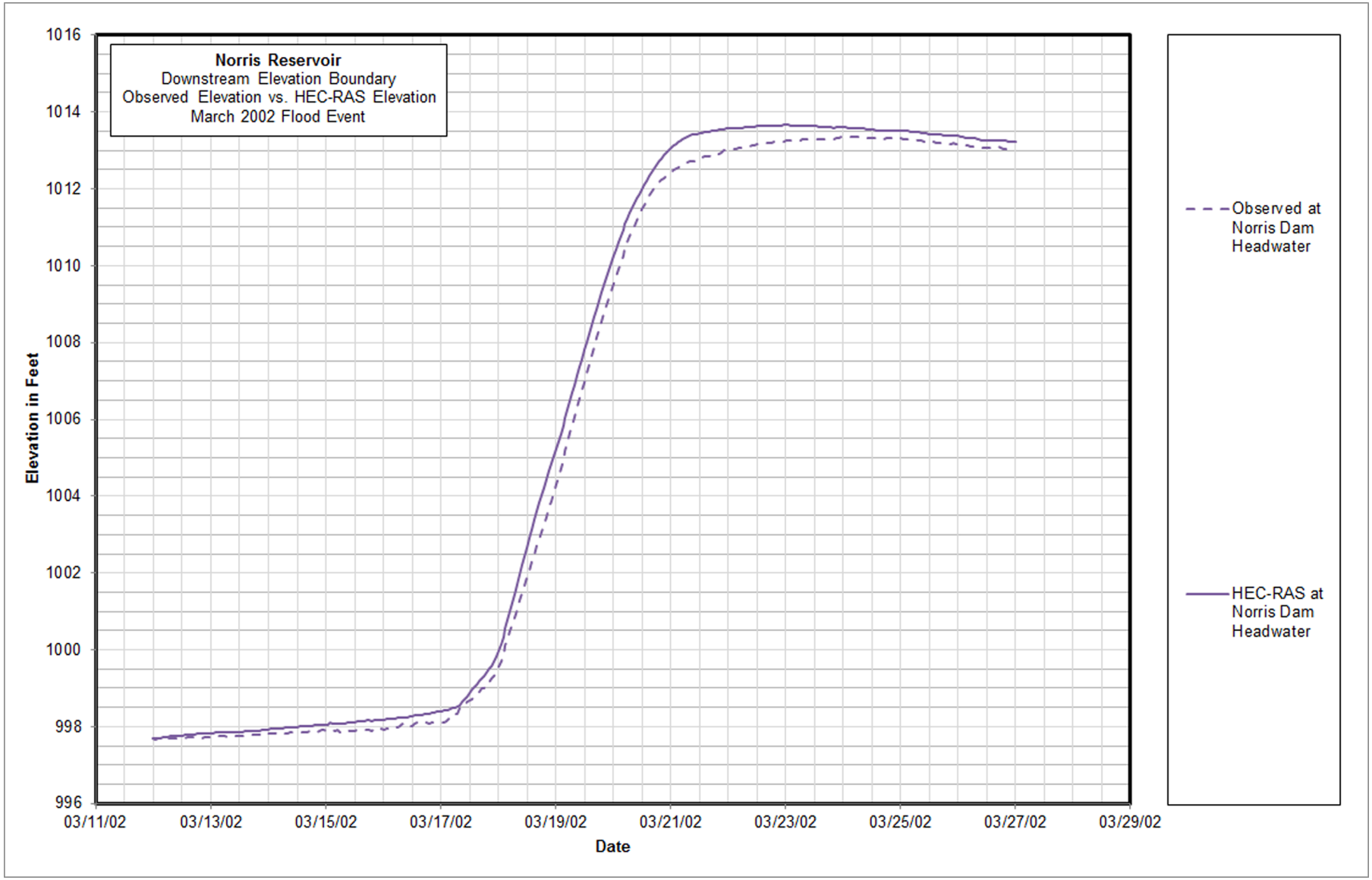


Figure 2.4.3-14. Norris Reservoir Calibration Results – Elevation: 2002 Flood Event

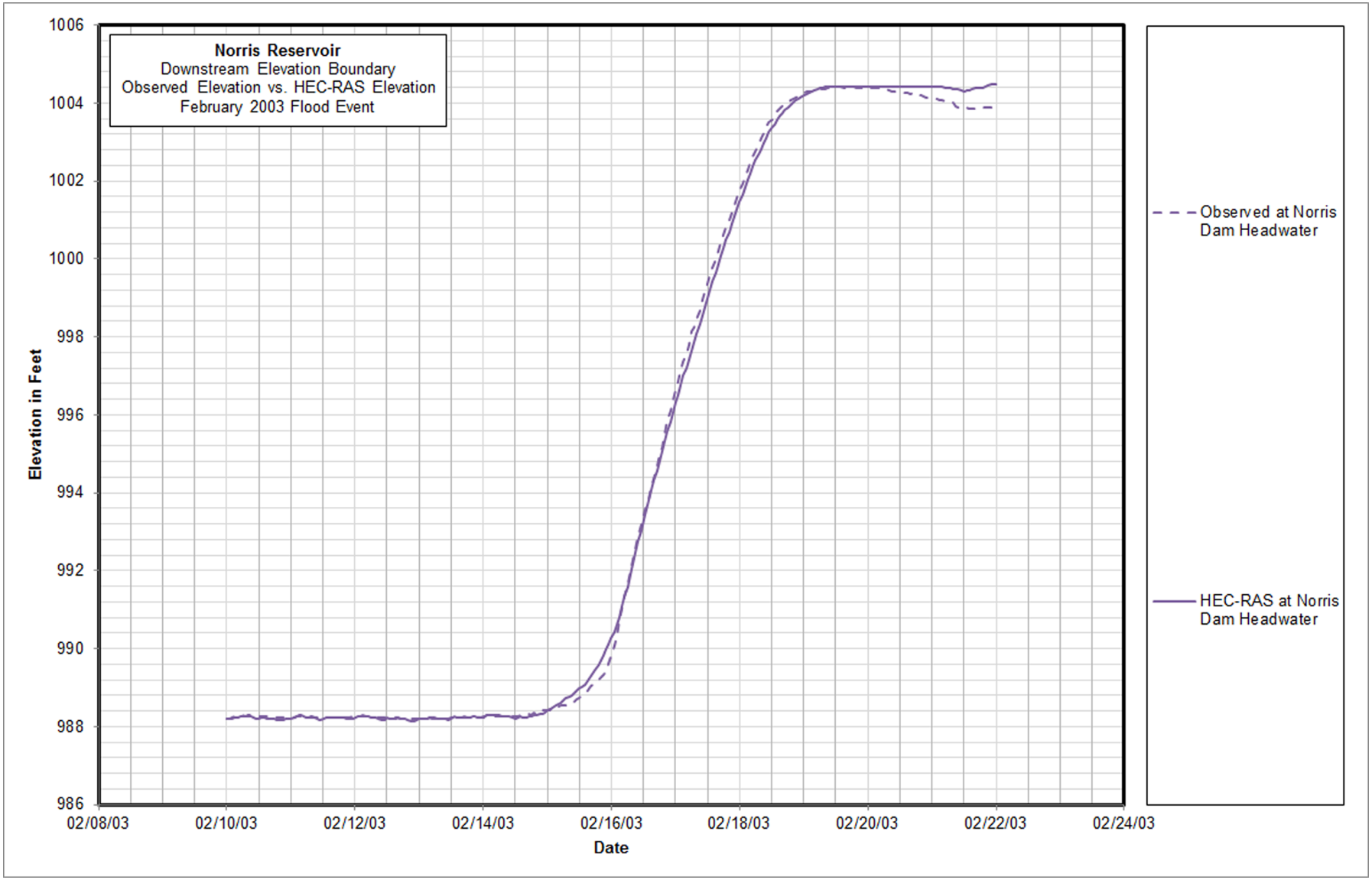


Figure 2.4.3-15. Norris Reservoir Calibration Results – Elevation: 2003 Flood Event

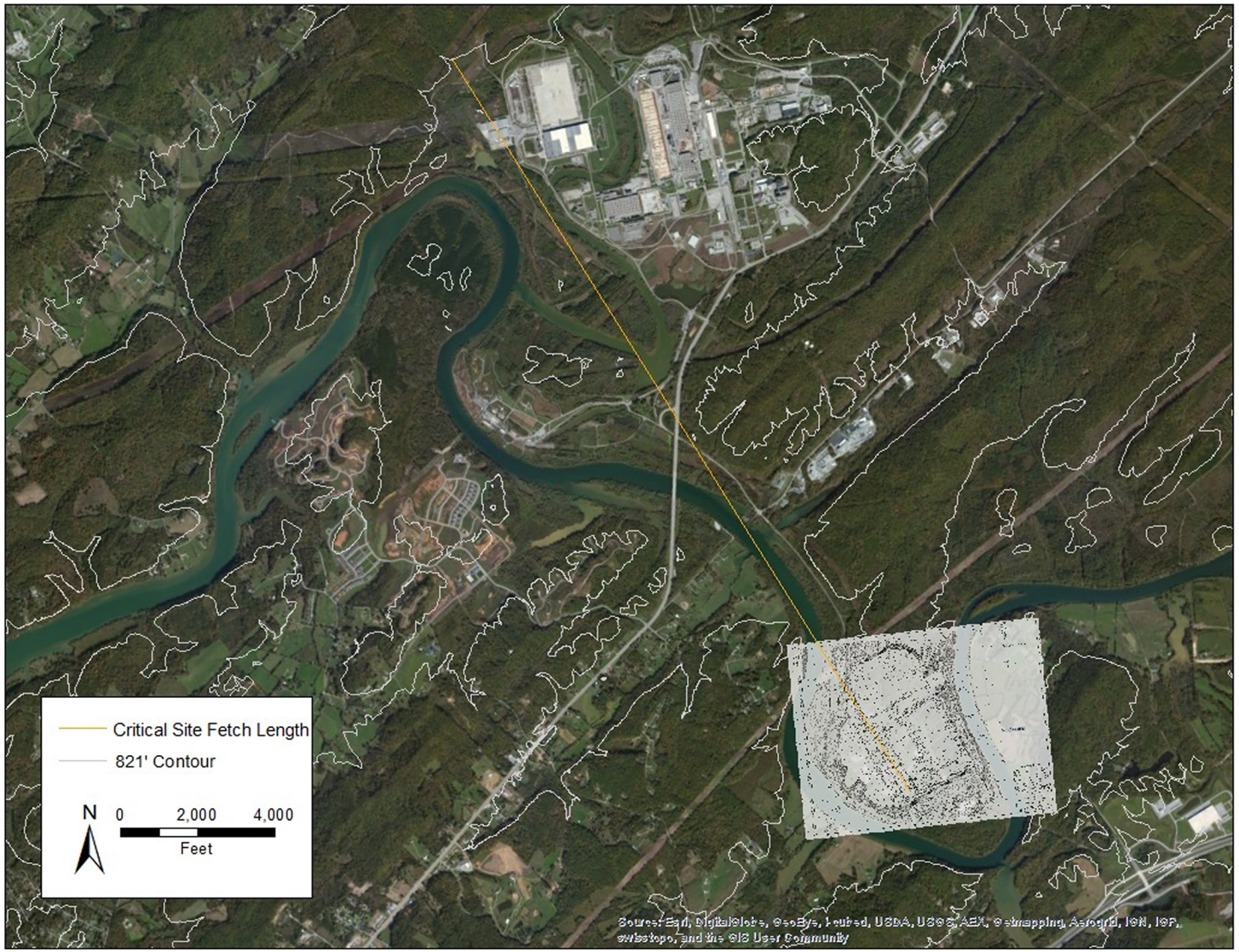


Figure 2.4.3-16. CRN Site Critical Fetch Length

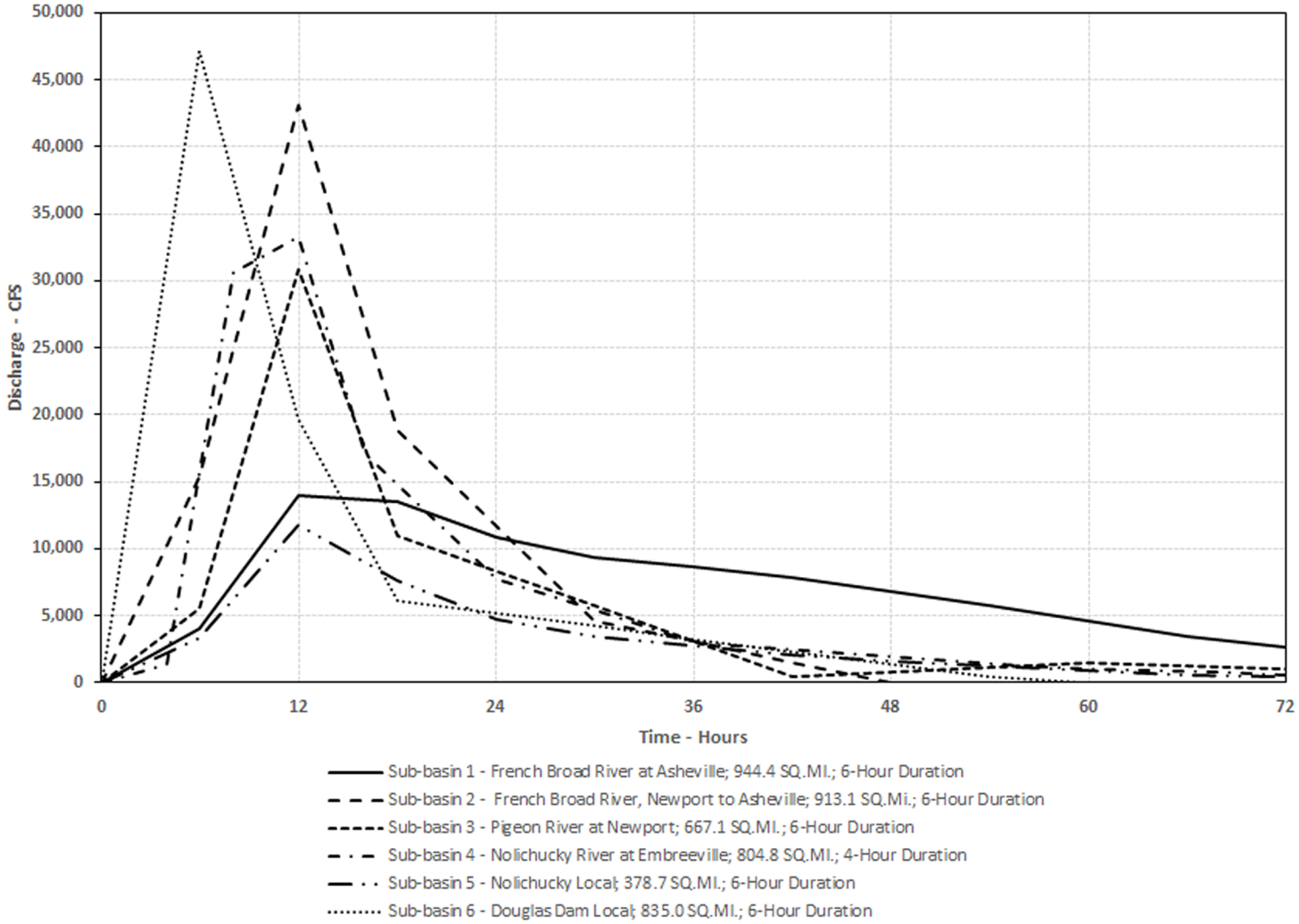


Figure 2.4.3-17. (Sheet 1 of 8) Unit Hydrographs, Sub-basins 1–6

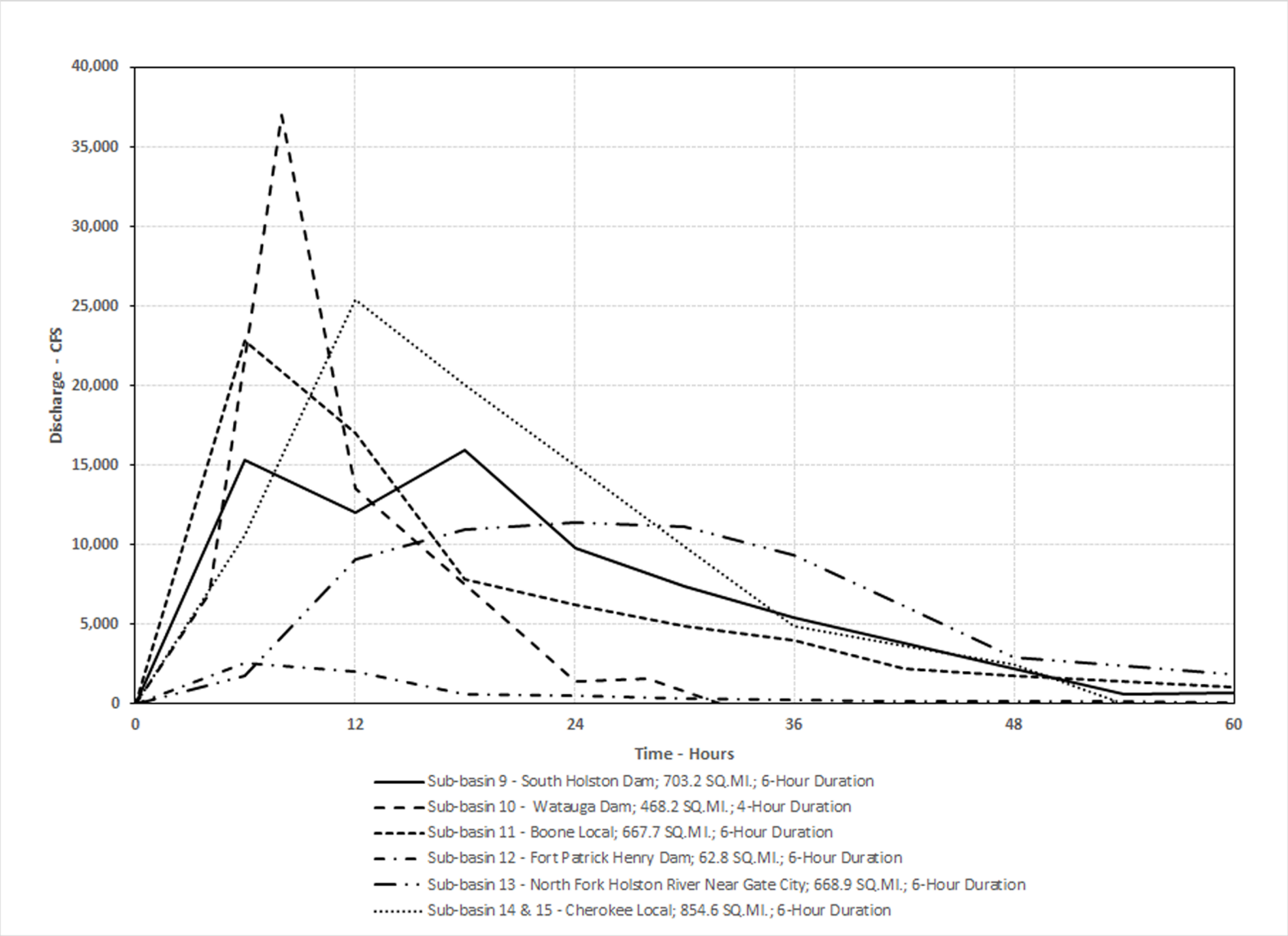


Figure 2.4.3-17. (Sheet 2 of 8) Unit Hydrographs, Sub-basins 9–15

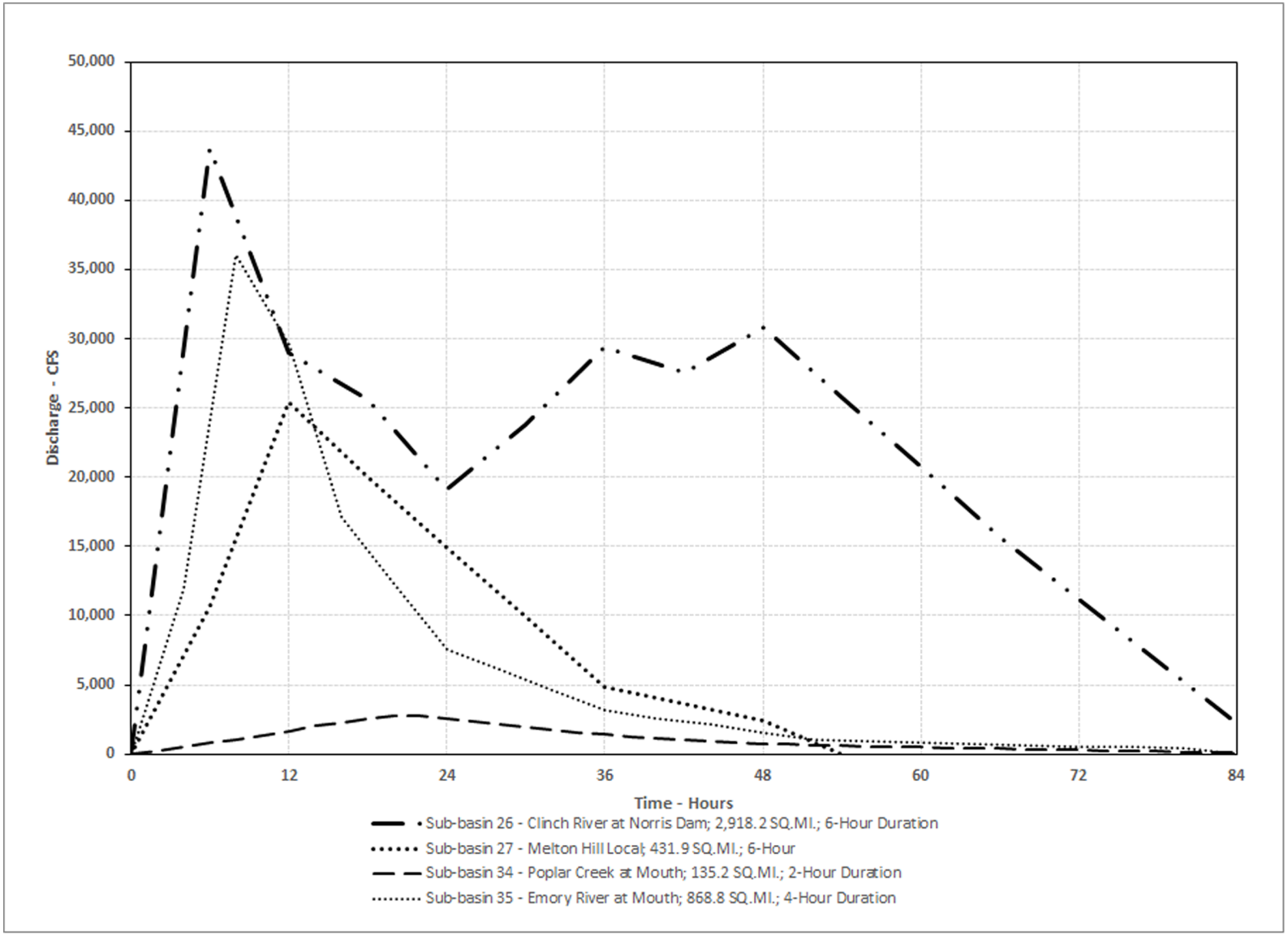


Figure 2.4.3-17. (Sheet 3 of 8) Unit Hydrographs, Sub-basins 26, 27, 34, & 35

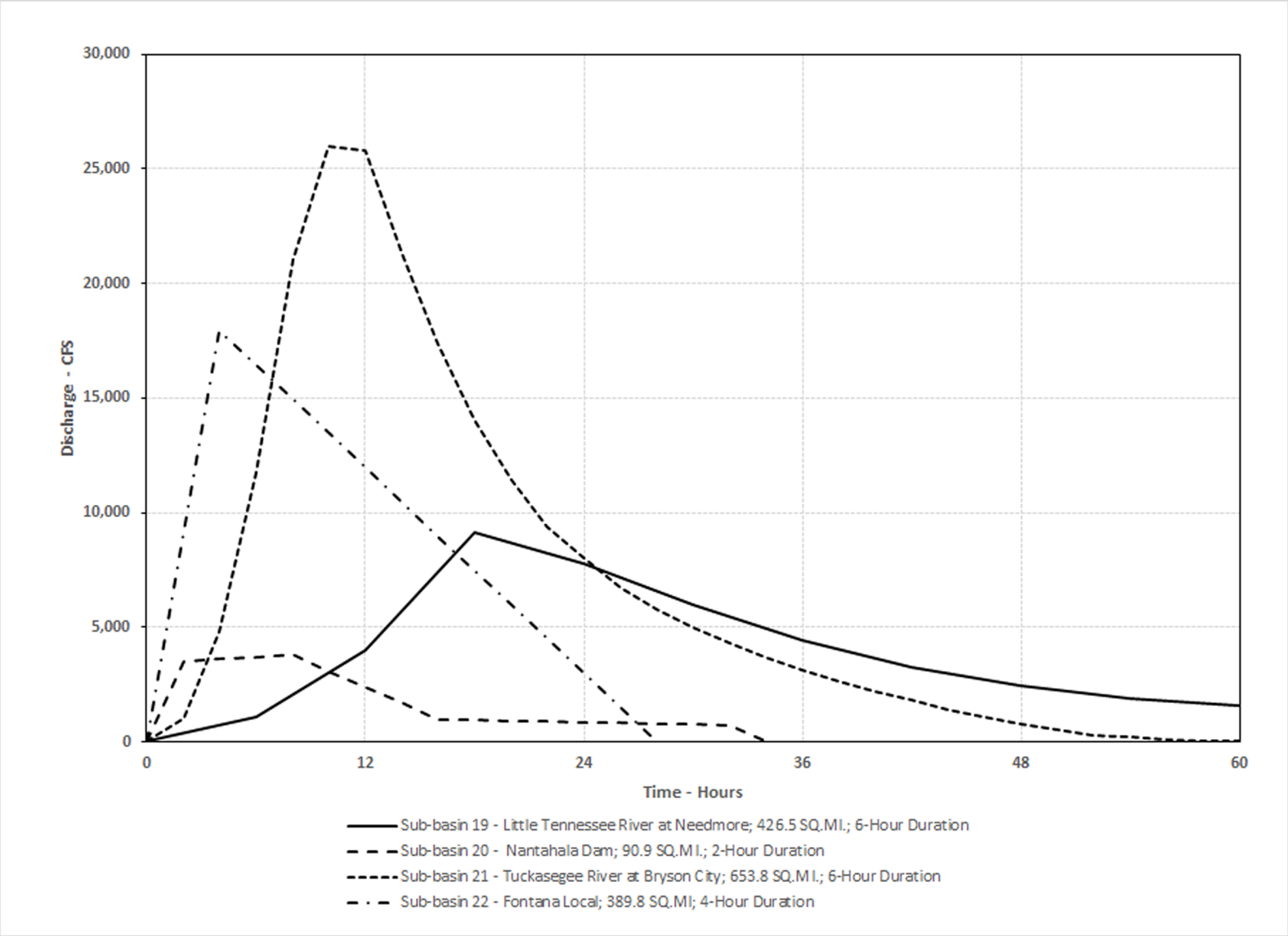


Figure 2.4.3-17. (Sheet 4 of 8) Unit Hydrographs, Sub-basins 19–22

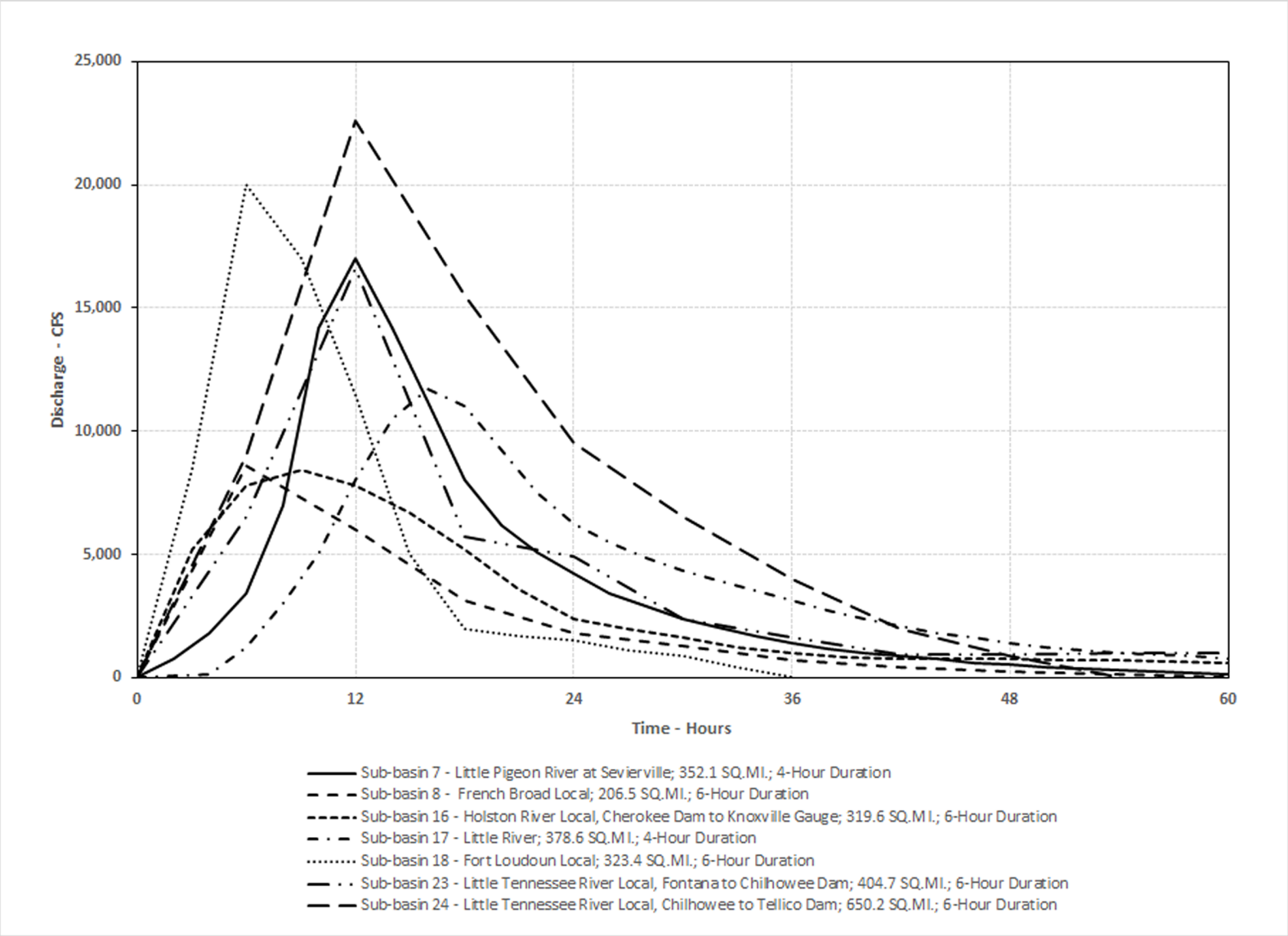


Figure 2.4.3-17. (Sheet 5 of 8) Unit Hydrographs, Sub-basins 7, 8, 16–18, & 23–24

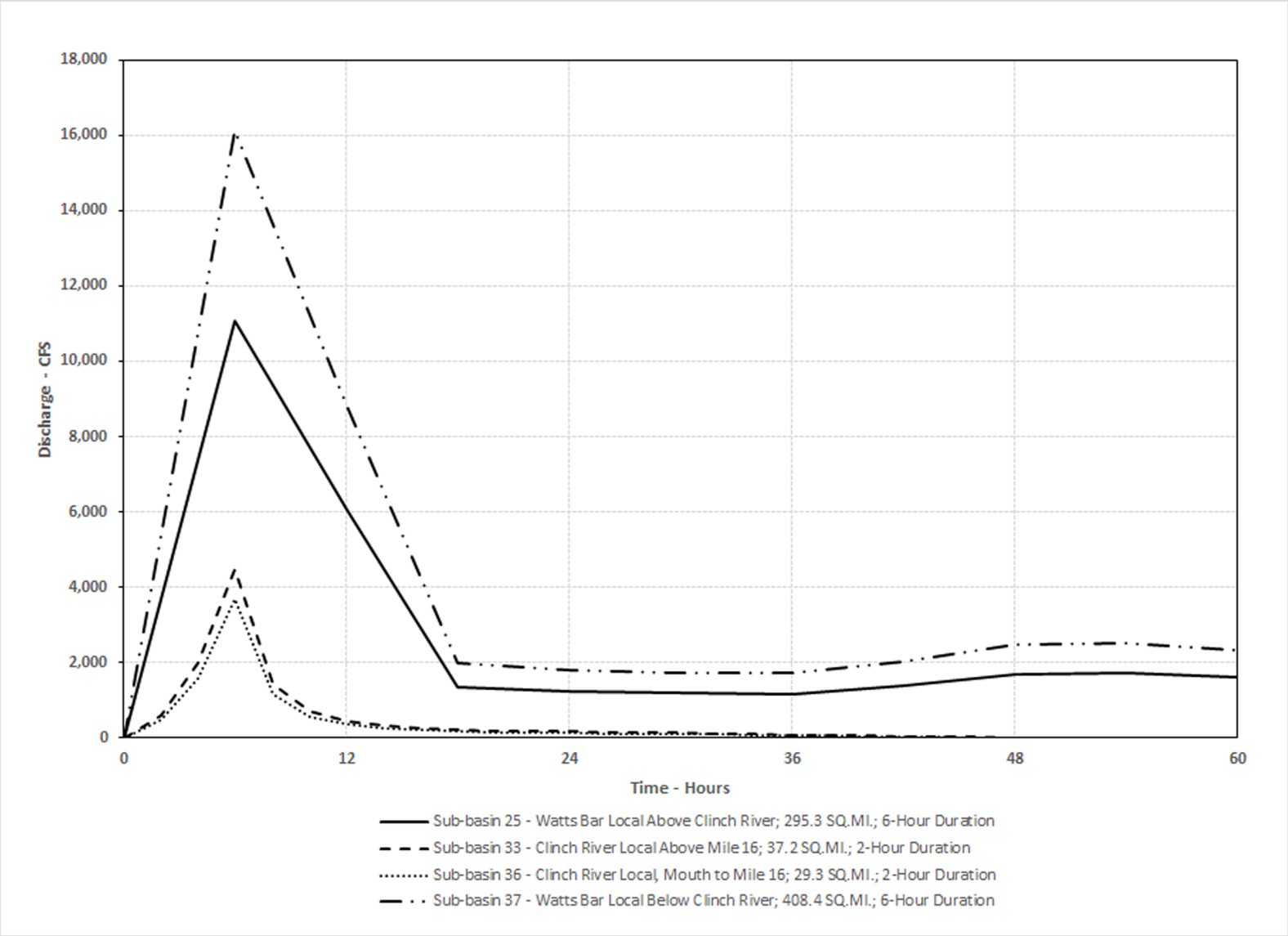


Figure 2.4.3-17. (Sheet 6 of 8) Unit Hydrographs, Sub-basins 25, 33, 36, & 37

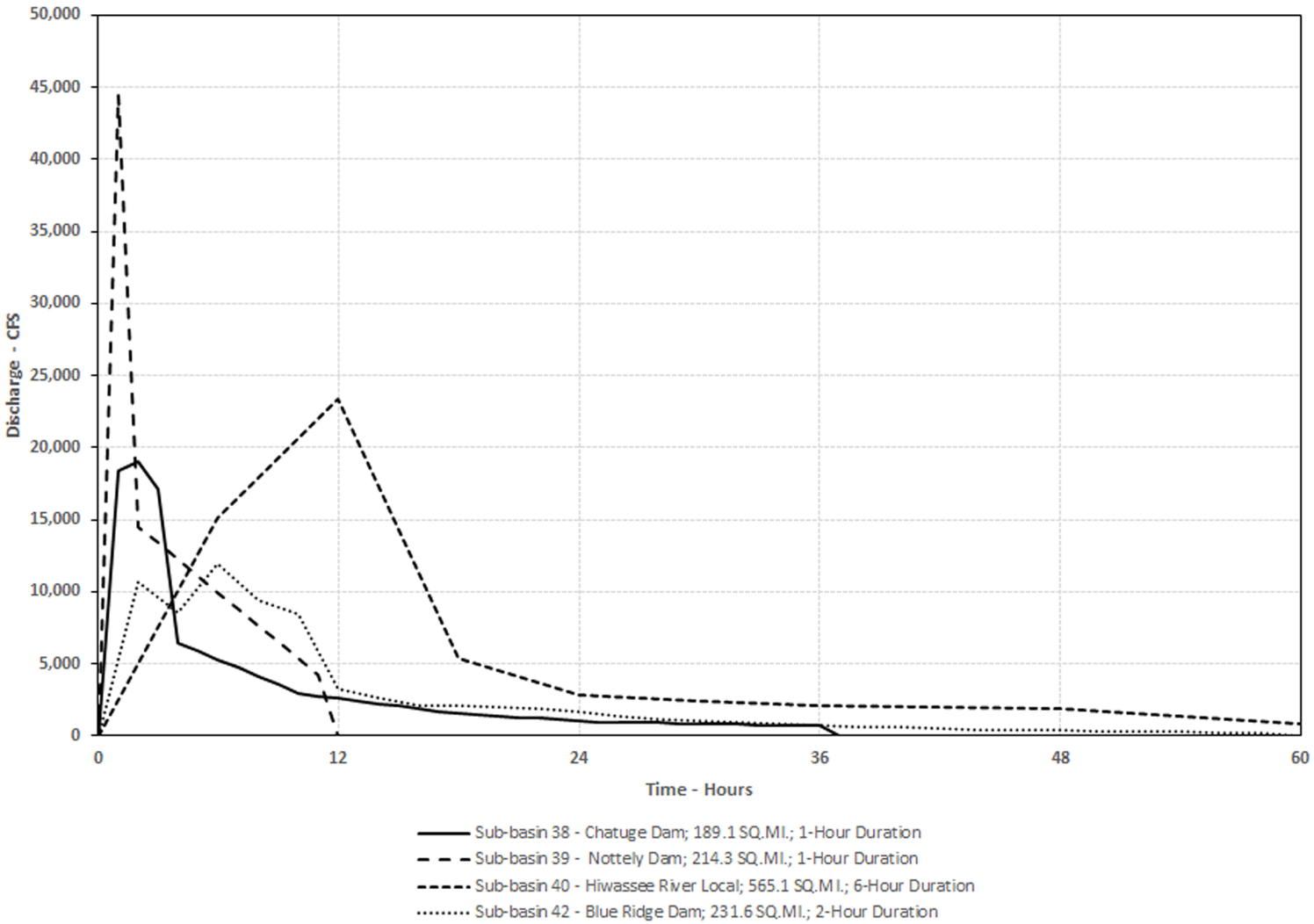


Figure 2.4.3-17. (Sheet 7 of 8) Unit Hydrographs, Sub-basins 38–40, & 42

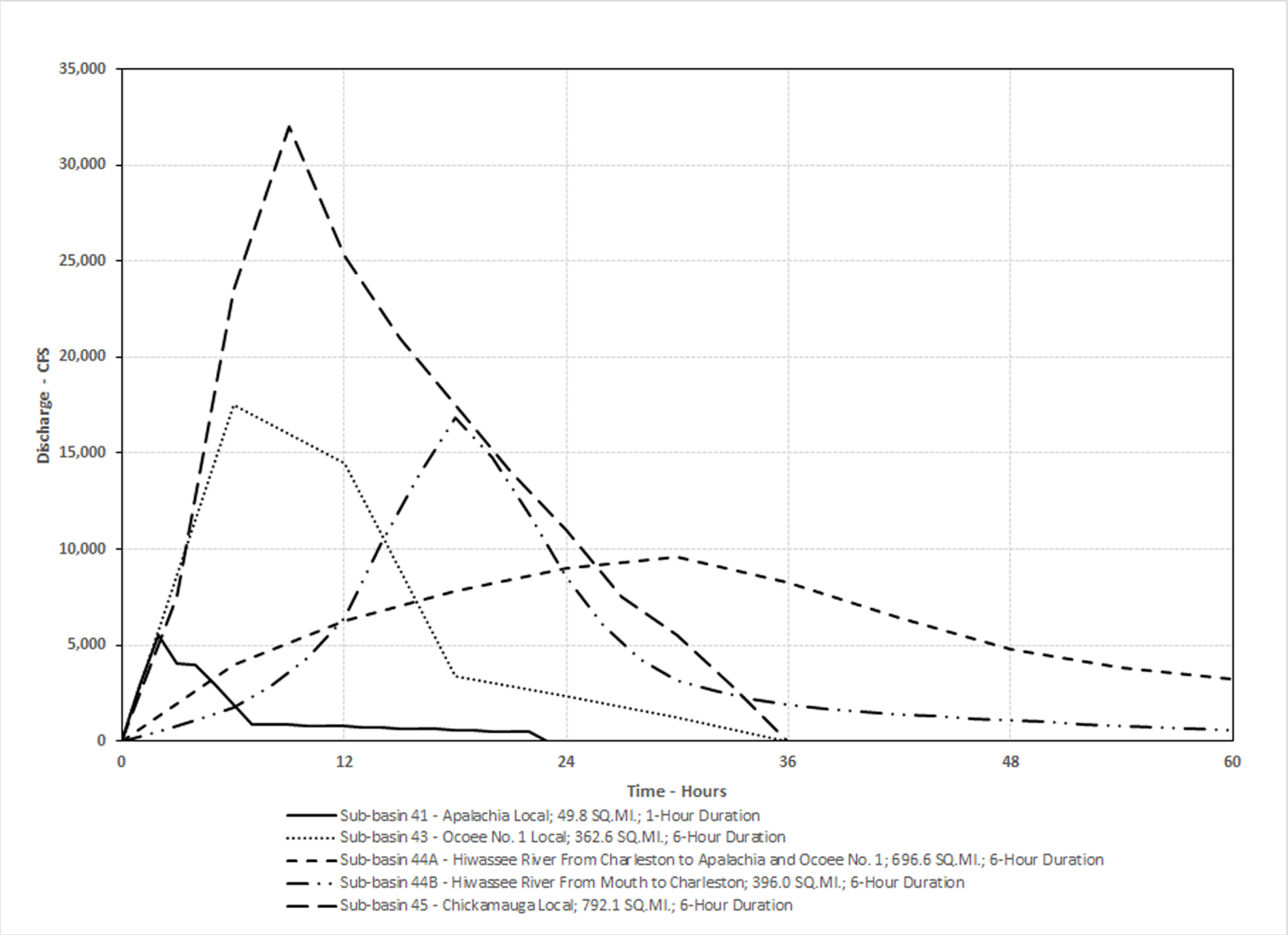


Figure 2.4.3-17. (Sheet 8 of 8) Unit Hydrographs, Sub-basins 41, 43, 44A, 44B, & 45

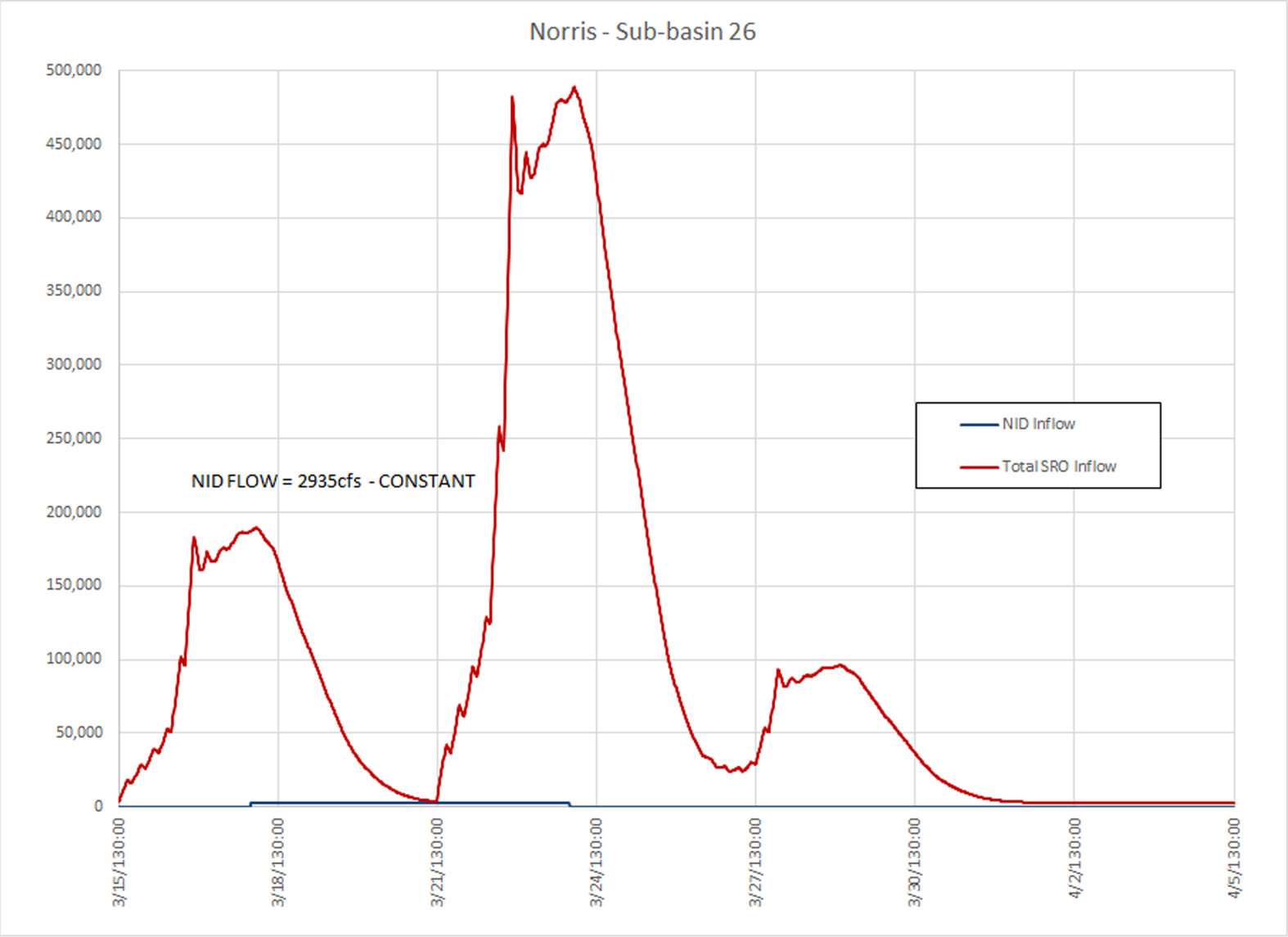


Figure 2.4.3-18. NID Inflow Hydrograph, Norris Reservoir

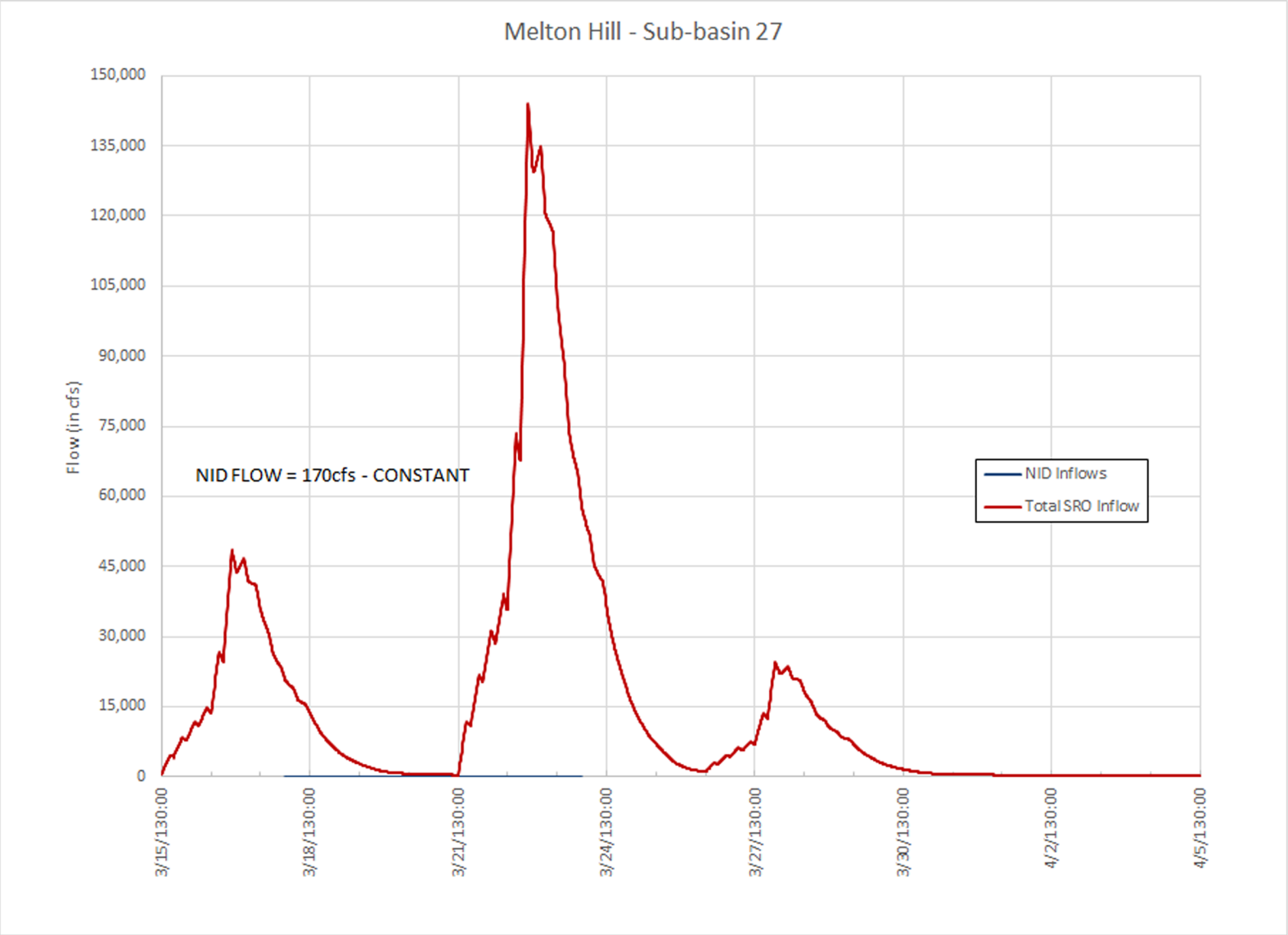


Figure 2.4.3-19. Melton Hill Reservoir

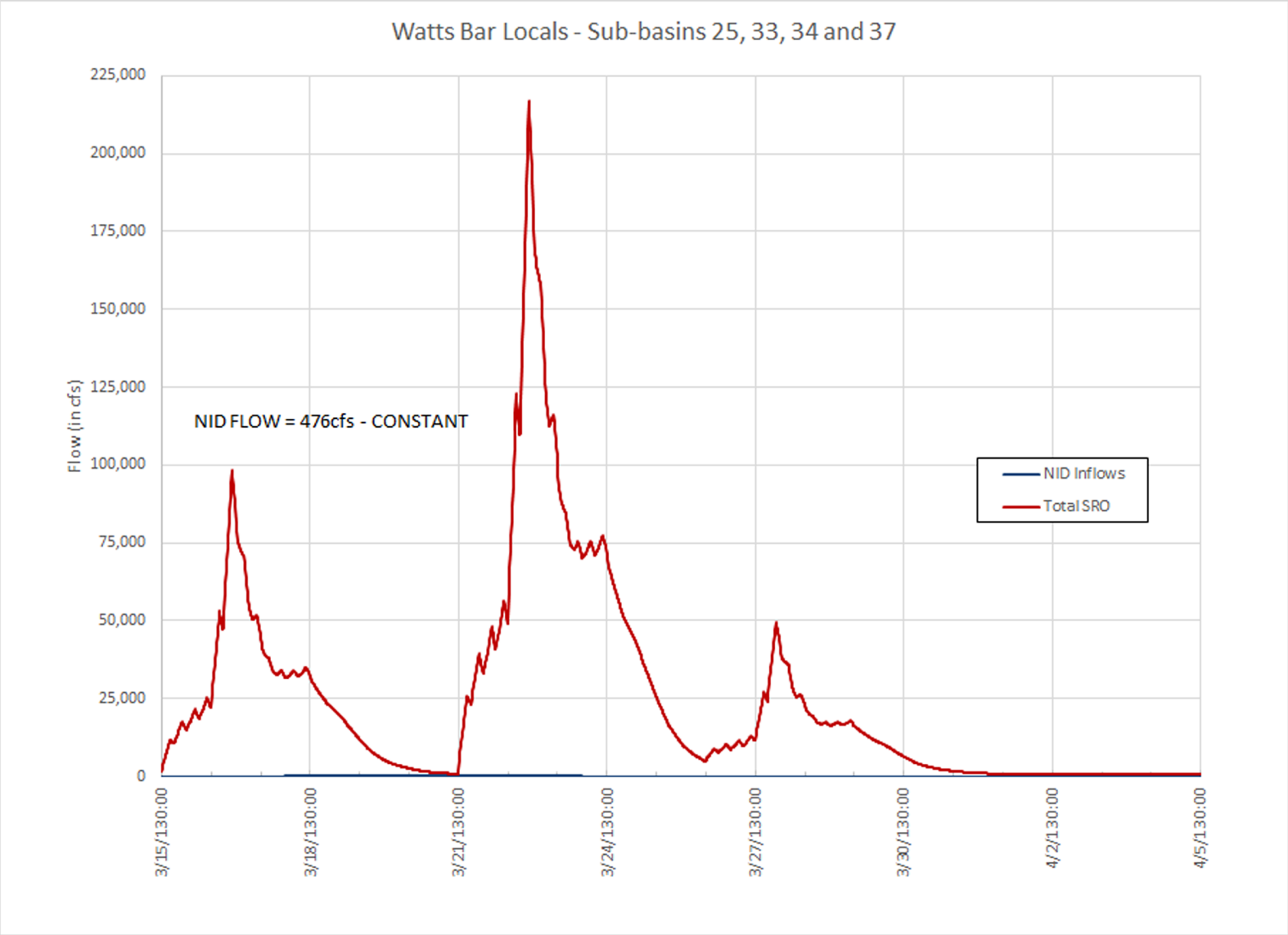


Figure 2.4.3-20. Watts Bar Reservoir

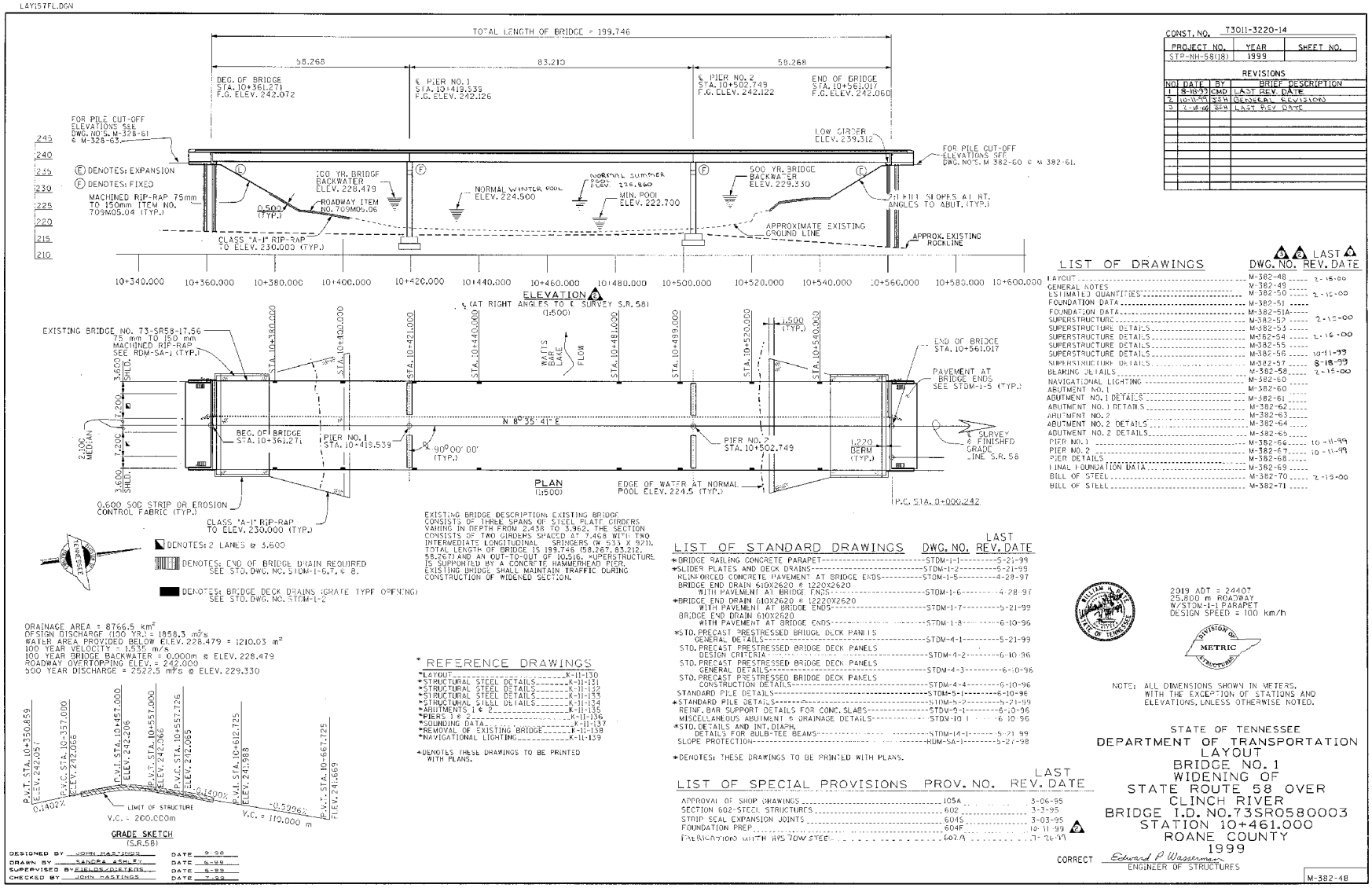


Figure 2.4.3-21. Highway 58 Bridge Profile

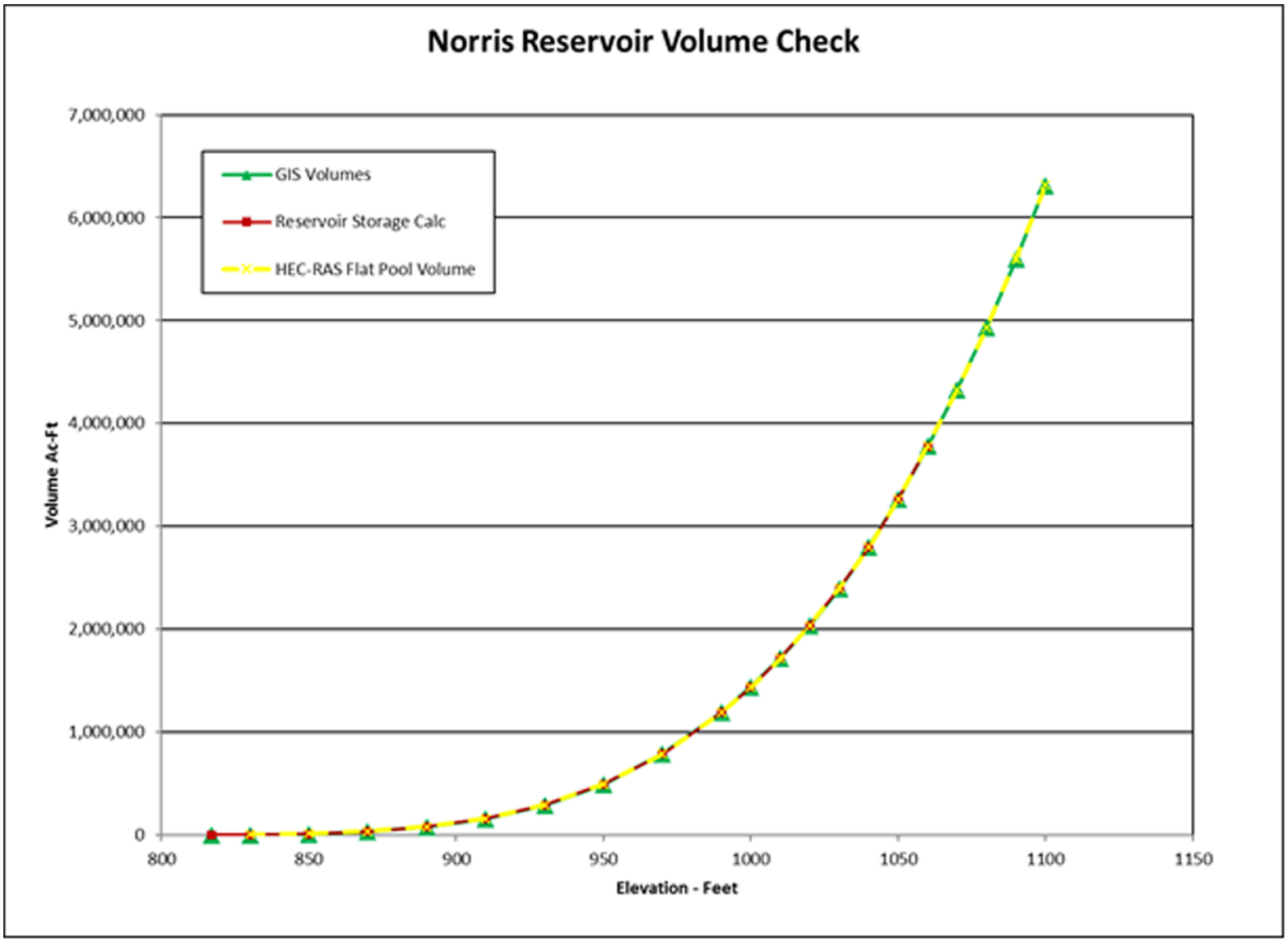


Figure 2.4.3-22. Norris Reservoir Volume Versus Elevation

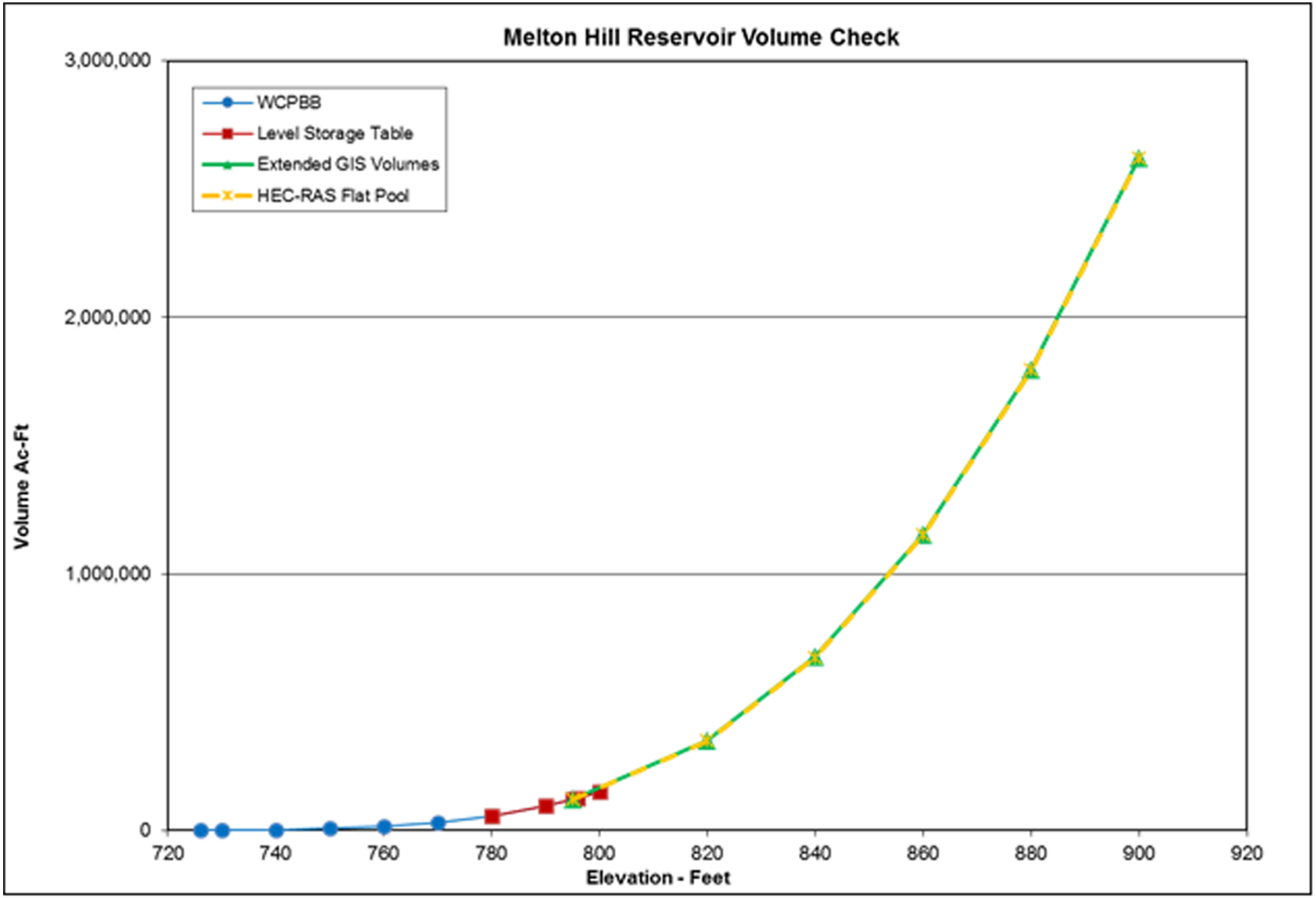


Figure 2.4.3-23. Melton Hill Reservoir Volume Versus Elevation

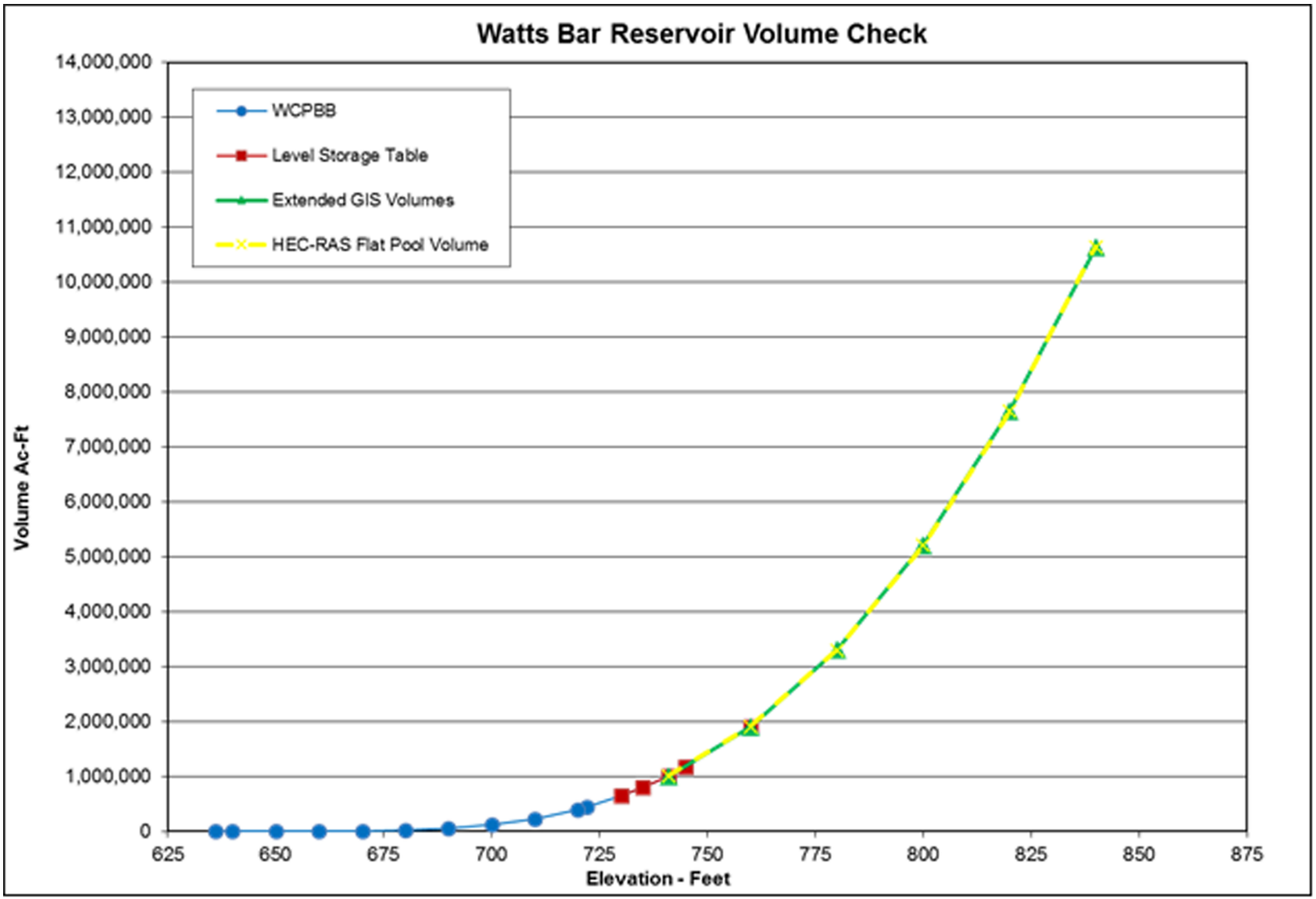


Figure 2.4.3-24. Watts Bar Reservoir Volume Versus Elevation