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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 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 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 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 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 the middle of the respective storms. The adopted sequence closely conforms to the method used by the U.S. Army Corps of Engineers (USACE) (Reference 2.4.3-7). 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 17.05 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 subareas and includes the total 30,747 sq mi watershed above Wilson Dam. 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 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.

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. 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. 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 Calibration

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.

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. 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. The model results were approximately equivalent to the 1973 flood at the Melton Hill Dam tailwater and reproduced the 2003 flood within one foot. 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. 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. 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 reservoir specific guidelines, or flood operational guides, are based on original project allocations and subsequent modifications, many years of historical flows, flood season conditions and experience with project and reservoir system operations. 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. 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. The dam rating curves are 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 2.4.3-9.

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. [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. 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

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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. Hansen, E.M., L.C. Schreiner, and J.F. Miller, *Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, Hydrometeorological Report 52*, Water Management Information Division, National Weather Service, NOAA, Silver Spring, Maryland, August 1982.

Table 2.4.3-1
Flood Events Analyzed

(SRI/CEII)

Clinch River Event	**Significant Failures Above Watts Bar Dam**	**Peak at CRN Site**	
		Elevation (ft NGVD29)	**Flow (cfs)**
2912 sq mi, centered above Norris Dam, June storm event	[redacted]	[redacted]	543,000
3382 sq mi, centered above CRN, June storm event	[redacted]	[redacted]	544,000
469 sq mi, centered between CRN and Norris Dam, June storm event	[redacted]	[redacted]	200,000
7980 sq mi, Bull's Gap centered, March PMF with 100% runoff and peaked/lagged unit hydrographs storm event	[redacted]	[redacted]	536,000
Half-10,000-Yr Douglas Centered Seismic Event During A 500–Yr June Flood Event	[redacted]	[redacted]	162,000
[redacted] Dam Sunny Day Failure	[redacted]	[redacted]	579,000

Notes:
CRN = Clinch River Nuclear
PMF = Probable Maximum Flood

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	Source
	150	20% of 2nd Day	Table 7-2, HMR-41
7	156	23% of 2nd Day	Table 7-2, HMR-41
	162	27% of 2nd Day	Table 7-2, HMR-41
	168	30% of 2nd Day	Table 7-2, HMR-41
	174	2nd 6-hr	Table 7-2, HMR-41
8	180	1st 6-hr	Table 7-2, HMR-41
	186	3rd 6-hr	Table 7-2, HMR-41
	192	4th 6-hr	Table 7-2, HMR-41
	198	28% of 3rd Day	Table 7-2, HMR-41
9	204	26% of 3rd Day	Table 7-2, HMR-41
	210	23% of 3rd Day	Table 7-2, HMR-41
	216	23% of 3rd Day	Table 7-2, HMR-41

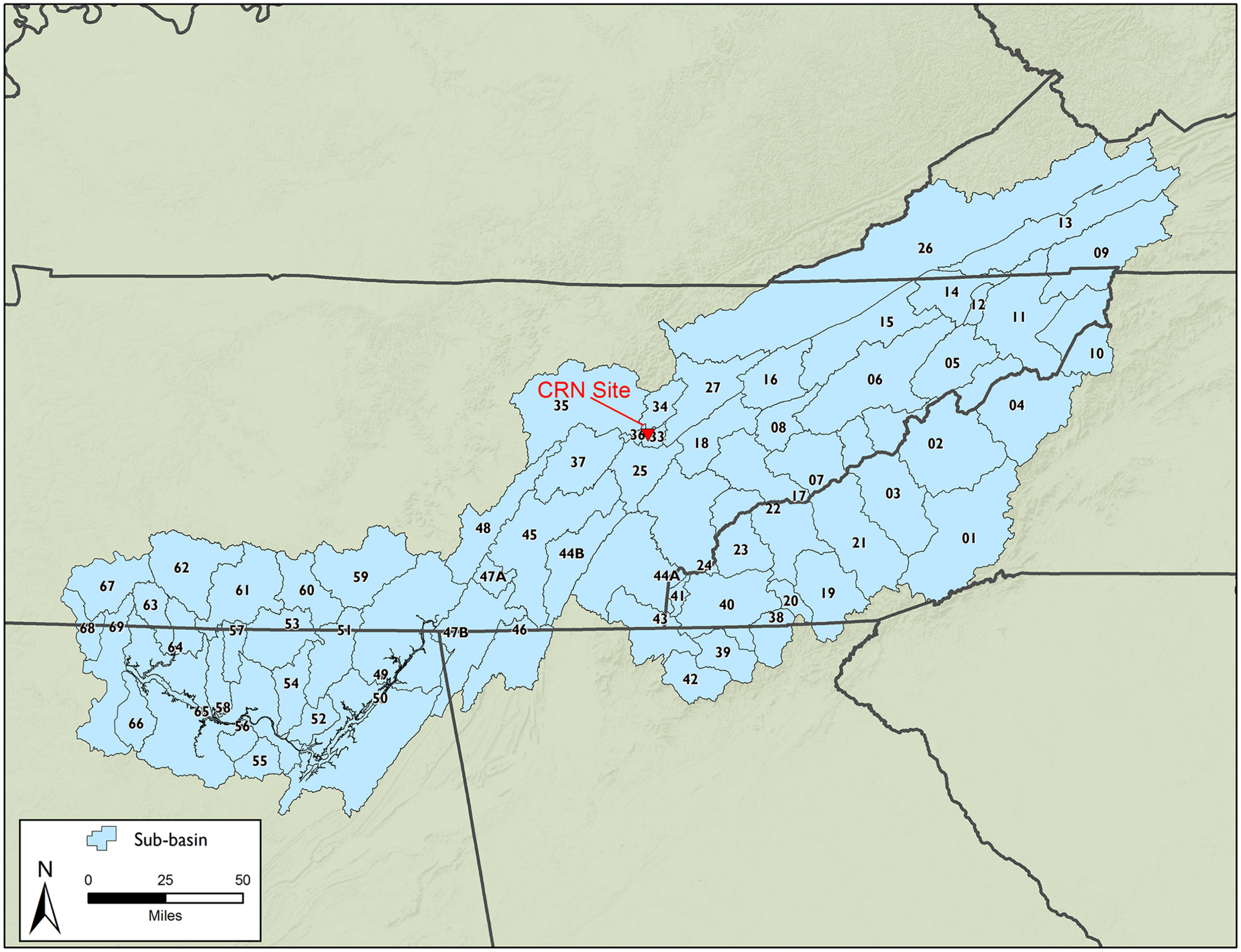


Figure 2.4.3-1. Tennessee River System Watershed Sub-Basins

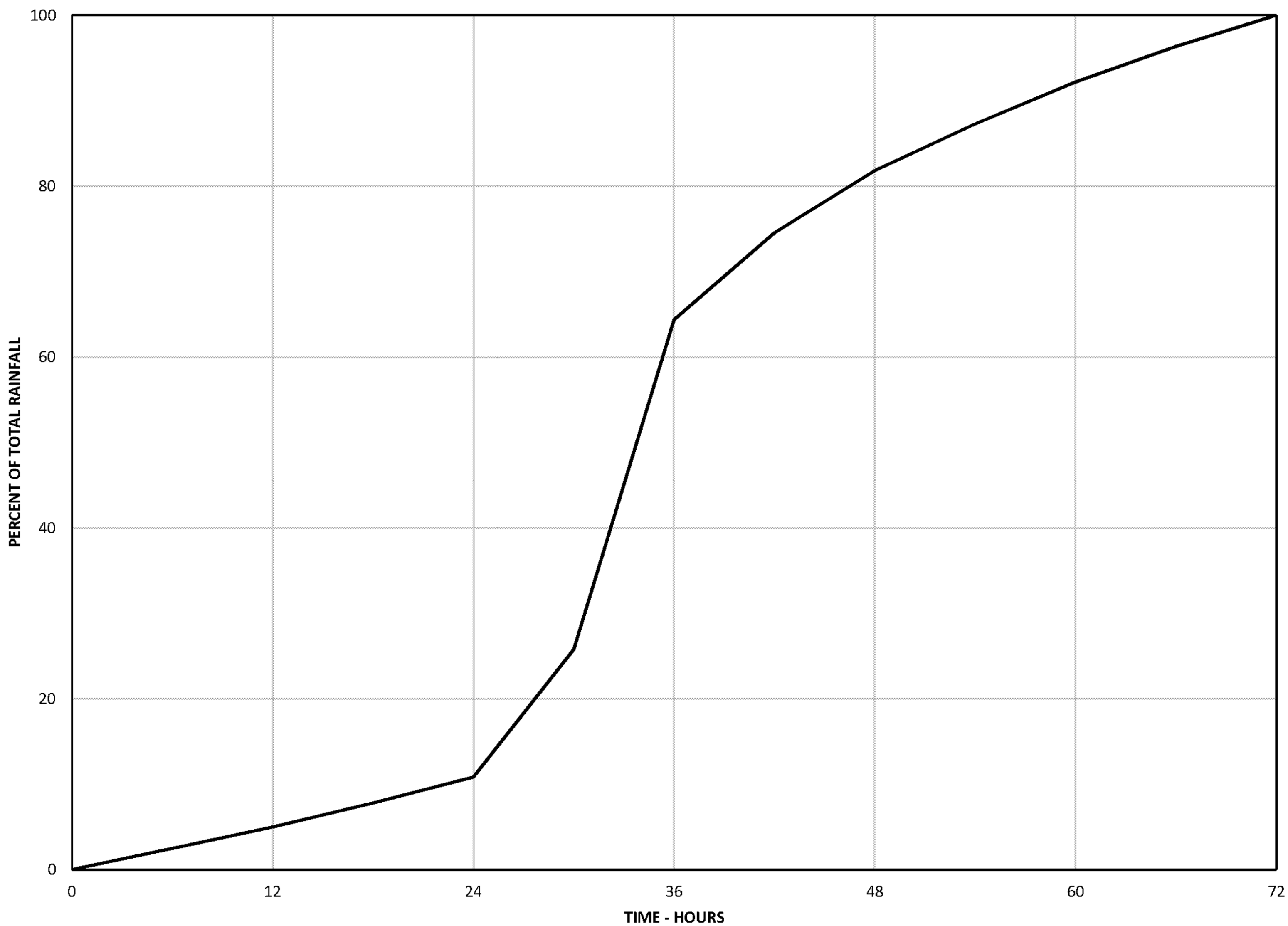
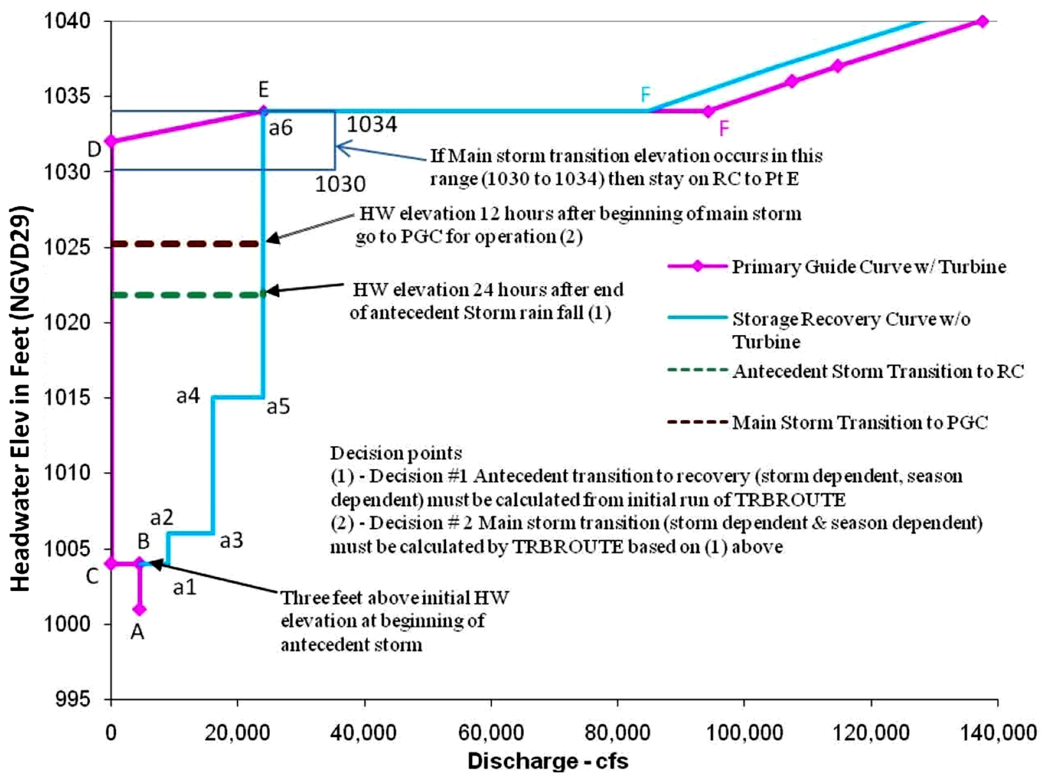


Figure 2.4.3-2. Rainfall Time Distribution – Typical Mass Curve

(SRI/CEII)

[redacted]

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

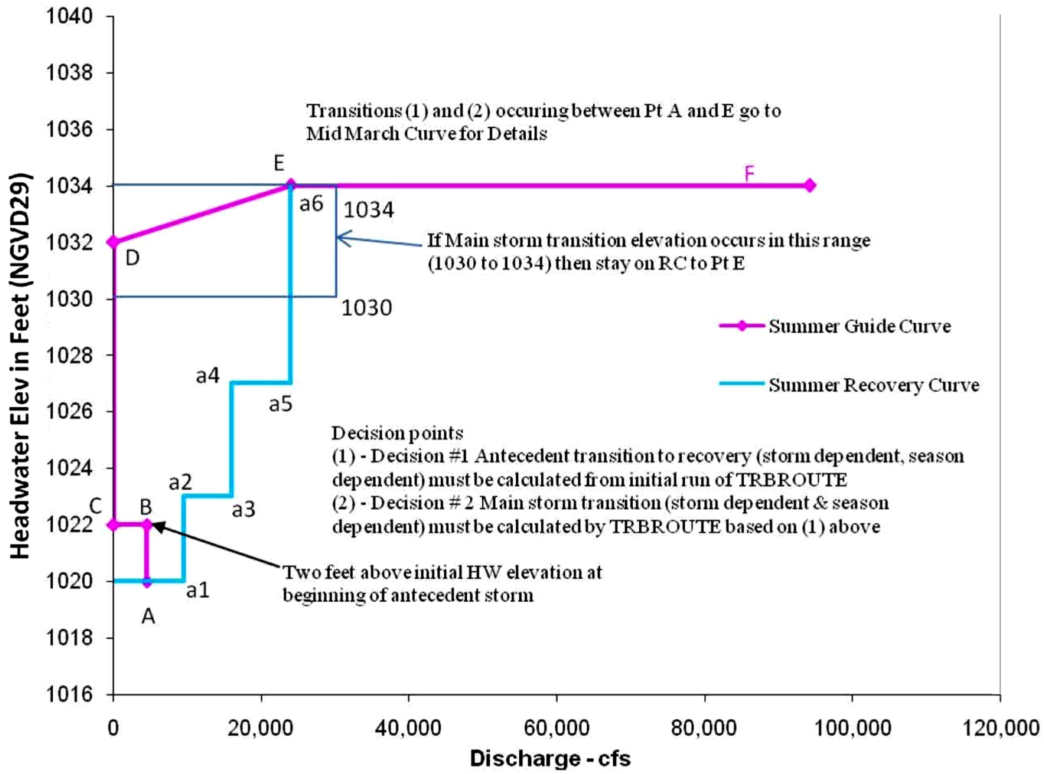


Notes:
1. All transition points (dashed lines) are shown for illustration purpose only. Transitions are storm dependent.
2. The possibility exists that the antecedent storm exceeds point E elevation prior to transition.

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

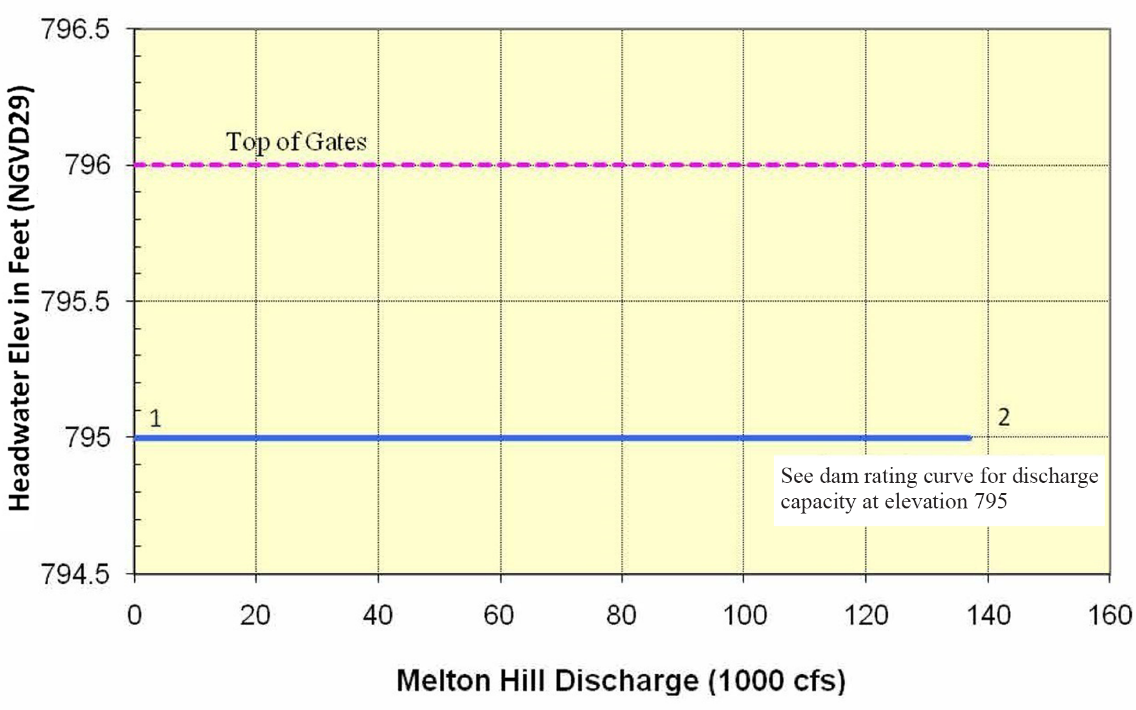
Figure 2.4.3-4. (Sheet 1 of 2) Flood Operational Guide – Norris Dam



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

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

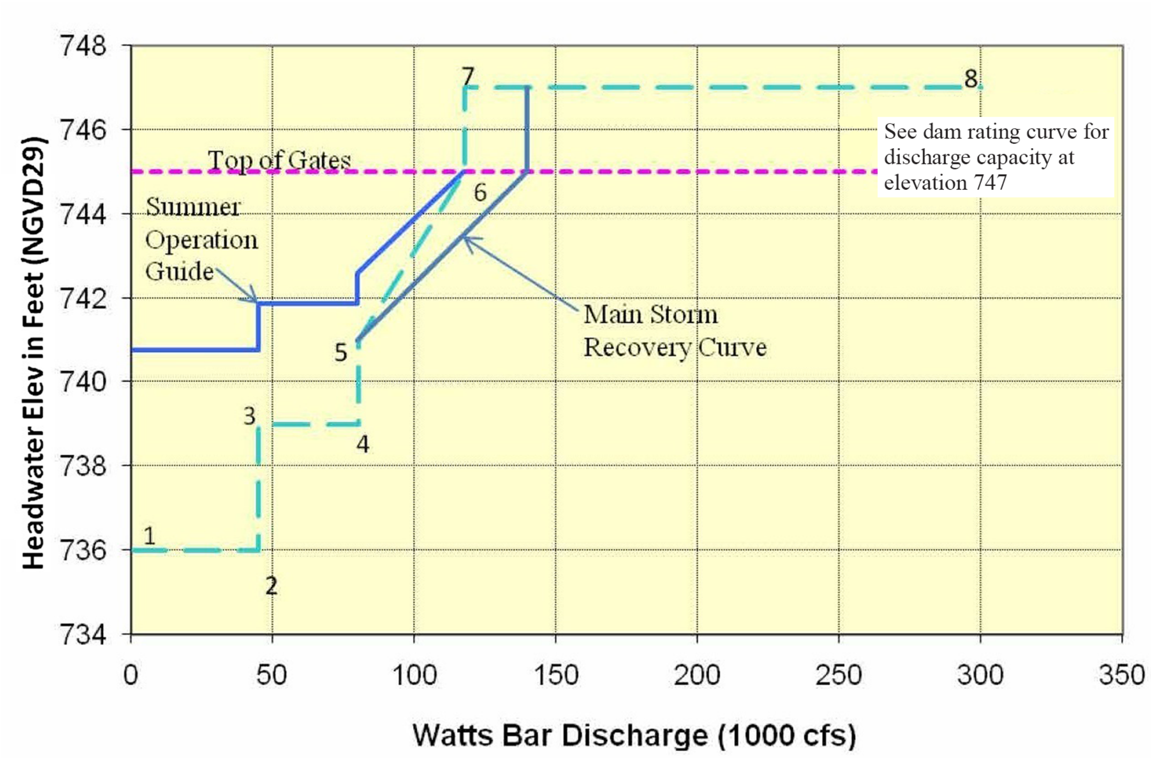
Notes:

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

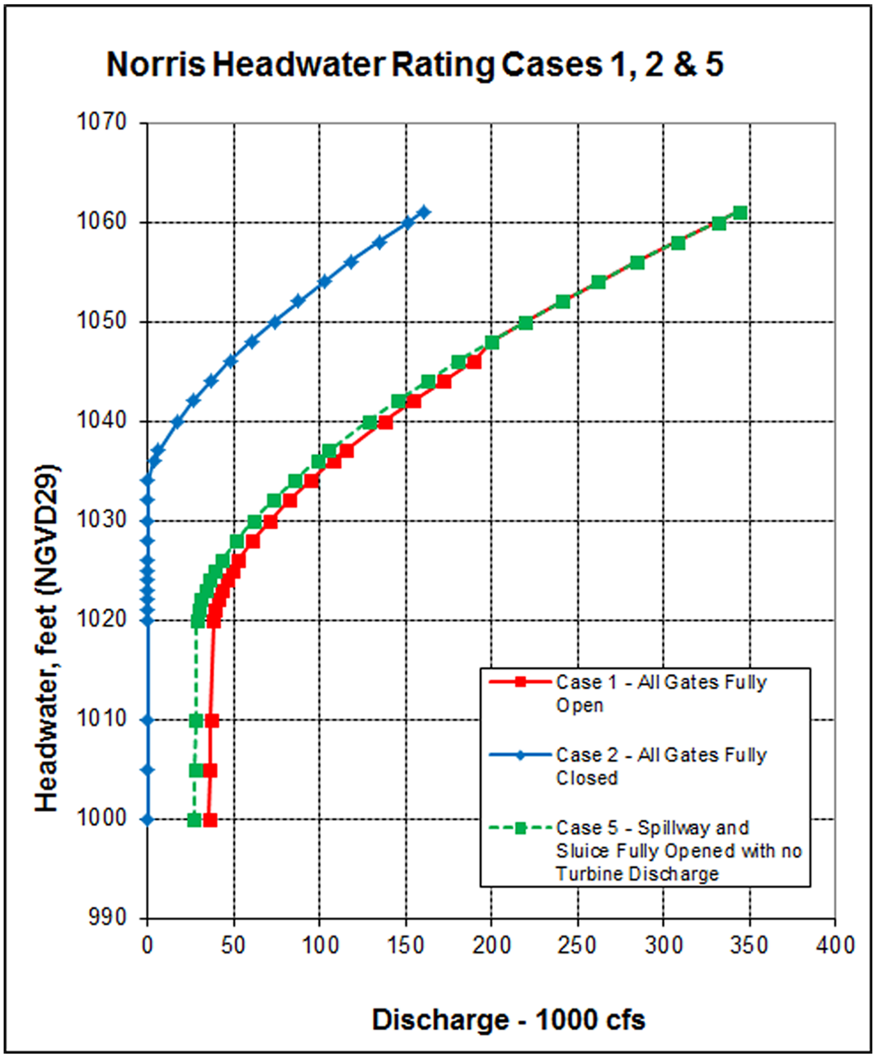


Figure 2.4.3-7. (Sheet 1 of 2) Dam Rating Curve – Norris Dam

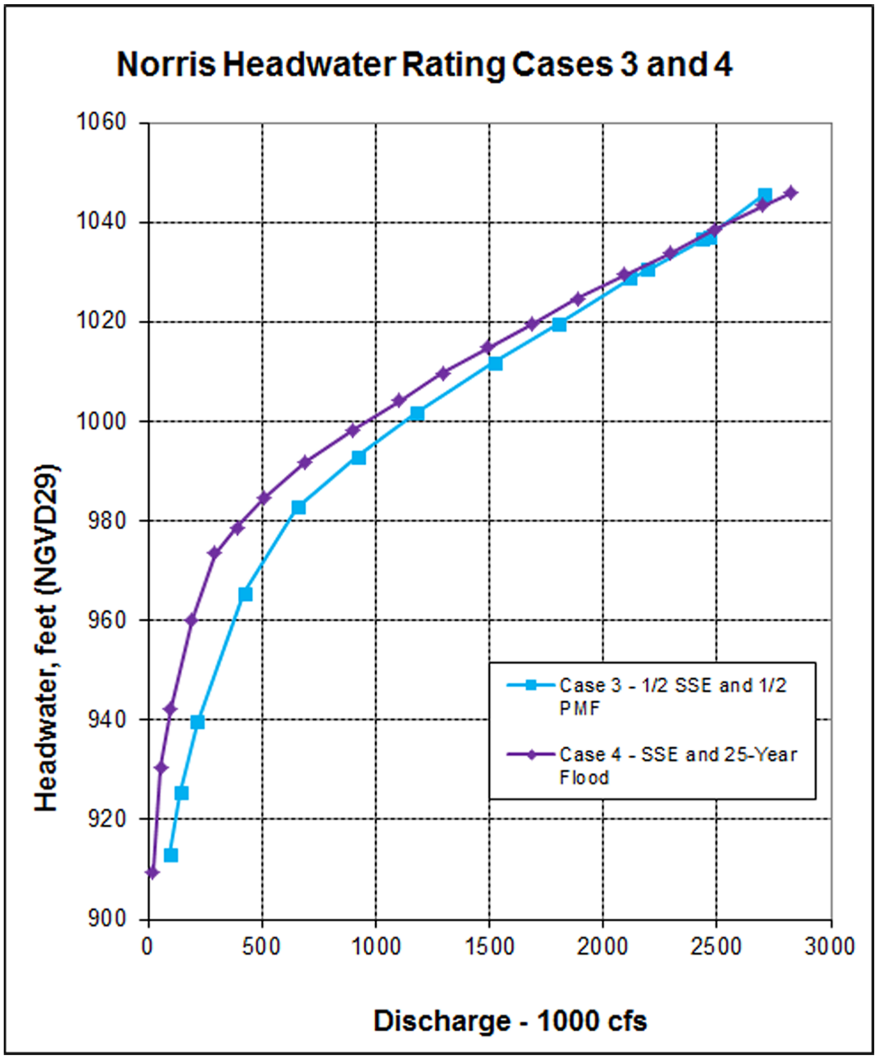


Figure 2.4.3-7. (Sheet 2 of 2) Dam Rating Curve – Norris Dam

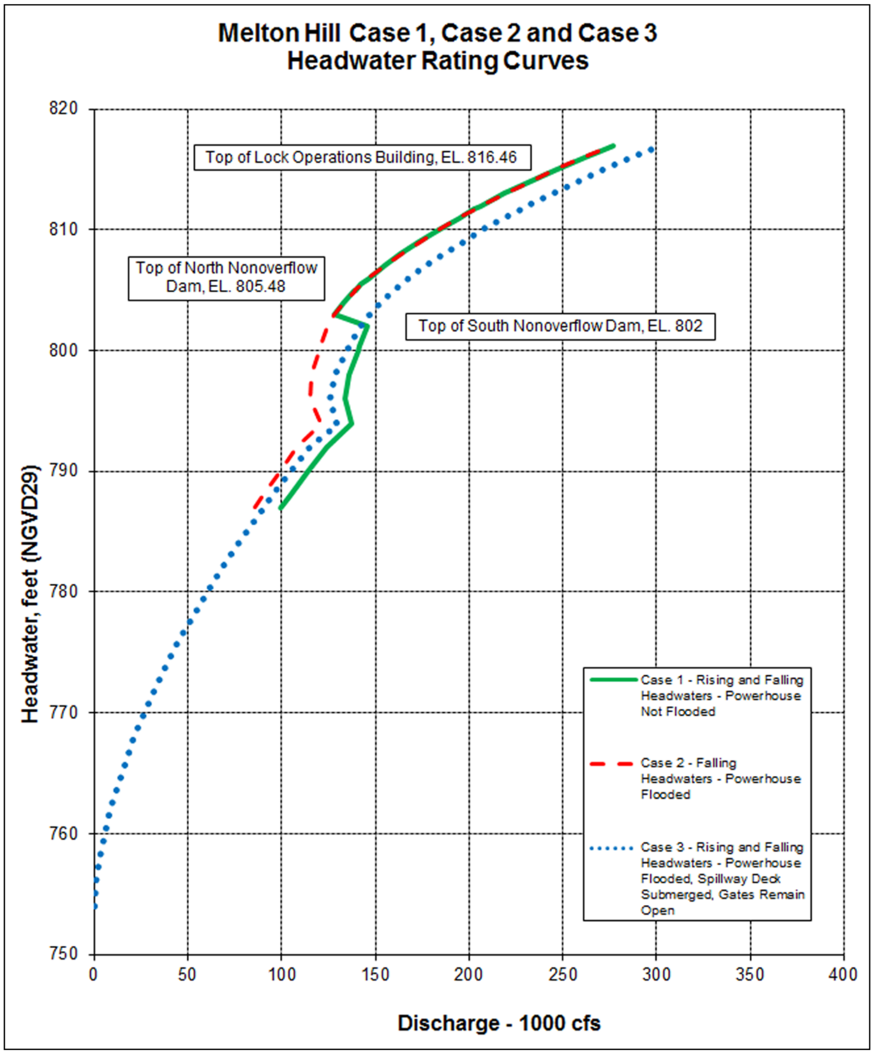


Figure 2.4.3-8. Dam Rating Curve – Melton Hill Dam

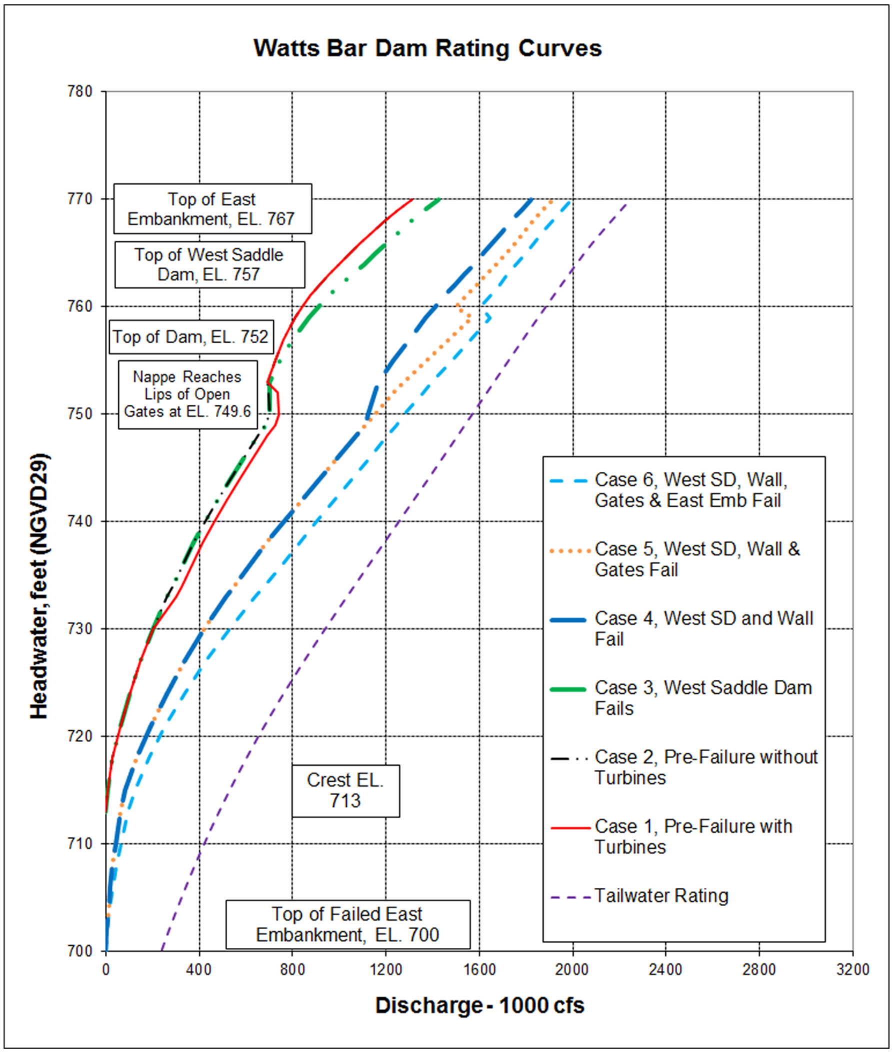


Figure 2.4.3-9. Dam Rating Curve – Watts Bar Dam