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

CNL-15-041

March 12, 2015

10 CFR 50.54(f)

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

Browns Ferry Nuclear Plant, Units 1, 2, and 3
Facility Operating License Nos. DPR-33, DPR-52, and DPR-68
NRC Docket Nos. 50-259, 50-260, and 50-296

Subject: **Flood Hazard Reevaluation Report for Browns Ferry Nuclear Plant, Response to NRC Request for Information Pursuant to Title 10 of the *Code of Federal Regulations* 50.54(f) Regarding Recommendations 2.1, 2.3 and 9.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident**

References:

1. NRC Letter, "Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," dated March 12, 2012 (ML12053A340)

2. NRC Letter, "Prioritization of Response Due Dates for Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Flooding Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," dated May 11, 2012 (ML12097A509)

3. Letter from NRC to NEI, "Trigger Conditions for Performing an Integrated Assessment and Due Date for Response," dated December 3, 2012 (ML12326A912)

4. Letter from TVA to NRC, "Tennessee Valley Authority (TVA) - Fleet Response to NRC Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Flooding Walkdown Results of Recommendation 2.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," dated November 27, 2012 (ML12335A340)

5. Letter from TVA to NRC, "Extension Request Regarding the Flooding Hazard Reevaluation Report Required by NRC Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendation 2.1, Flooding, of the Near-Term Task Force Review of Insights from the Fukushima Dai- ichi Accident," dated March 11, 2014 (ML14071A094)

6. Letter from NRC to TVA, " Browns Ferry Nuclear Plant Units 1, 2, and 3 - Relaxation of Response Due Dates Regarding the Flooding Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident (TAC Nos. MF3620, MF3621, and MF3622)," dated May 9, 2014 (ML14115A287)

On March 12, 2012, the NRC issued Reference 1 to all power reactor licensees and holders of construction permits in active or deferred status. Enclosure 2 of Reference 1 requested that each licensee perform a reevaluation of external flooding sources and report the results in accordance with the NRC's prioritization plan (Reference 2).

The report due date established for Browns Ferry Nuclear Plant (BFN), Units 1, 2, and 3, was March 12, 2014. Due to the amount of time required to complete evaluation of upstream and downstream dams to fully assess the flood hazard for the BFN site, BFN submitted Reference 5 requesting an extension of the due date from March 12, 2014 to March 12, 2015. The NRC authorized the due date extension in Reference 6.

The purpose of this letter is to provide the Flooding Hazard Reevaluation Report (HRR) for BFN, Units 1, 2, and 3. Specifically, Enclosure 1 of this letter provides the BFN Flooding HRR. The enclosed flooding HRR describes the approach, methods and results from the reevaluation of flood hazards at BFN, Units 1, 2, and 3. The eight flood-causing mechanisms, and a combined effect flood identified in Attachment 1 to Enclosure 2 of Reference 1, are described in the report along with the potential effects on BFN, Units 1, 2, and 3.

The Flooding HRR shows that some flood levels exceed the Current Licensing Basis (CLB) levels. The increased levels are the results of newer methodologies and guidance which typically exceed the methodologies and guidance which were used to establish the CLB for existing plants.

In accordance with Reference 3, an Integrated Assessment (IA) is required if flood levels determined during the flood hazard reevaluation are not bounded by the CLB flood levels. Enclosure 2 of Reference 1 specifies that the IA be completed and a report submitted within two years of submitting the Flooding HRR. An IA will be completed and a report submitted no later than March 12, 2017.

As discussed in Reference 1, the NRC stated that the current regulatory approach and the resultant plant capabilities, gave the NRC the confidence to conclude that an accident with consequences similar to the Fukushima accident is unlikely to occur in the United States. The NRC concluded that continued plant operations and the continuation of licensing activities did not pose an imminent risk to public health and safety. The flooding walkdowns for BFN, Units 1, 2, and 3 CLB flood protection features have been performed and the results were provided in Reference 4. The flood walkdowns verified that the flood protection systems for BFN, Units 1, 2, and 3, are available, functional and implementable and any degraded or nonconforming flood protection features were entered in TVA's Corrective Action Program.

Section 12 of the Flooding HRR provides a discussion of the interim actions taken or planned to address the higher flooding levels relative to the CLB flood levels. These actions will enhance the current capability to maintain the plant in a safe condition during the beyond-design-basis external flooding events that exceed the CLB flood levels, and, as a result, continued plant operation does not impose an imminent risk to the public health and safety while completing the IA.

In parallel with development of the required IA for BFN, TVA is continuing with several additional actions associated with understanding and mitigating potential flood hazards. Specifically, TVA is developing updates to site specific precipitation data using advanced techniques and may engage in further dialogue with the NRC in the future.

Enclosure 2 of this letter provides a list of new regulatory commitments.

If you have any questions regarding this submittal, please contact Beth Wetzel at (423) 751-2403.

I declare under penalty of perjury that the foregoing is true and correct. Executed on the 12th day of March 2015.

Respectfully,

for J W Shea

J. W. Shea
Vice President, Nuclear Licensing

Enclosures

cc: See Page 4

Enclosures:

1. Near-Term Task Force (NTTF) - Recommendation 2.1 Mitigating Strategies Flood Hazard Evaluation Report for Browns Ferry Nuclear Plant

2. List of Commitments

cc (Enclosures):

NRR Director - NRC Headquarters
NRO Director - NRC Headquarters
NRR JLD Director - NRC Headquarters
NRC Regional Administrator - Region II
NRC Project Manager - Browns Ferry Nuclear Plant
NRC Senior Resident Inspector - Browns Ferry Nuclear Plant

ENCLOSURE 1

NEAR-TERM TASK FORCE (NTTF) - RECOMMENDATION 2.1
MITIGATING STRATEGIES FLOOD HAZARD EVALUATION REPORT
FOR BROWNS FERRY NUCLEAR PLANT

NEAR-TERM TASK FORCE (NTTF) – RECOMMENDATION 2.1 MITIGATING STRATEGIES FLOOD HAZARD EVALUATION REPORT

Response to United States Nuclear Regulatory Commission (USNRC) – Code of Federal Regulations 10 CFR Part 50, Section 50.54 (f)

Browns Ferry Nuclear Plant
Tennessee Valley Authority

March 12, 2015

Revision 1

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FIGURES

TABLES

1 PURPOSE

In response to the accident at the Fukushima Dai-ichi nuclear power plant resulting from the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami, the United States Nuclear Regulatory Commission (NRC) established the Near Term Task Force (NTTF) to conduct a systematic and methodical review of NRC processes and regulations, and to make recommendations to the Commission for its policy direction. The NTTF reported a set of recommendations that were intended to clarify and strengthen the regulatory framework for protection against natural phenomena.

On March 12, 2012, the NRC issued an information request pursuant to Title 10 of the Code of Federal Regulations, Section 50.54(f) herein after referred to as the 50.54(f) letter which included six enclosures. (Reference 1)

In Enclosure 2 of the 50.54(f) letter, the NRC requests that licensees reevaluate the flooding hazards at their sites. (Reference 1)

This report provides the information for Browns Ferry Nuclear Plant (BFN) requested in the 50.54(f) letter; specifically, the information listed under the Requested Information section of Enclosure 2, paragraph 1 (a through e). The Requested Information section of Enclosure 2, paragraph 2 (a through d), Integrated Assessment Report, will be addressed separately.

1.1 Requested Actions

Per Enclosure 2 of the 50.54(f) letter, addressees are requested to perform a reevaluation of all appropriate external flooding sources. The reevaluation applies present-day methodologies and regulatory guidance, supplemented with interim staff guidance developed for review of the reevaluations. This includes current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The requested information is gathered in Phase 1 of the NRC staff's two phase process to implement Recommendation 2.1, and is used to identify potential vulnerabilities.

For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard. Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them.

1.2 Requested Information

Per Enclosure 2 of the 50.54(f) letter, the final Hazard Reevaluation Report should document results as well as pertinent site information and detailed analysis including the following:

a. Site information related to the flood hazard, including relevant structures, systems, and components (SSCs) important to safety and the ultimate heat sink (UHS). Pertinent data concerning SSCs and the UHS should be included. Other relevant site data includes the following:

   1. Detailed site information (both designed and as-built), including present-day site layout, elevation of pertinent SSCs important to safety, site topography, and pertinent spatial and temporal data sets;

   2. Current design basis flood elevations for all flood causing mechanisms;

   3. Flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance;

4. Changes to the watershed and local area since license issuance;
5. Current licensing basis flood protection and pertinent flood mitigation features at the site;
6. Additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results, etc.).

b. Evaluation of the flood hazard for each flood causing mechanism, based on present-day methodologies and regulatory guidance including an analysis of local intense precipitation (LIP) and site drainage, flooding in streams and rivers, dam breaches and failures, storm surge and seiche, tsunami, ice-induced flooding, channel migration or diversion, sediment transport, and combined effects. Mechanisms that are not applicable at the site may be excluded with appropriate justification. Evaluation will provide a basis for inputs and assumptions; methodologies; and models used.

c. Comparison of current and reevaluated flood causing mechanisms at the site. Provide an assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood causing mechanism. Include how the findings from Enclosure 4 of the 50.54(f) letter (i.e., Recommendation 2.3 flooding walkdowns) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, include how this finding was determined.

d. Interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment described below, if necessary.

e. Additional actions beyond Requested Information item 1.d taken or planned to address flooding hazards, if any. (Reference 1)

2 BACKGROUND

As shown in Figure 2-1, the Browns Ferry Nuclear Plant is located on the north bank of Wheeler reservoir at Tennessee River Mile (TRM) 294.0 with the lowest natural ground elevation in the site vicinity of about 560.0 ft. The Tennessee River above BFN site drains 27,130 square-miles. Guntersville Dam, 55 miles upstream, has a drainage area of 24,450 square-miles. Wheeler Dam, the next dam downstream, has a drainage area of 29,590 square-miles. The watershed is about 60 percent forested with much of the mountainous area being 100 percent forested. (Reference 2)

There are 22 major dams (Apalachia, Blue Ridge, Hiwassee, Chatuge, Nottely, South Holston, Boone, Fort Patrick Henry, Watauga, Wilbur, Fontana, Norris, Cherokee, Douglas, Tellico, Fort Loudoun, Melton Hill, Watts Bar, Chickamauga, Nickajack, Guntersville, and Tims Ford) in the TVA system upstream from Wheeler Dam, eleven of which provide substantial reserved flood-detention capacity during the main flood season. Figure 2-1 presents a simplified flow diagram for the Tennessee River system. In addition, there are six major dams not owned by TVA (Mission (not shown), Calderwood, Chilhowee, Santeetlah, Cheoah, and Nantahala Dams). These reservoirs often contribute to flood reduction, but they do not have dependable reserved flood detention capacity. (Reference 2)

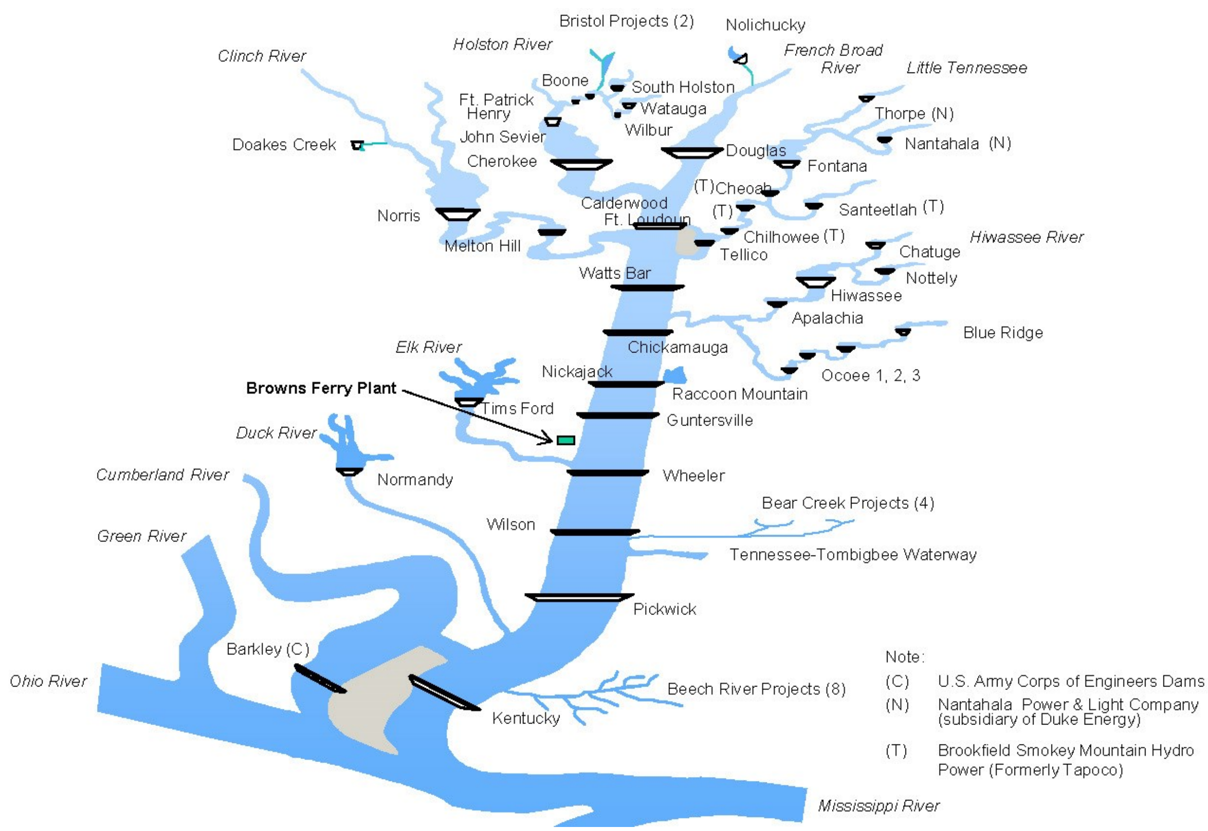

Figure 2-1 River System Schematic

3 PLANT SITE DESCRIPTION

3.1 Current Site Layout

The BFN is located in Limestone County, Alabama on the north bank of Wheeler Reservoir at TRM 294.0 with lowest site grade elevation at 560.0 ft. The site is approximately 30 miles west of Huntsville. The location of BFN is shown in Figure 3-1 and Figure 3-2. Details of the current site layout and plant structures are shown in Figure 3-3. (Reference 2)

3.2 Site Topography

The Browns Ferry Nuclear plant site comprises approximately 840 acres on the north bank of Wheeler Reservoir. As shown in Figure 3-4, the site is on high ground with the Tennessee River being the major potential source of flooding. (Reference 2)

3.3 Bathymetry in Vicinity of Plant – Tennessee River

Between April 2007 and June 2008, the U.S. Army Corps of Engineers (USACE) conducted a hydrographic survey of Wheeler Reservoir. This survey includes only the part of the reservoir that was below the water-surface at the time of the survey and was focused primarily on the navigation channel. A Triangular Irregular Network (TIN) was constructed from the USACE Hydrographic Survey data. Using this TIN, cross-sections were cut at various locations on the Tennessee River in Wheeler Reservoir in the vicinity of BFN, as shown in Figure 3-5. The actual cross-sections taken at these locations are shown in Figure 3-6. As shown, the depth of the river in the vicinity of BFN ranges between 7 and 36 feet at normal summer pool elevation, 556.0 ft. (Reference 3)

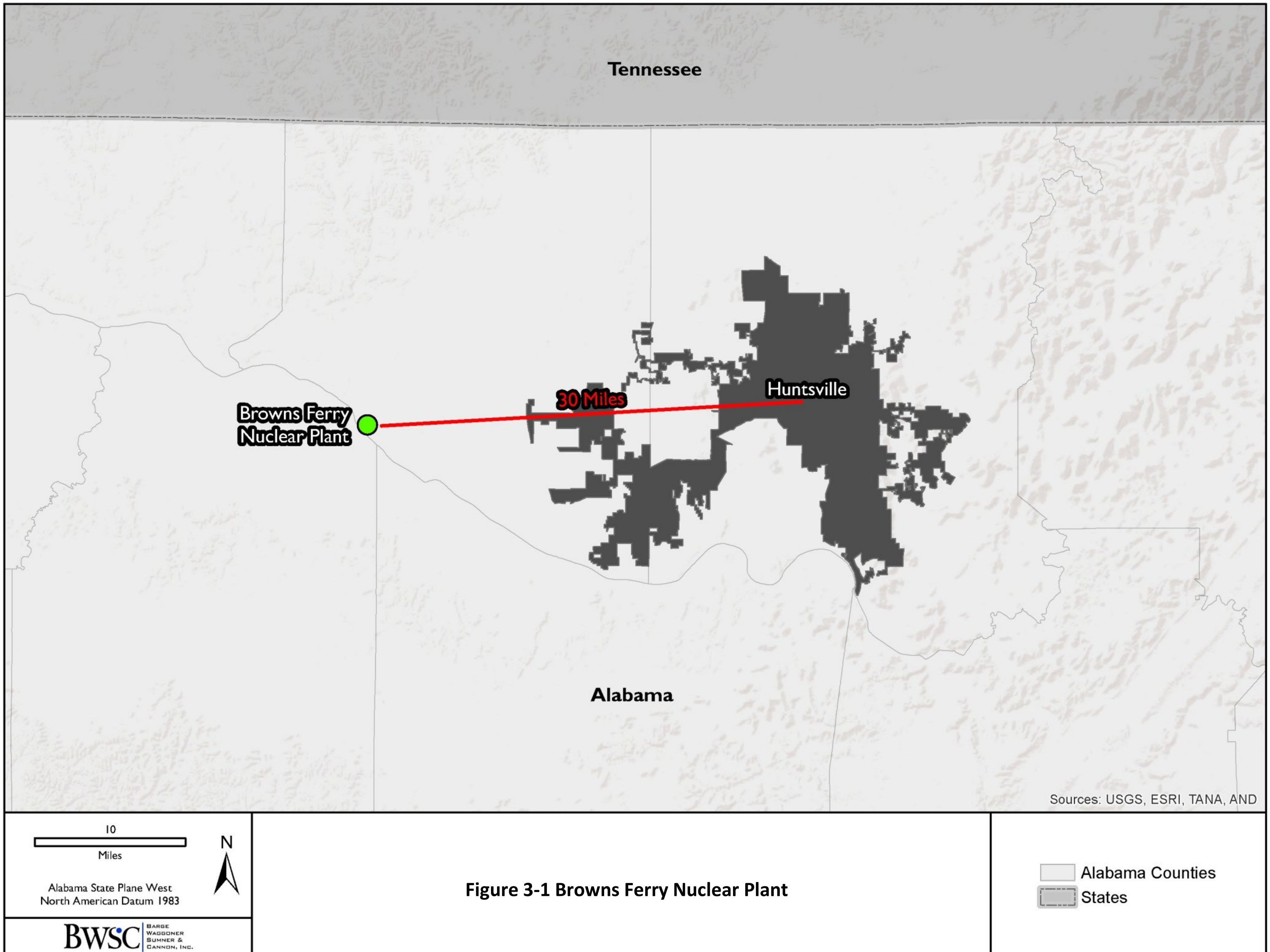

Figure 3-1 Browns Ferry Nuclear Plant

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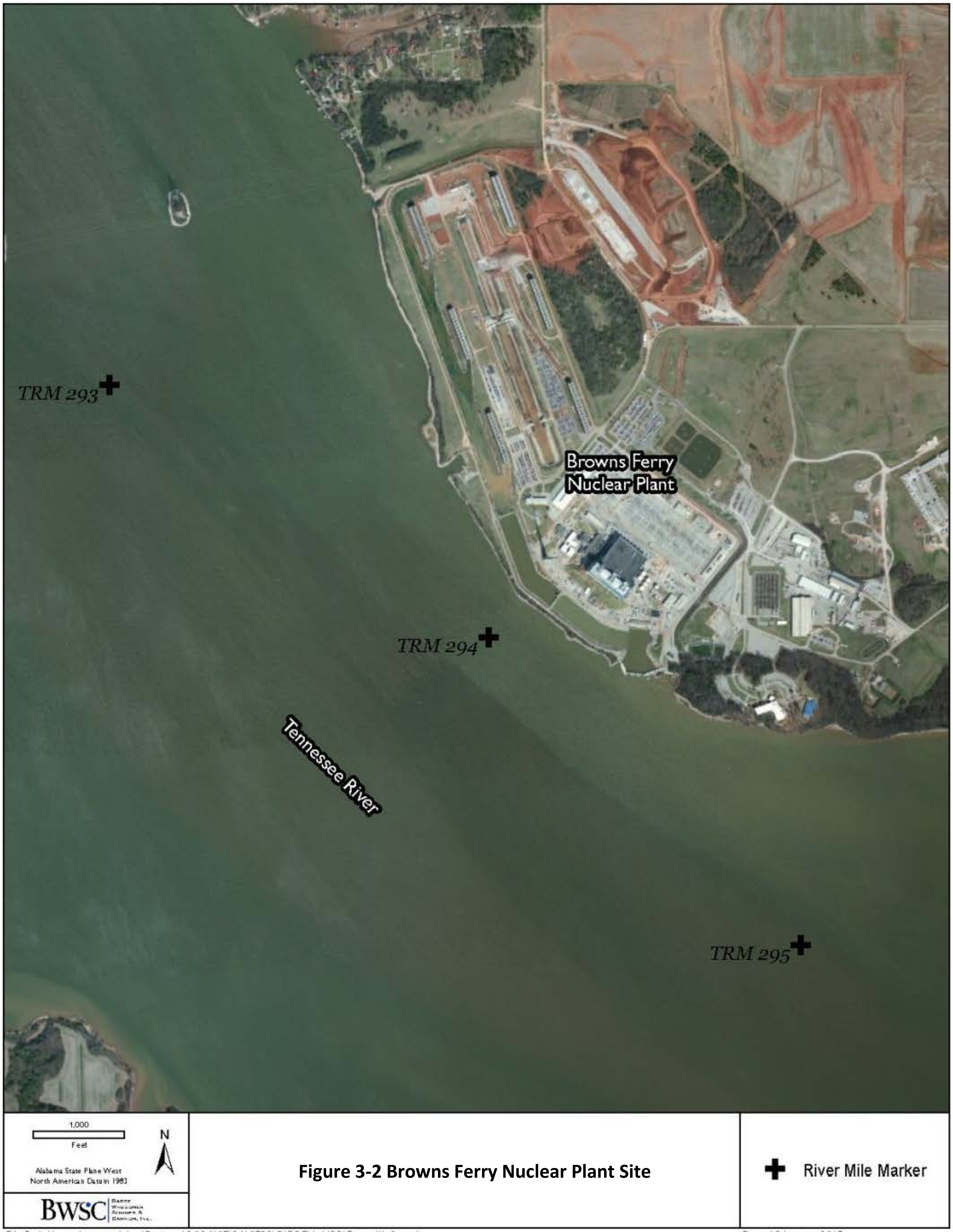

Figure 3-2 Browns Ferry Nuclear Plant Site

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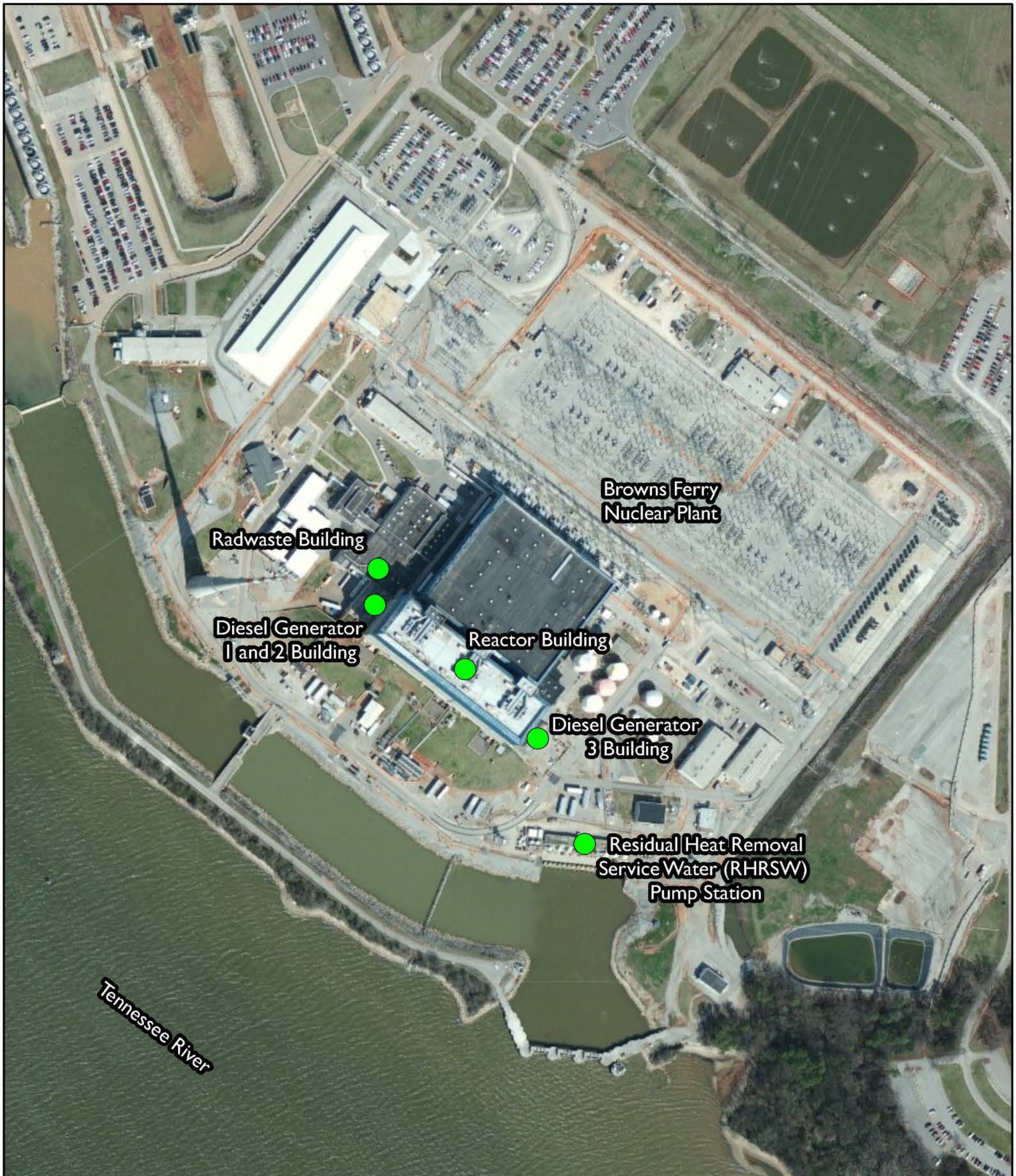

Figure 3-3 Browns Ferry Nuclear Plant Site Layout

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Figure 3-4 Browns Ferry Nuclear Plant Topography

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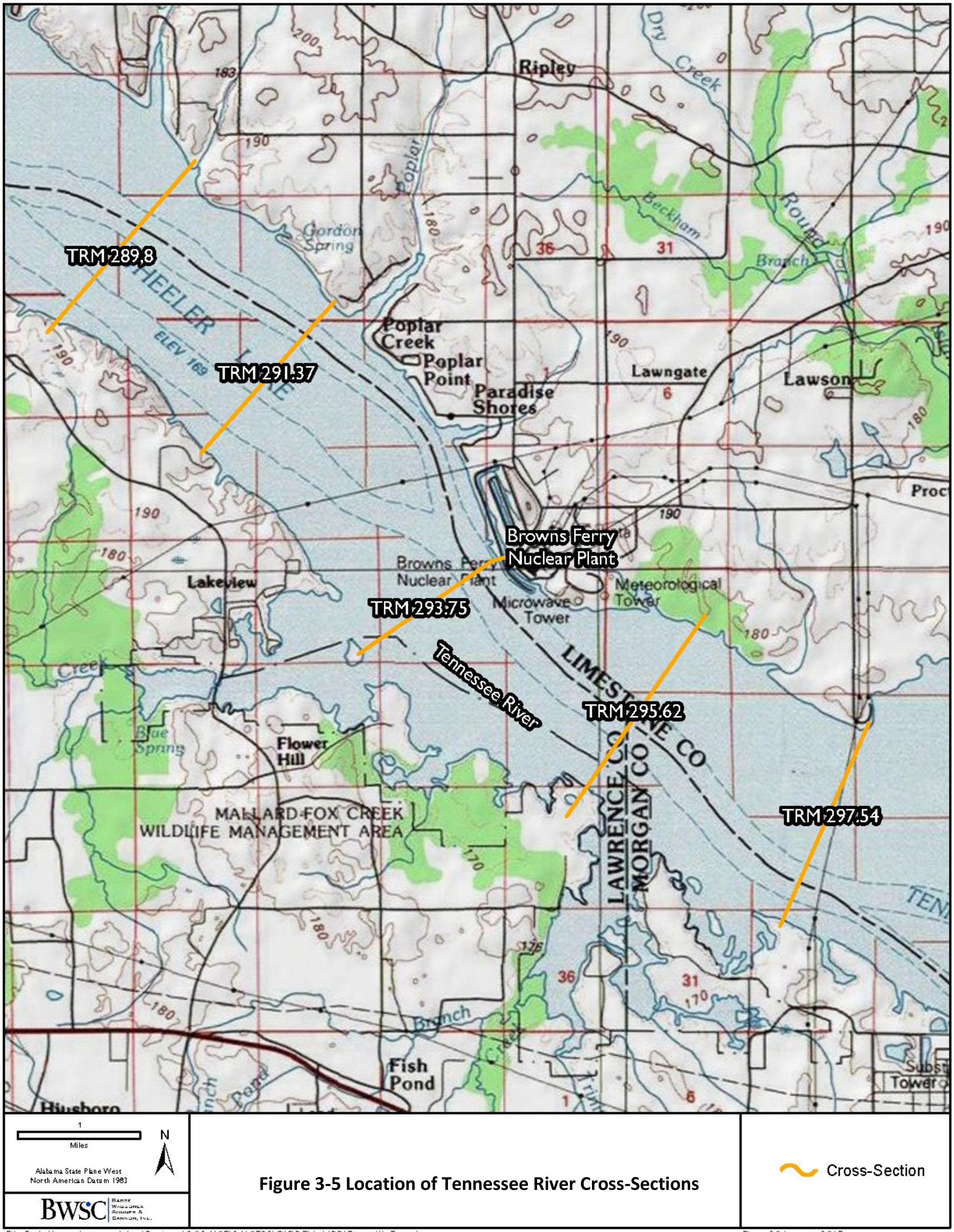

Figure 3-5 Location of Tennessee River Cross-Sections

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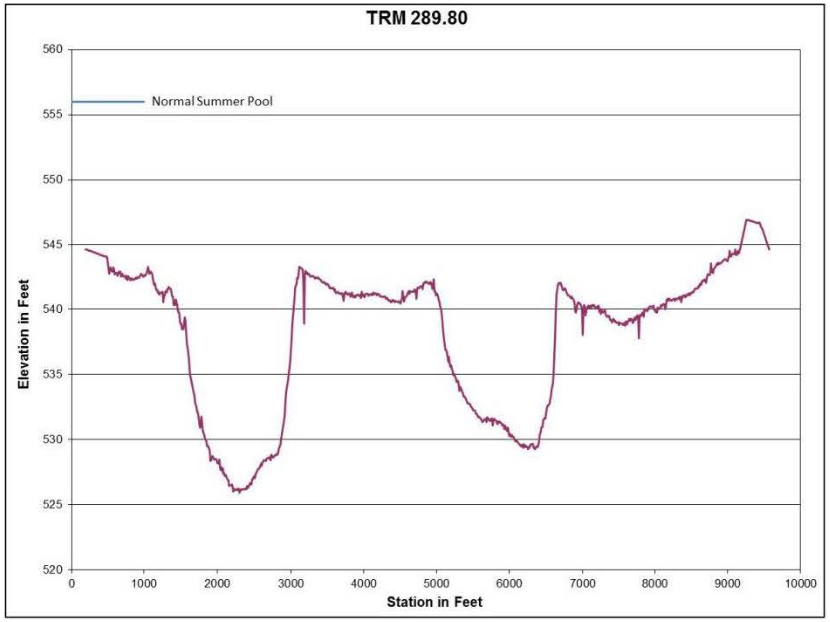
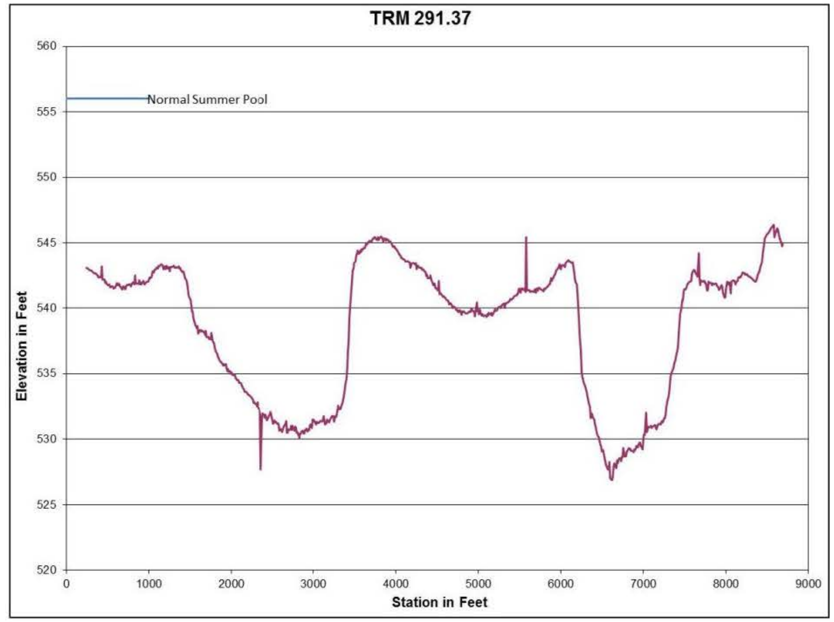
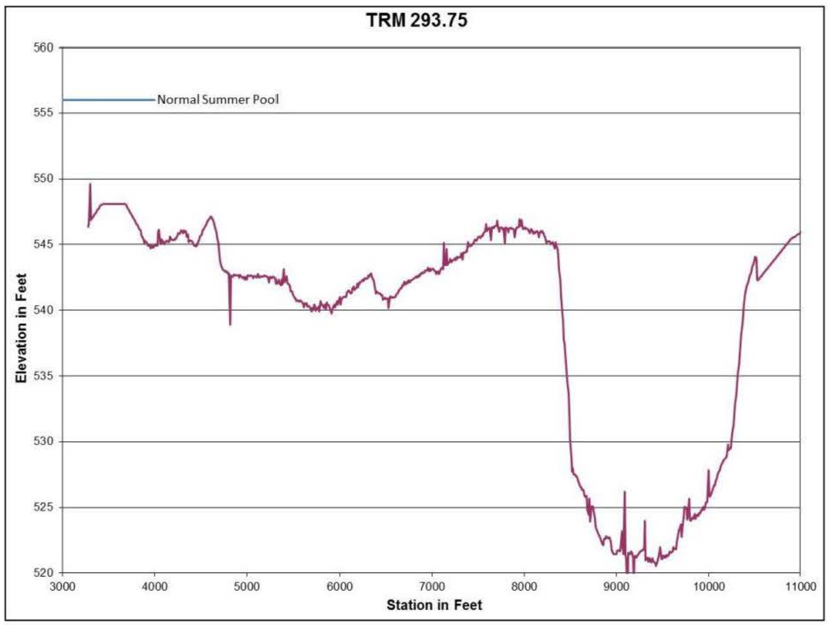


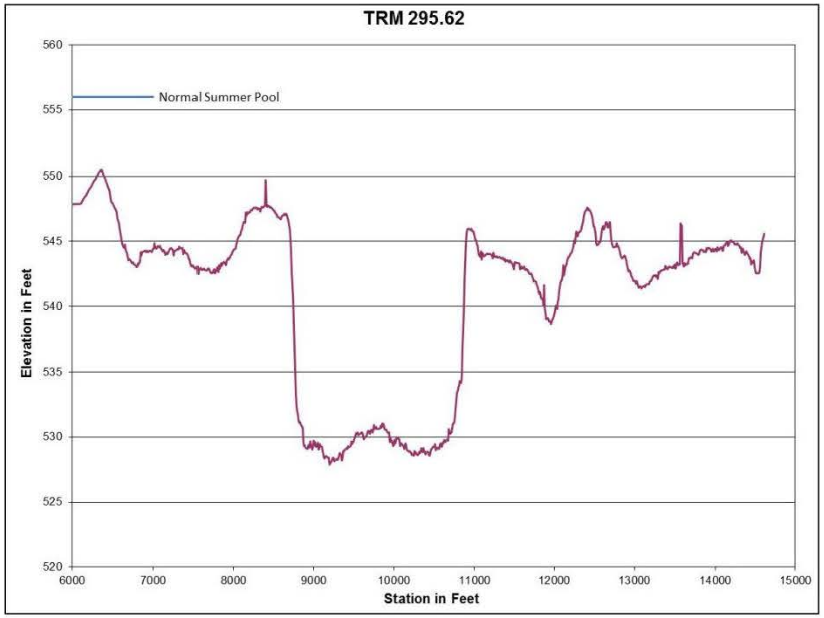


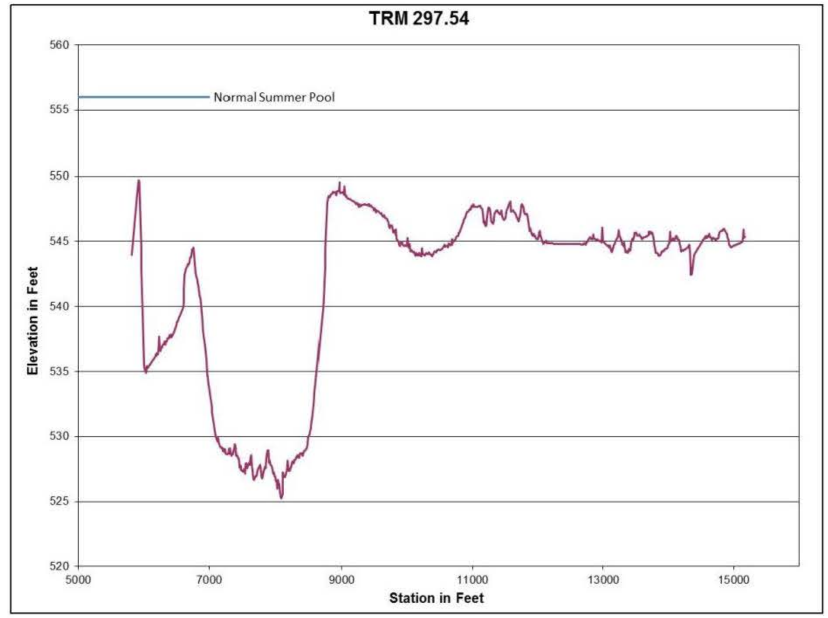


Bathymetry

Tennessee River in the Vicinity of Browns Ferry Nuclear Plant (Sheet 1 of 2)

Figure 3-6 Bathymetry – Tennessee River in the Vicinity of the Browns Ferry Nuclear Plant





Bathymetry

Tennessee River in the Vicinity of
Browns Ferry Nuclear Plant
(Sheet 2 of 2)

Figure 3-6 Bathymetry – Tennessee River in the Vicinity of the Browns Ferry Nuclear Plant

3.4 Current Design Basis Flood Elevations

On November 21, 2012, Browns Ferry SAR Change Package 25-044 (Reference 4) revised the Browns Ferry Living Final Safety Analysis Report (FSAR) (Reference 5) to supplement and clarify the current licensing basis (CLB). The calculated probable maximum flood (PMF) elevation resulting from the revised hydrologic analysis was 571.7 ft. However, the design and licensing basis was maintained as 572.5 ft. providing 0.8 feet of margin. This SAR Change Package was included in Amendment 25 FSAR and is the CLB for Browns Ferry.

The evaluation of LIP was not revised in the SAR change package 25-044.

3.4.1 Local Intense Precipitation

The effects of LIP are evaluated in the FSAR Appendix 2.4A. For the probable maximum precipitation (PMP) applied to the site, Hydrometeorological Report 56 (HMR 56) (Reference 6) is used to define the storm event. The design-basis storm is six hours long. In the analysis of the LIP, the underground drains were assumed clogged and runoff was assumed to equal rainfall.

The LIP evaluation sub-divides the site into four areas of analysis: (1) the western channel which diverts the inflow from an unnamed stream northwest of the plant to the Tennessee River; (2) the eastern switchyard drainage channel which diverts flow draining from a 100-acre area northeast of the plant to the Tennessee River; (3) the lower plant area in the immediate vicinity of the plant and (4) the cooling tower discharge channels.

The computed maximum surface elevation in the western channel is below the ground, dike and road, protecting the plant site from flooding. The water surface in the eastern switchyard channel does not exceed the switchyard elevation of 578.0 ft. In the lower plant area, the water surface elevations in the vicinity of the radioactive waste, reactor and diesel generator buildings do not exceed 565.0 ft. The cooling tower discharge channels have capacity to divert the run-off from a 179-acre drainage area and condenser water to the Tennessee River without flooding of the plant site. (Reference 2)

3.4.2 Flooding from Rivers and Streams

Two basic storms have the potential to produce a maximum flood at the Browns Ferry site; (1) maximum rainfall on the 21,400-square-mile watershed above Chattanooga with the downstream storm pattern as defined in HMR 41 (Reference 7) and (2) maximum rainfall on the 16,170-square-mile watershed above Wheeler Dam and below the major tributary dams as defined in HMR 47 (Reference 8).

Precipitation excesses for the storms are determined by the antecedent precipitation index (API) and geographical location. Sixty-two unit hydrographs, representing the total watershed for the Guntersville-Wheeler Reservoir, are used to determine outflows from each basin. These flows are combined with appropriate time sequencing to compute inflows into the most upstream reservoirs which in turn are routed through the reservoirs using standard techniques. The Simulated Open Channel Hydraulics (SOCH) Model is used for flood routing calculations. The SOCH suite of codes includes TRBROUTE, CONVEY, WWIDTH, and SOCH. Resulting reservoir outflows are combined with the additional local inflows and continued downstream using appropriate time sequencing or routing procedures. The main river reservoirs are routed using unsteady flow techniques.

The critical storm is the 21,400 square-mile downstream centered storm which follows an antecedent storm commencing on March 15. The antecedent storm produces an average precipitation of 6.08 inches on the basins above Wheeler, is followed by a 3-day dry period, and then by the main storm producing an average precipitation of 14.48 inches in 3 days. The licensing basis flood elevation for this storm is elevation 572.5 ft.

3.4.3 Flooding from Dam Breaches or Failures

Breaching failure of the Watts Bar West Saddle Dam, Nickajack, Guntersville and Chickamauga earthen embankments is evaluated in the CLB analysis for the PMF. The CLB flood elevation associated with this event is provided in the evaluation of the streams and rivers flooding hazard, Section 3.4.2.

The potential for breaching failure of the Cherokee, Fort Loudoun, Tellico and Watts Bar earthen embankments was identified in the revised PMF analysis as documented in Amendment 25 of the Browns Ferry UFSAR (Reference 2). To prevent failure of the earthen embankments at these four dams, HESCO® barriers were installed temporarily to prevent overtopping and failure of these earthen embankment dams.

Breaching failure of Wheeler dam was considered in the CLB analysis for loss of ultimate heat sink during non-flood conditions. This loss of downstream dam failure does not present a BFN flooding elevation hazard and is not considered in the CLB.

3.4.4 Flooding from Storm Surge and Seiche

Surges and seiches are not considered applicable in the BFN CLB because of the size and configuration of the lake.

3.4.5 Flooding from Tsunami

Tsunami is not considered applicable in the BFN CLB because of the inland location of the plant.

3.4.6 Flooding from Ice-Induced Events

Ice-induced flooding is not considered applicable in the BFN CLB because of the temperate zone location of the plant.

3.4.7 Channel Migration or Diversion

Channel diversion is not considered applicable in the BFN CLB because the configuration of the flood plain would not produce major channel meanders or cutoffs.

3.4.8 Flooding from Combined Effects

As described in Section 3.4.3, the current licensing basis is defined for BFN in Browns Ferry UFSAR (Reference 2).

3.4.8.1 Floods Caused by Precipitation Events

The revised PMF elevation provided above was combined with a 45 mph wind (Reference 2) which generates a 5-foot wind wave (crest to trough). The combined effects of the flood plus wind are provided in Table 3-1for BFN.

Table 3-1 Combined Effects of Flood and Wind

Plant Location	Design Basis Flood (DBF) Elevation (ft.)
Probable Maximum Flood (still reservoir)	572.5
Run-up on critical structure vertical wall	578.0

3.4.8.2 Floods Caused by Seismic Dam Failure Events

Floods caused by seismic dam failures are not considered in the CLB.

3.5 Current Flood Protection and Mitigation Features

Flood protection and mitigation for the BFN site are provided by three key elements: dams upstream of BFN, structures, structural features at the BFN site protecting equipment from the effects of flooding above plant grade, and operational procedures used when flooding exceeds plant grade. Each of these elements is described below.

3.5.1 Dam and Reservoir System

Flood control above the plant is provided largely by tributary reservoirs. On March 15, near the end of the flood season, these reservoirs provide a minimum of 4,484,000 acre-feet of storage. This is approximately 82 percent of the total storage available above the plant. The four main river reservoirs: Fort Loudoun, Watts Bar, Chickamauga, and Guntersville; provide 997,400 acre-feet of storage. The flood detention capacity reserved in the TVA system varies seasonally, with the greatest amounts during the January through March flood season.

Wheeler Dam, the headwater elevation of which affects flood elevations at the plant, has a drainage area of 29,590 square miles, 5,140 square miles more than Guntersville Dam. There is one major tributary dam, Tims Ford Dam, in the 5,140- square-mile intervening watershed. On March 15, near the end of the flood season, this project provides a minimum of 167,000 acre-feet of detention capacity. Wheeler Dam contains 326,500 acre-feet of detention capacity on March 15.

3.5.2 Browns Ferry Nuclear Site Protective Structures

The BFN protection and mitigation features that are considered in the licensing basis evaluation to protect against external ingress of flood water are described in the BFN Units 1, 2, and 3 UFSAR. The design loads to be considered for safety-related structures are described taking into account a PMF of elevation 572.5 ft. plus wind waves where applicable. The structures that are considered in the licensing basis evaluation to protect against external ingress of flood water are further described below as presented in the BFN UFSAR.

3.5.2.1 Flooding Protection Design Features

3.5.2.1.1 Reactor Building

The Reactor Building concrete structure exterior wall adjacent to Turbine Building is designed for allowable stresses resulting from a PMF load of elevation 572.5 ft. (Reference 2). The Reactor Building wall at the Turbine Building interface is an interior wall and not subjected to wind and wave action. Entrances to the Reactor Building are protected by appropriately designed sealed doors.

The equipment access flood gate is located on the outside face of the equipment access lock and is part of the Reactor Building flood protection for the PMF (Reference 2). The gate will normally be in the open position for access into or from the equipment access lock, but may be lowered either mechanically or manually in the event of impending high water. The gate consists of a structural steel frame with a solid steel skin plate on one side. Rubber seals provide sealing to elevation 578.0 ft. This gate provides adequate protection against flooding of the Reactor Building.

The Reactor Building watertight personnel access door is located at the south (outside) end of the personnel corridor, which is on the east side of the equipment access lock (Reference 2). This door is a part of the flood protection for the Reactor Building from the PMF.

3.5.2.1.2 Radwaste Building

The Radwaste Building will not flood because the entrances are protected by appropriately designed sealed doors (Reference 2). The piping penetrations below flood level are sealed to exclude the water and withstand the water pressure. Thus, the Radwaste Building is adequately protected from the PMF.

3.5.2.1.3 Residual Heat Removal Service Water Intake Pumping Station Structures

The walls of the Residual Heat Removal Service Water Intake Pumping Station Structures from the deck and grade elevation 565.0 ft. to elevation 578.0 ft. are designed to protect the Residual Heat Removal Service Water (RHRSW) pumps from water and wave forces resulting from the PMF (Reference 2). Full hydrostatic heads measured from the reservoir surface are applied to the entire area of the structure. The deck was investigated for a flood to elevation 578.0 ft. creating maximum water pressure on the underside and no pressure on the top with the determination that the deck has a strength capability of 1.4 times that required to resist this flood condition.

The personnel access doors are normally closed and latched to provide watertight units. These personnel access doors provide adequate flood protection against the PMF for the RHRSW pumps (static water pressure to elevation 578.0 ft). (Reference 2)

3.5.2.1.4 Diesel Generator Buildings

The BFN Units 1 and 2 Diesel Generator Building and separately located Unit 3 Diesel Generator Building access doors provide adequate protection for the diesel-generator units. During plant operation, the doors are normally closed, latched, and locked (Reference 2). Rubber seals on the doors are in place and sealed anytime the doors are closed. Flood conditions consist of floods up to and including a PMF to elevation 572.5 ft., with wave run-up to elevation 578.0 ft.

3.5.2.1.5 Radwaste Evaporator Building

The Radwaste Evaporator Building is designed to be flood-proof for the PMF (water level elevation 572.5 ft.) plus waves (Reference 2). The construction interface between the structure and the radwaste building is sealed for flood protection to elevation 578.0 ft.

3.5.3 BFN Flood Response Procedures

Flood preparations for BFN, as defined in plant procedure 0-AOI-100-3 (Reference 9), begin with (1) notification by plant personnel that the forebay level is greater than or equal to elevation 558.0 ft., (2) notification by the Wilson Load Dispatcher (or other reliable sources) of impending river conditions, (3) annunciation in the unit 1 Control Room of Lake Elevation High (564 ft. setpoint), or (4) when water is detected in the corridor from the lunchroom to the Turbine Building due to PMP. If River Operations (RO) projects that the river level will rise above elevation 565.0 ft., shutdown of all three units (if not already shutdown) will begin and the flood preparation outlined in the flood response AOI will be executed.

Specific design features required for flood protection are addressed as necessary in the AOI, and include watertight doors, flood gates, and portable bulkheads. Additional design features addressed in the AOI include manholes, equipment hatches, valves, and drain plugs.

4 CHRONOLOGY OF FLOOD RELATED CHANGES SINCE LICENSING

4.1 Watershed Changes since Licensing

The potential impacts of land use change in the Tennessee River basin are evaluated using the National Land Cover Data (NLCD) to determine the change in impervious area over the watershed above Wheeler Dam. The Landsat data are acquired by satellite sensor at 30 meter resolution. The NLCD data have been used for many applications including national environmental reporting, climate change, Clean Water Act studies and conservation assessments. Using this product the land cover for the watershed was defined as shown in Figure 4-1. (Reference 10)

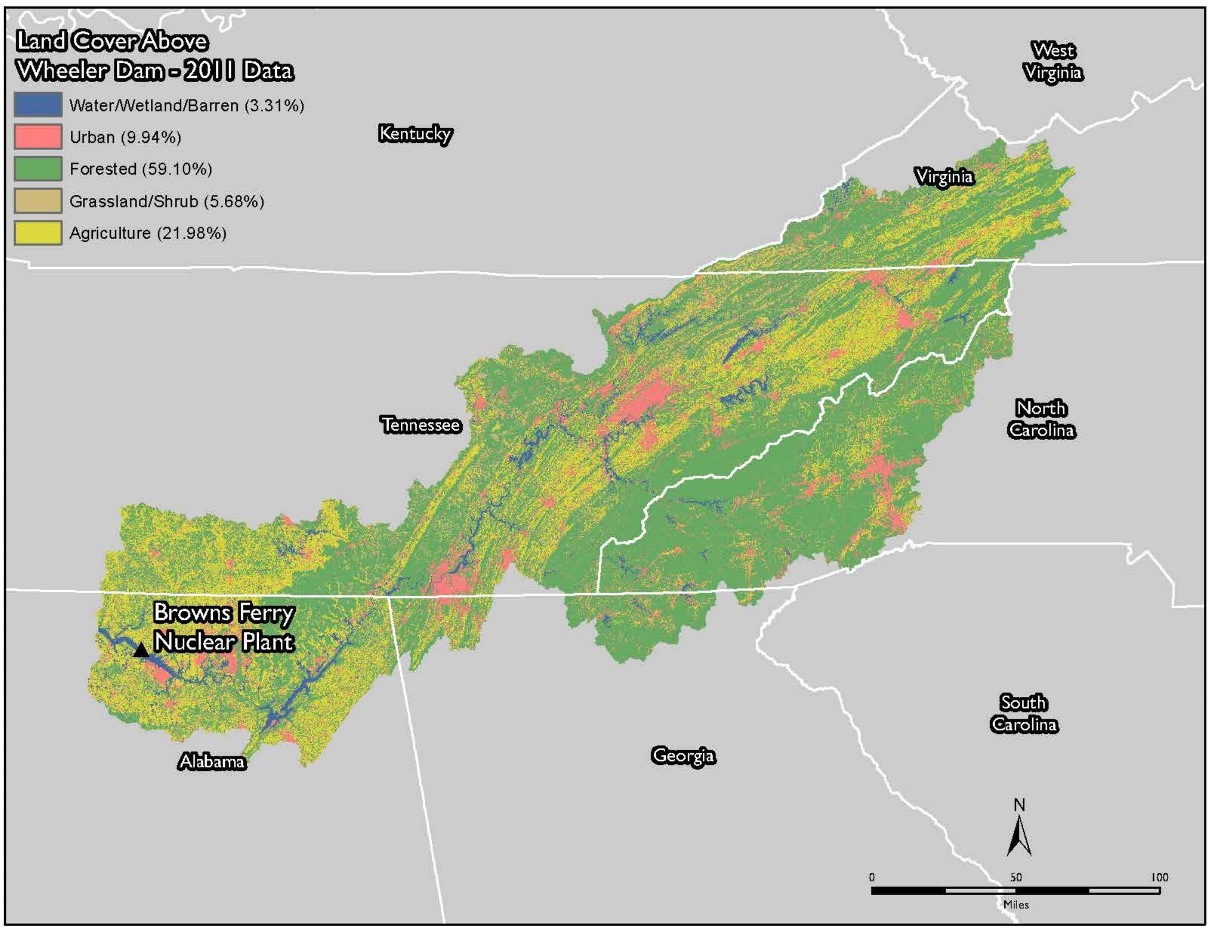


Figure 4-1 Land Cover for the Tennessee River Watershed above Wheeler Dam

The data for this urban land use change assessment are derived from the NLCD 2001 and 2011 Retrofit Change Project (Reference 10). The raster product from this project is converted to polygon vector data that could be displayed, manipulated, and analyzed in a geographic information system (GIS) to determine change in impervious area (land use) between 2001 and 2011 as follows:

Table 4-1 Land Use above Wheeler (2001 – 2011)

Land Use Classification	2001	2011	Change
Agricultural	22.40%	21.98%	-0.42%
Forest	59.94%	59.10%	-0.84%
Urban	9.46%	9.94%	0.48%
Grassland/Shrub	4.93%	5.68%	0.75%
Water/Wetland/Barren	3.27%	3.31%	0.04%

Table 4-2 Impervious Area above Wheeler (2001 – 2011)

	2001	2011	Change
Impervious Area	1.74%	1.97%	0.23%

Based on this assessment the Tennessee Valley watershed is experiencing a change in impervious area at a rate of about 0.02% per year in a 10 year period. Thus it is judged at this rate that any potential impacts of watershed changes due to urbanization would have minimal impact on runoff from the basin over the life of the project. Additional lands owned by federal agencies, including US Fish and Wildlife Service, US Forestry Service, National Park Service and the Protected Areas Database comprise 29% of the entire watershed above Wheeler Dam. These lands are set aside for public use and include prohibitive development restrictions. Further, a computation of runoff coefficients using the land use data and soils data is presented in Reference 11. The results of the computed runoff coefficients show that there is good agreement with coefficients used in the flood hazard reevaluation. In general the computed runoff coefficients are lower than those used in this analysis and result in less runoff.

4.2 Summary of Changes to Design Basis Flood Elevations

The original licensing basis is documented in the 1972 Browns Ferry Nuclear Plant FSAR. Figure 4-2 provides a timeline for the changes that have occurred between the 1972 FSAR and this flooding reevaluation.

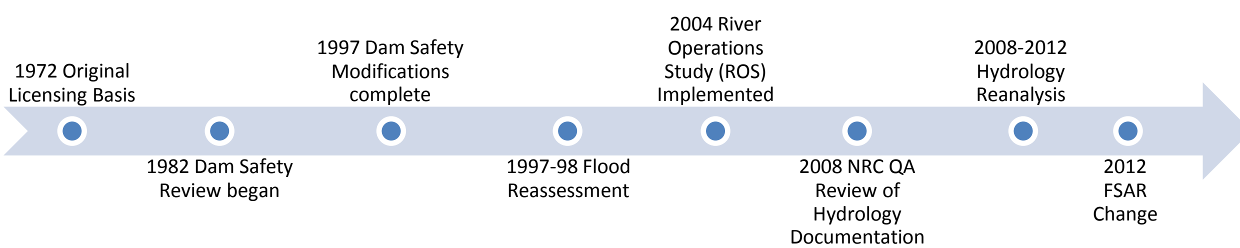


Figure 4-2 Timeline of Flood Related Changes Since Licensing

4.2.1 Local Intense Precipitation

The original licensing basis evaluation for the Browns Ferry Local Intense Precipitation (LIP) was established in Appendix 2.4A of the FSAR in 1972. This initial evaluation concluded the LIP flood levels would not exceed the 565.0 ft. critical elevation at safety-related plant structures. The original analysis also investigated a local stream northwest of the plant site which flowed into a channel along the western boundary of the plant site. The original analysis concluded that the western channel was sufficient to carry the maximum possible flood without flooding the plant. Subsequently, the LIP evaluation of the switchyard drainage channel was added to the FSAR. This evaluation concluded that the flood elevation at the north corner of the switchyard would not exceed 578.0 ft.

The LIP has been reevaluated throughout the life of the plant for site configuration changes, as necessary. The BFN CLB plant area LIP flood elevations have been maintained at or below the critical elevation of 565.0 ft. The flood elevation in the switchyard drainage channel has remained at or below the 578.0 ft. critical bank overflow elevation. The western channel has maintained the capacity to carry the maximum possible flood without flooding the plant site area. The cooling tower discharge channels have maintained the capacity to divert drainage area and condenser water to the Tennessee River without flooding of the plant site. (Reference 2)

4.2.2 Flooding in Rivers and Streams

4.2.2.1 1970s – Licensing Basis

4.2.2.1.1 PMF (Reference 12)

The original PMF analysis for BFN was completed in 1972. The maximum possible flood was determined from the maximum winter-type storms observed in eastern United States adjusted in transposition over the Browns Ferry Watershed. Lower elevations were computed in the 1970s analysis but not listed in this report. Later PMF storms were evaluated. In the original PMF analysis, overtopping and breaching failure of earthen embankments was assumed to occur at Fort Loudoun Dam and all downstream dams through Guntersville Dam. The PMF elevation in the original analysis was 572.5 ft.

4.2.2.1.2 Seismically Induced Failure of Upstream Dams

Seismically induced failure of upstream dams was not included for BFN as part of the original licensing basis.

4.2.2.2 1997 – 1998 Reassessment

4.2.2.2.1 PMF (Reference 13)

A reassessment of maximum flood levels was performed between 1997 and 1998. The reassessment of maximum flood levels was made to address dam safety modifications that had been made subsequent to the flood level determinations in the original licensing basis. Other inputs to the reanalysis were not changed.

In 1982 TVA established the Dam Safety Program and began a safety review of TVA dams. This dam safety effort was designed to be consistent with the Federal Guidelines for Dam Safety and similar efforts of other Federal agencies. Technical studies, engineering analyses, and modifications were performed to ensure hydrologic and seismic integrity of TVA dams. Table 4-3 provides the modification status (hydrologic) of the dam safety effort as of 1998. The reassessment addressed the effects of these dam safety modifications on maximum flood levels at BFN and on warning time available for safe plant shutdown.

The reassessment of the PMF involved evaluation of three candidate flood events:

1. March 7,980-square-mile
2. March 21,400-square-mile
3. March 12,030-square-mile

The March 7,980 square-mile and March 21,400 square-mile events were described in the original analysis. The March 12,030 square-mile event became a candidate on the lower main river after Fort Loudoun, Tellico, Watts Bar, Nickajack, and Guntersville dams were modified to prevent their failure in an extreme flood event. This new storm produces maximum rainfall on the 12,030 square-mile watershed above Pickwick Dam and below Chickamauga Dam.

As a result of the reassessment the controlling event at BFN would result from the March 12,030-square-mile event centered above Pickwick Dam and below Chickamauga Dam as shown below:

Storm Event	Maximum Discharge	Maximum Elevation
BFN (March 12,030-square-mile)	810,000 cfs	569.2 ft.

The BFN FSAR was not updated based on this reassessment and remained at 572.5 ft., providing a margin of 3.3 feet.

Table 4-3 Dam Modifications Completed by 1998 (Reference 13)

Dam	Dam Modification	Year Modifications Completed
Main River Dams Fort Loudoun-Tellico	Fort Loudoun Dam was raised 3.25 feet with a concrete wall to elevation 833.25 feet. A 2000-foot uncontrolled spillway with crest at elevation 817 feet was added at Tellico Dam.	1989
Watts Bar	Embankment was raised 10 feet with earth-fill/concrete wall to elevation 767 feet.	1997
Nickajack	South embankment was raised 5 feet with earth-fill/concrete wall to elevation 657 feet. A 1900-foot roller-compacted concrete overflow dam with top at elevation 634 feet was added below the north embankment.	1992
Guntersville	Embankment was raised 7.5 feet with earth-fill to elevation 617.5 feet.	1996
Tributary Dams Beech	Embankment was raised 4.5 feet with earth-fill to elevation 475.5 feet.	1992
Blue Ridge	Three (3) additional spillway bays were added in 1982. Embankment was raised 7 feet with earth-fill/concrete wall to elevation 1713 feet, and a 395-foot uncontrolled spillway with crest at elevation 1691 feet was added in 1995.	1995
Boone	Embankment was raised 8.5 feet with earth-fill to elevation 1408.5 feet.	1984
Cedar Creek	Embankment was raised 5.5 feet with concrete wall to elevation 605 feet.	1997
Chatuge	Embankment was raised 6.5 feet with earth-fill to elevation 1946.5 feet.	1986
Cherokee	A portion (600 feet) of the non-overflow dam was raised 7.75 feet to elevation 1089.75 feet.	1982
Douglas	A portion of the non-overflow dam was raised 13.5 feet to elevation 1022.5 feet and eight saddle dams were raised 6.5 feet with earth-fill to elevation 1023.5 feet.	1988
Nottely	Embankment was raised 13.5 feet with rock-fill to elevation 1807.5 feet.	1988
Upper Bear Creek	Embankment was raised 4 feet with concrete wall to elevation 817 feet.	1997
Watauga	Embankment was raised 10 feet with rock-fill to elevation 2012 feet.	1983
Fontana	Dam post-tensioned	1988
Melton Hill	Dam post-tensioned	1988

4.2.2.2.2 Seismically Induced Failure of Upstream Dams (Reference 13)

Seismically induced failure of upstream dams was not evaluated for BFN as part of the 1997-1998 Reassessment because it is not part of the licensing basis.

4.2.2.3 2012 Updated FSAR

On October 30, 2007, TVA submitted an application for a combined operating license (COLA) for the proposed Bellefonte Nuclear Plant (BLN) Units 3 and 4, in accordance with 10 CFR 52. During review of the BLN Units 3 and 4 FSAR, the NRC performed an audit of the hydrologic analysis which resulted in the issuance of three Notice of Violations (NOVs) on March 19, 2008 (Reference 14). In response to these NOVs, TVA completed a revised hydrologic analysis to support the BLN Units 3 and 4 COLA.

While the February 2008 QA inspection was for the BLN licensing request, it directly impacted BFN because the analysis is similar for TVA nuclear plants located along the Tennessee River. As a result of the NOV, TVA initiated a confirmatory analysis of the PMF computations and installed temporary flood barriers at Cherokee, Fort Loudoun, Tellico, and Watts Bar Dams to increase the height of the embankments. Increasing the height of the embankments at these dams prevents embankment overflow and failure during the PMF. (Reference 2)

4.2.2.3.1 PMF

As a result of the reassessment the controlling event at BFN changed from the previous analysis and results from the March 21,400-square-mile downstream-centered event as shown below:

Storm Event	Maximum Discharge	Maximum Elevation
March 21,400-square-mile	1,194,000 cfs	571.7 ft.

The design and licensing basis was maintained at elevation 572.5 ft. providing 0.8 feet of margin.

4.2.2.3.1.1 Summary of Differences between 1972 FSAR and 2012 FSAR Analyses for PMF

a. Dam safety modifications at tributary dams, Douglas and Watauga, eliminated overtopping and breach

b. Dam safety modifications at main river dams, Fort Loudoun – Tellico and Watts Bar as shown in Table 4.2 1, eliminated failure from overtopping

c. The only postulated embankment failures from rainfall floods that influenced plant site elevations were those at Chickamauga, Guntersville, Nickajack, and Watts Bar West Saddle Dam

d. An unsteady flow model was added for the Fort Loudoun – Tellico complex

e. Cross section bathymetry was updated based on recent USACE Doppler profiler navigation surveys

f. Dam operating guides were updated in the hydrology model to reflect current reservoir operating policy

g. Dam rating curves were updated to include discharge coefficients derived using model test data

h. Turbine discharges were used at all river reservoirs up to the point where the head differentials were too small and/or the powerhouse would flood

i. The unsteady flow model was extended to include Tellico Dam.

j. Model refinements were made at the Fort Loudoun-Tellico canal

k. The Dallas Bay rim leak on Chickamauga Reservoir was modeled as a reach with a junction with the Tennessee River and a downstream boundary at the overflow consisting of a rating curve

l. Operational Allowance approach was developed to allow flood simulations to more nearly mimic the integrated operation of the reservoir system

m. Correction for tailwater submergence was applied at dams as appropriate

n. West saddle dam failure discharge was input at the mouth of Yellow Creek where it would enter Chickamauga Reservoir instead of combining with Watts Bar Dam discharge and routing downstream

o. Height of embankments modified by use of temporary HESCO® barriers at Cherokee, Fort Loudoun, Tellico, and Watts Bar dams to prevent overtopping.

4.2.2.3.2 Seismically Induced Failure of Upstream Dams

Seismically induced failure of upstream dams was not evaluated for BFN as part of the 2012 Updated FSAR analysis because it is not part of the licensing basis.

5 SUMMARY OF PLANT WALKDOWN RESULTS

TVA completed flooding walkdowns in accordance with the NEI 12-07 walkdown guidelines. In Reference 15, TVA provided the results of the flooding walkdowns in response to Recommendation 2.3, Item 2 in Enclosure 4 of Reference 1.

The BFN external flood protection features were visually inspected. The NEI walkdown record forms included in Appendix B of the guidance document were used as a template for the inspections. Training was provided, as recommended in the guidance document and TVA procedure CTP-FWD-100.The walkdown team was made up of four engineers consisting of one senior mechanical engineer with knowledge of the design and operation of the Browns Ferry Nuclear Plant, one senior civil engineer with knowledge of hydrology and storm drainage, and two mechanical engineers with nuclear experience. The walkdown team was supported by a retired TVA SRO and active TVA auxiliary unit operators in planning and performance of the walkdowns.

Walkdowns were performed in the flood protected areas at BFN including the Reactor Building, Radwaste Building, Diesel Generator Buildings, and RHRSW Pump Rooms. Deficiencies identified from the walkdowns were entered in the BFN corrective action program.

As a result of plant walkdowns and disposition of other previous corrective actions related to flooding, BFN has implemented the following flood protection improvements:

a. The four watertight doors providing access to the RHRSW Pump Rooms have been replaced due to material deficiencies.

b. The Reactor Building south access portal watertight door frame cracked weld has been repaired.

c. The BFN flood operations procedure has been enhanced.

6 IDENTIFICATION OF POTENTIAL FLOOD CAUSING MECHANISMS

The sections that follow discuss previous and proposed analyses or provide justification for exclusion for each of the flood-causing mechanisms. The hierarchical hazard assessment approach recommended in NUREG/CR-7046 (Reference 16) is employed in these analyses. This approach allows a stepwise, progressively refined series of analyses that demonstrates that SSCs important to safety are protected from the adverse effects of severe flooding at the site.

Guidance, in addition to NUREG/CR-7046 (Reference 16), for potential flood-causing mechanisms, or causal phenomena, is provided in Table 6-1:

Table 6-1 Potential Flood Causing Mechanisms or Causal Phenomena

Flood Causing Mechanism	Guidance	Reference
Local intense precipitation	HMR 52 and HMR 56	17 & 6
Flooding from rivers and streams	ANSI/ANS-2.8-1992	18
Flooding from upstream dam breaches or failures	Dam Failure ISG	19
Flooding from storm surges or seiches	Not Applicable	NA
Flooding from tsunamis	Not Applicable	NA
Flooding from ice-induced events	Not Applicable	NA
Flooding from channel diversion or migration toward the site	Dam Failure ISG	19
Flooding from combined effects	ANSI/ANS-2.8-1992 and Dam Failure ISG	18 and 19

6.1 Local Intense Precipitation

The LIP was previously evaluated for BFN and is included in the reevaluation for BFN. The analysis assumes fully functional site grading, partially blocked drainage channels, and fully blocked storm drains.

6.2 Flooding from Rivers and Streams

The PMF was previously evaluated for BFN and is included in the reevaluation for BFN. The PMF in rivers and streams adjoining the site is determined by applying the PMP to the drainage basin of these rivers and streams. The model inputs and assumptions, as well as the previous analysis, technical approach, and results are described in subsequent sections.

6.3 Flooding from Dam Breaches or Failures

6.3.1 Project Specific PMF

The Project Specific PMF was not previously evaluated and is included in the reevaluation for BFN. The Project Specific PMF is the design basis flood level for a dam, and is defined as the most severe flood that may be reasonably predicted to occur at a site as a result of severe hydrometeorological conditions. Failures of upstream dams during their Project Specific PMF haves not previously been analyzed for BFN. The model inputs, assumptions, technical approach and results of this analysis are presented in subsequent sections.

6.3.2 Sunny Day Failure of Upstream Dams

Sunny day failure of upstream dams has not previously been analyzed for BFN. Inputs, assumptions, technical approach and results of this analysis are presented in subsequent sections.

6.3.3 Seismic Failure of Upstream Dams

Seismic failure of upstream dams was previously qualitatively evaluated and determined to be non-governing. The seismic failure of upstream dams is included in the reevaluation for BFN. Seismic failure of single dams combined with flood events is evaluated as part of this analysis. The inputs, assumptions, technical approach, and results are presented in subsequent sections. Seismic failure of upstream multiple dam combinations with coincident flood events are described in Section 9.4.2.

6.3.4 Sediment Transport

A sediment transport analysis was performed to determine the impact of sediment released from a hypothetical Guntersville Dam Breach (Reference 20). Sediment core samples of the Guntersville Dam embankment were examined. The embankment is composed of homogeneous earth fill made of a lean clay and sand with a riprap protective shell.

The incipient motion results for various flows between 200,000 and 400,000 cfs were assessed. The latter being the peak flow for the Sunny Day dam breach. Relatively large size particles (d > 0.5 mm) would move throughout the reach between Guntersville and Wheeler Dams for a 400,000 cfs event. During the peak flow, particles larger than 10 mm (fine to medium gravel) would move in the upper 40 miles of the reach down past TRM 310 (16 river miles upstream of BFN). As the dam breach hydrograph attenuates with time, smaller size particles have the possibility of depositing downstream of TRM 310.

These silts and clay particles would move through the system in suspension during the whole simulation, and sand size particles would move in suspension and settle-out at various locations during the simulation.

It was estimated that the amount of suspended sediment that would settle in the intake channel is minimal, at less than 0.01 inch. Sedimentation would not result in a loss of storage in the reservoir, which would lead to flooding, because of the small amount of settlement of suspended sediment (less than 1 inch).

6.4 Flooding from Storm Surge and Seiche

Flooding from storm surge and seiche has not previously been evaluated for BFN and is not considered a credible flood-causing mechanism at this site. The BFN site is located on the north bank of Wheeler Reservoir at TRM 294 approximately 1,300 River miles inland (Tennessee River Miles 294, Ohio River Miles 48, and Mississippi River Miles 953) from the Gulf of Mexico at elevation 565.0 ft. The Wheeler Reservoir level during non-flood conditions would not exceed approximate elevation 556.0 ft. at the plant, for any significant period of time. The plant elevation 565.0 ft. is nine feet above the normal maximum pool levels.

While seismic seiche has been recorded in the Tennessee Valley area it has been of very small magnitude. For example the March 1964 Alaska Earthquake which was a 9.2 magnitude event resulted in seiche being observed on about 25% of the 130 gages available in Tennessee at the time. The largest amplitude of seiche recorded on lakes, reservoirs, and/or ponds in Tennessee was 0.14 feet and in Kentucky 0.57 feet (Reference 21). Reference 22 indicates that BFN is outside the Eastern Tennessee Seismic Zone and there has been no recorded seiche of any significant magnitude reported as a result of earthquake events in the Tennessee Valley area.

Further examination of landslide activity as taken from the United States Geologic Survey (USGS) in the vicinity of the plant indicate that landslide incidence is low as shown in Figure 6-1. (Reference 23)

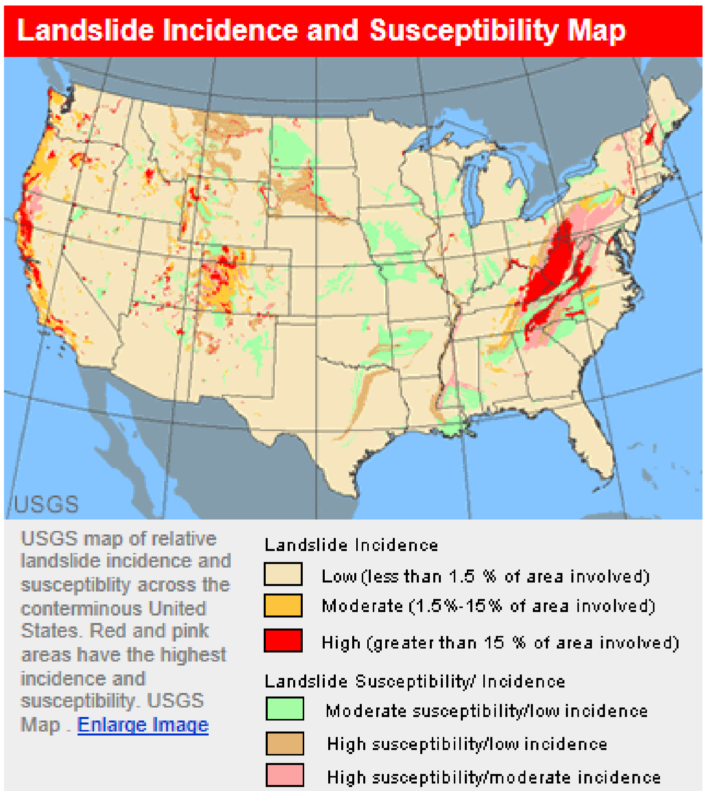


Figure 6-1 Landslide Incidence Map of United States

Examination of the slopes in the vicinity of the plant does not indicate instabilities or the potential for a landslide. There also have been no recorded incidences where landslides have generated a seiche in the TVA reservoir system.

Because the site is not located on an open or large body of water, surge or seiche flooding will not produce the maximum water levels at the site with nine feet of margin between normal non-flood conditions and plant grade.

6.5 Flooding from Tsunami

Flooding from tsunami was not previously evaluated for BFN and is not a credible flood causing mechanism at this site. The BFN site is located in Limestone County, Alabama about 30 miles west of the city of Huntsville, Alabama. At this location BFN is approximately 1,300 river miles inland (Tennessee River Miles 294, Ohio River Miles 48, and Mississippi River Miles 953) from the Gulf of Mexico. The Gulf of Mexico is the nearest body of open water directly downstream from Wheeler Lake that is subject to seismically generated tsunamis. Further, the site is more than

400 miles inland from the Atlantic coast and more than 470 miles inland from the Great Lakes. The BFN site with an elevation of 565.0 ft. will not be subject to the effects of tsunami flooding because the site is not adjacent to a coastal area.

The potential for a seismically induced hill-slope failure which could produce a tsunami-like wave in the vicinity of the plant were also examined. The slopes near the BFN site have been stable for many years and no landslides into the reservoir have been documented for Limestone County. (Reference 23)

6.6 Flooding from Ice-Induced Events

Flooding from ice-induced events was not previously evaluated for BFN and is not a credible flood causing mechanism at this site. The BFN plant is located in a temperate climate where significant amounts of ice do not form on lakes and rivers in the vicinity of the plant and ice jams are not a source of major flooding. On several occasions, ice has formed near the shore and across protected inlets but has not constituted a problem on the main river reservoirs. There has been no recorded incidence of ice near the plant site or of ice-induced flooding (Reference 24). The potential for significant surface ice formation is further reduced by the daily water level fluctuation resulting from power operations at Wheeler Dam located 19 river miles downstream of the plant and Guntersville Dam located 55 river miles above the plant.

There are no safety-related facilities at the BFN site which could be affected by an ice jam flood, wind driven ice ridges, or ice-produced forces. There are no valley restrictions in the 19 mile reach above Wheeler Dam to initiate a jam and an ice dam would need to reach more than 15 feet above normal winter levels to reach site elevation 565.0 ft. Thus, it is judged that an ice jam sufficient to cause plant flooding is not credible. Additionally, the flow of river water into the plant used for cooling water of safety related and non-safety related is well below the surface of the river pool and would not be affected by surface ice.

6.7 Channel Migration and Diversion

Channel migration and diversion was not previously evaluated for BFN and is not a credible flood causing mechanism at this site. The reservoir in the vicinity of BFN and above has been stable for many years with no indication of the potential for migration or diversion. Historic floods have not produced any major changes in the reservoir configuration. The reservoir width in the vicinity of the plant ranges from a low of around 7,000 feet to over 10,000 feet with stable slopes. It is judged that the potential for a channel diversion is not credible.

6.8 Flooding from Combined Effects

The following combinations will be considered in the reevaluation for BFN:

6.8.1 Floods Caused by Precipitation Events

Floods caused by precipitation events were previously evaluated and will be included in the reevaluation for BFN. This scenario evaluates the effects of wind-wave activity during floods at a site along the shore of an enclosed body of water. This combination is described in NUREG/CR-7046 (Reference 16) and includes the antecedent, PMP event, and waves induced by 2-year wind speed applied along the critical direction.

6.8.2 Floods Caused by Seismic Dam Failures

The load combinations identified for the reevaluation include present day methodology for probabilistic seismic hazard analysis and are as follows:

a. 10,000-year ground motion coincident with a 25-year flood and 2-year wind.

b. ½ of the 10,000-year ground motion combined with the lesser of ½ PMF or 500-year flood and 2-year wind.

7 DESCRIPTION OF MODELS USED FOR REEVALUATION

7.1 HEC-RAS

HEC-RAS is an integrated system of hydraulic analysis programs, designed for interactive use in a multi-tasking environment. The HEC-RAS modeling system was developed by the Hydrologic Engineering Center (HEC) which is a division of the Institute for Water Resources, U.S. Army Corps of Engineers (Reference 25). It was designed to simulate one-dimensional steady and unsteady flow in subcritical, supercritical or mixed flow regime in open channels.

TVA is using the HEC-RAS model for flood analyses performed in response to the post-Fukushima request for information letters under 10CFR50.54(f) because the HEC-RAS model is a well-documented and supported industry standard program; it allows the entire watershed to be modeled in a single continuous simulation. Additionally, use of internal and lateral structure rules allows the computation of any correction for submergence that may influence dam discharge directly at each time step during model runs.

HEC-RAS can be used to perform the following functions:

a. Steady-flow backwater profiles
b. Unsteady flow for subcritical flow regime
c. Unsteady flow for mixed flow
d. Dam breach modeling
e. Hydraulic design computations
f. Sediment transport computations
g. Water quality analysis
h. RAS mapper graphical inundation mapping

In steady flow mode, HEC-RAS explicitly solves the energy equation with an iterative procedure called the standard step method. In unsteady flow, HEC-RAS implicitly solves for flow and stage at every cross section using a finite difference approximation of the Saint Venant equations called the box scheme. Outputs of the HEC-RAS model include flood stages and flows.

7.1.1 Description of HEC-RAS Model Verification

The HEC-RAS model is dedicated in accordance with QA procedures. A Software Dedication Report was prepared that presented a test plan to identify the critical characteristics and limitations of the software. As a result 22 test problems were developed and verified using either hand calculations or other means.

7.1.2 Description of HEC-RAS Model Extents

The TVA total watershed HEC-RAS model is documented in the September 30, 2014 WBN License Amendment Request (LAR) supplement (Reference 26) and the January 28, 2015 WBN SER (Reference 27) and is used in this hazard reevaluation. Figure 7-1 and Figure 7-2 show the extent of the model, as well as the location of dams. The model is described in more detail in Section 8.

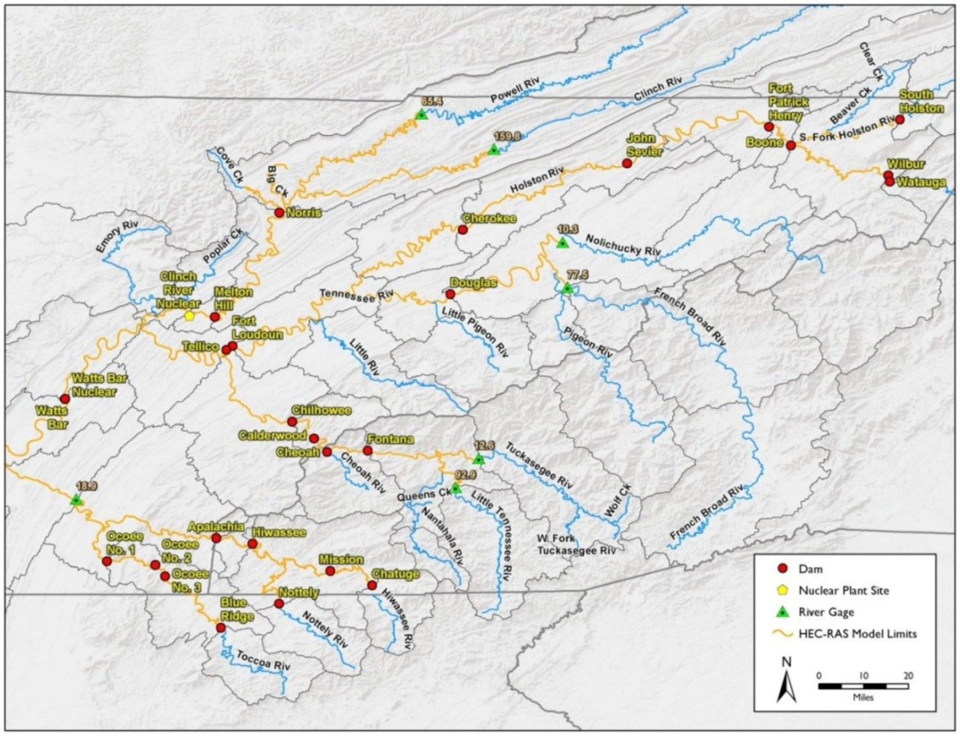


Figure 7-1 Upper HEC-RAS Model Extents

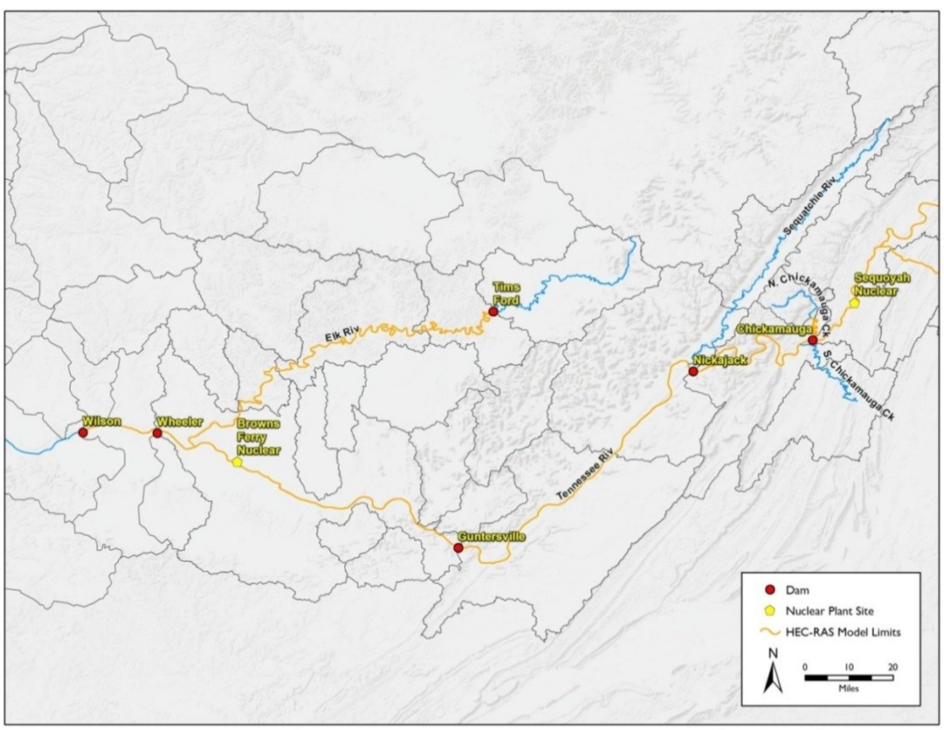


Figure 7-2 Lower HEC-RAS Model Extents

HEC-RAS channel modeling was employed to model the BFN East and BFN West areas. The West Channel utilized a steady-state HEC-RAS model to calculate water surface elevations. The contributing subareas' discharges that were computed in HEC-HMS models are inputs for HEC-RAS models. Only the highest peak HEC-HMS discharges, were applied as inflows to the HEC-RAS models. HEC-RAS channel modeling was employed to model the BFN East and BFN West areas. Summaries of these models follow.

The LIP analysis for the East Channel utilized an unsteady-state HEC-RAS to determine water surface elevations at critical areas. Storage areas in the HEC-RAS model were interconnected via weirs of variable profile. Some storage areas were connected directly to the channel simulations as upstream stage boundaries, and others were connected to the channel simulation at specific cross-sections via lateral structures, also modeled as weirs. Volume in the upstream storage area was permitted to overflow the site road towards the cooling tower hot water channel. This discharge hydrograph was used as input to the cooling tower HEC-HMS model.

7.2 HEC-HMS

The software Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) is a public source numerical modeling tool developed by the US Army Corps of Engineers (USACE) to model the hydrologic cycle and understand the behaviors and implications of watershed, channel, and water-control structure by simulating watershed precipitation and evaporation, runoff volume, direct run-off (overland and interflow), base flow and channel flow. The results of this software are used as an aid in decision making for: (a) planning and designing new flood-damage reduction facilities, (b) operating and/or evaluating existing hydraulic conveyance and water-control facilities, (c) preparing for and responding to floods, (d) regulating floodplain activities and (e) restoring or enhancing the environment.

7.2.1 Description of HEC-HMS Model Verification

The software is dedicated in accordance with QA procedures. Dedication is based on completion of the procedures in the HEC-HMS Validation Guide and parallel testing through alternative software. This Validation Guide is provided by the US Army Corps of Engineers Hydrologic Engineering Center and outlines a protocol for validating the HEC-HMS software on designated hardware. The protocol includes a Test Suite of thirty-four projects to test the ability to properly perform the HEC-HMS simulation commands. In addition, a sample of the projects included in the Validation Guide was completed using alternative software and compared to the HEC-HMS results.

7.2.2 Description of HEC-HMS Model Extents

HEC-HMS models are used to calculate runoff hydrographs from each subarea and maximum water surface levels of storage areas in the LIP analysis. The BFN LIP HEC-HMS model comprises twelve sub-basins totaling 890 acres. Reservoir modeling is used where no clear drainage channel exists and as an alternate method to determine water surface elevations. Figure 7-3 through Figure 7-5 present the model extents of the HEC-HMS models used in the LIP analysis.

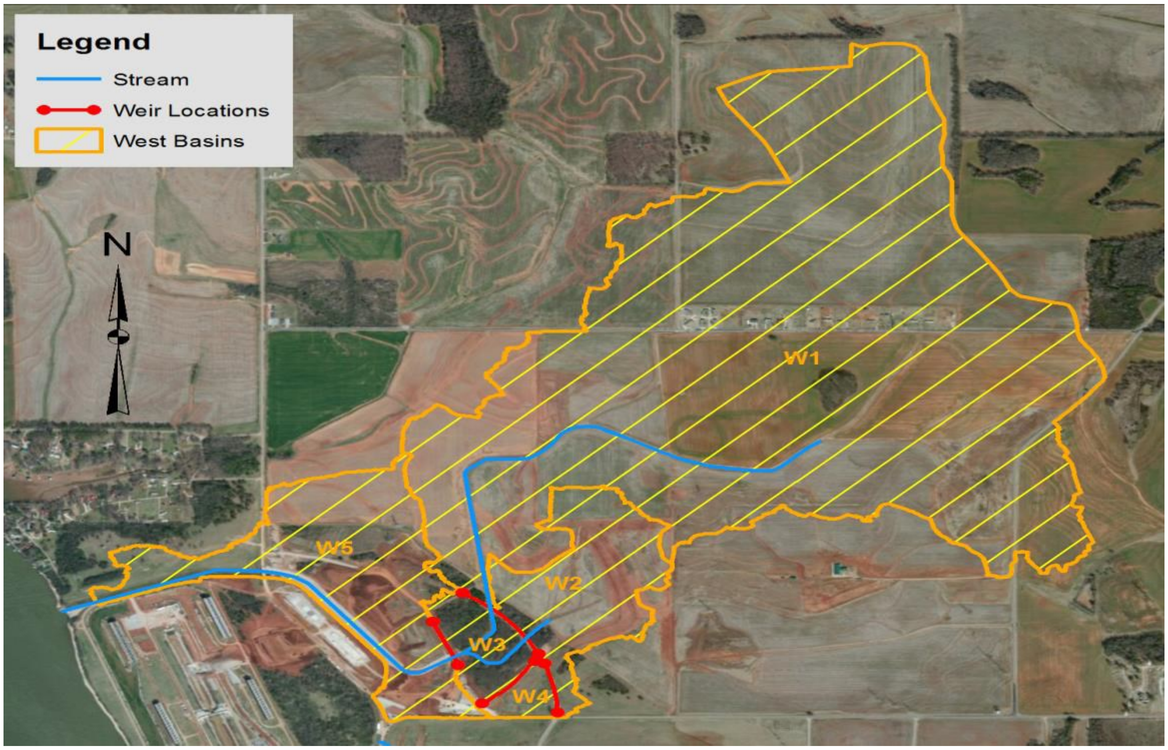


Figure 7-3 HEC-HMS Model Extents – West Channel Drainage Areas

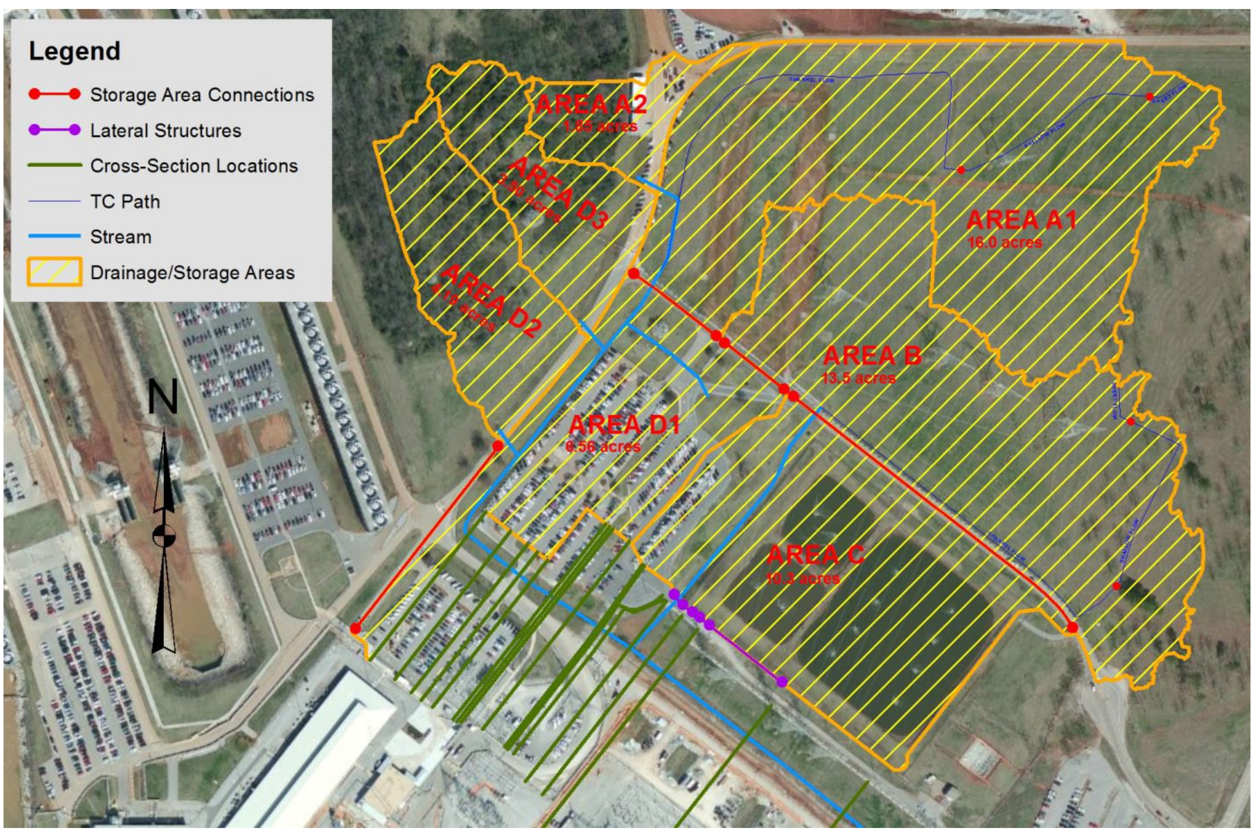


Figure 7-4 HEC-HMS Model Extents – East Switchyard Drainage Areas

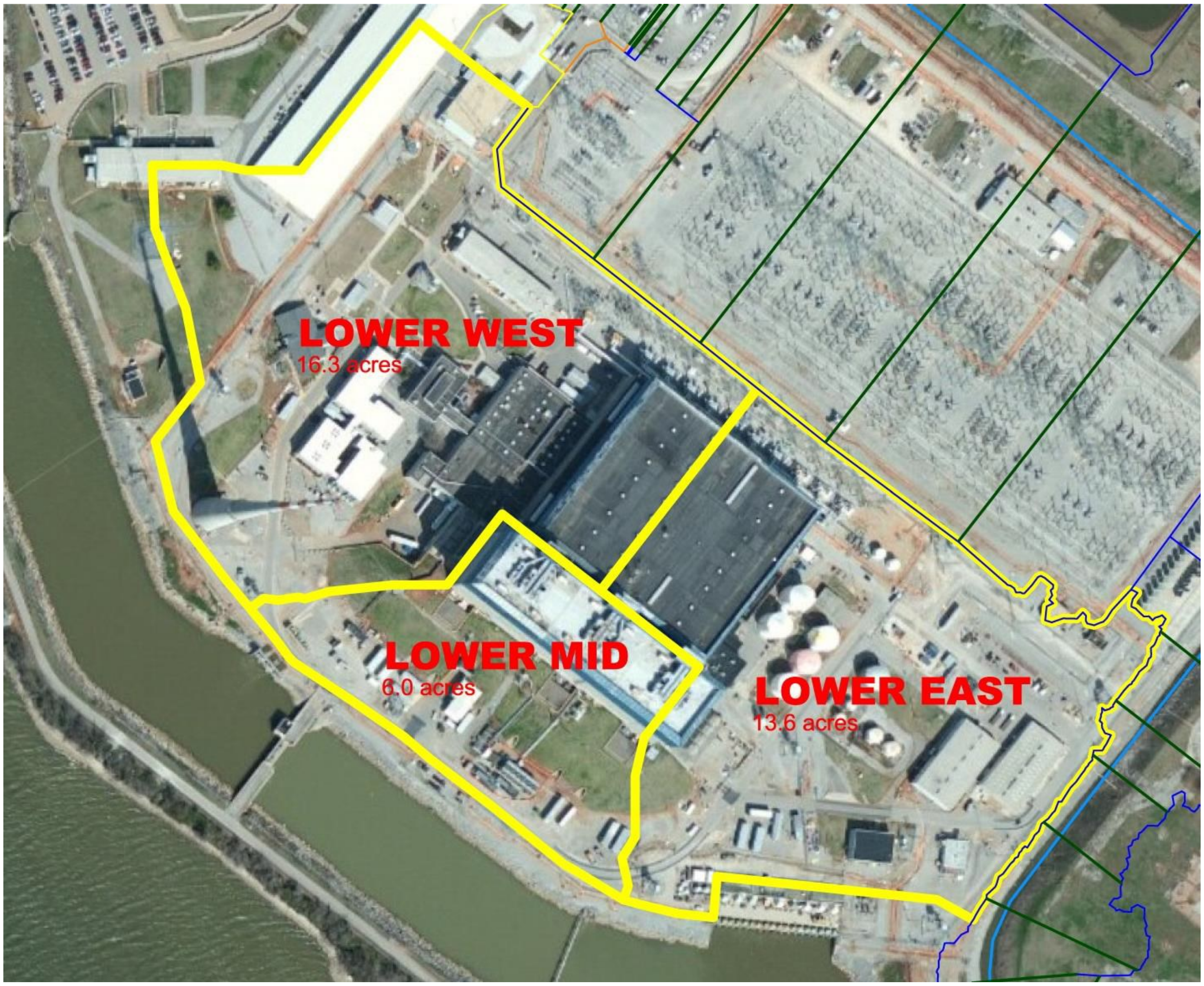


Figure 7-5 HEC-HMS Model Extents – Lower Plant Subareas

8 JUSTIFICATION OF INPUTS

The verified HEC-RAS unsteady flow model of the Tennessee River System documented in the September 30, 2014 WBN LAR supplement (Reference 26) and the January 28, 2015 WBN SER (Reference 27) is used in the BFN hazard reevaluation. It is utilized to predict flood elevations and discharges for floods of varying magnitudes including the PMF and flooding from dam failures. Inputs to the HEC-RAS model are described in the following subsections.

8.1 HEC-RAS Model Geometry Development and Calibration

Tennessee River geometry information was developed for the HEC-RAS model with the primary objectives of generating and/or verifying cross-sectional data and augmenting cross-sections to account for reach storage. USACE bathymetric surveys, DTM/DEM data from state and local databases, and USGS topographical maps were used in developing and verifying cross-sections. The development of the HEC-RAS geometry is detailed in References 28 and 29.

Once the HEC-RAS geometry was developed, the HEC-RAS model for each reach was calibrated to historic events. The majority of the main stem reservoirs are calibrated to the March 1973 and May 2003 flood events, Wheeler Reservoir is calibrated to the March 1973 and December 2004 flood events, Wilson Reservoir is calibrated to the May 2003 flood event, and the tributaries are calibrated to two large flood events of record for the respective reaches. This enabled reliable prediction of flood elevations and discharges at downstream locations in the Tennessee River system. The flow, elevation, and date information for the historic events used in calibration is from TVA, National Weather Service, and United States Geological Survey data as well as FEMA flood profiles. The calibrated reaches were linked together to form a continuous model for flood simulation. The HEC-RAS model calibration produced final geometry files to be used in simulations of floods at TVA nuclear plant sites. The calibration process is further described in References 30 and 31.

8.2 Dam Rating Curves

Initial dam rating (headwater rating) curves are required as inputs to TVA's HEC-RAS model used in performing flood-routing calculations for the Tennessee River System. The initial dam rating curves provide total dam discharge as a function of headwater elevation. Final dam rating curves are simulation specific and determined in the HEC-RAS model which incorporates tailwater effects.

The Dam Safety group of TVA's River Operations (RO) division evaluated the stability of TVA's dams under various load conditions in accordance with TVA Dam Safety acceptance criteria; they also considered load conditions that comply with nuclear guidelines. The load conditions studied included PMF level headwater elevations with varying tailwater elevations and multiple seismic load conditions. Each stability analysis report was examined and the results provided the basis for the dam failure cases presented in these dam rating curves. Earthen embankment breaches were determined from empirically based methods as recommended in Reference 19. The dam rating curves are documented in References 32 and 33.

8.3 Unsteady Flow Rules

HEC-RAS uses unsteady flow rules to control complex releases from hydraulic structures. In the reevaluation, unsteady flow rules were developed in HEC-RAS to represent the operations of the reservoirs in the Tennessee River system upstream of Wilson Dam. The rules blend the flood operational guides (Reference 34) and dam rating curves (References 32 and 33). The unsteady flow rules incorporate the flood operational guides, as they provide prescribed operating ranges of reservoir levels for the reservoirs in the TVA system. The rules reflect the flexibility provided in the guides to respond to unusual or extreme circumstances through the use of elbow recovery curves; seasonal variability in the operational guides is also included in the unsteady flow rules. Elbow recovery curves are used during the antecedent storm to expedite the recovery of the reservoir to a more normal state. The use of elbow curves is explained in detail in the Flood Operational Guides (Reference 34). The antecedent storm, in which the elbow recovery curves are implemented, is a three-day storm occurring prior to a three-day dry period and the three-day main storm. Once the antecedent storm is complete and the surcharge elevation is exceeded the discharges will be calculated using the dam rating curves for the applicable cases. The surcharge elevation is the elevation at which gates are fully open and discharge through the dam is computed by the dam rating curve.

The dam rating curves are used in concert with the flood operational guides in the unsteady flow rules to define total dam discharge as a function of headwater elevation, tailwater elevation, and outlet configuration. 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, the tops of open spillway gates, tops of spillway piers, saddle dams, and rim leaks. If the operating deck is not exceeded, operations return to the Flood Operation Guides (Reference 34). The unsteady flow rules are documented in References 35 and 36.

8.4 Probable Maximum Flood Inflows

To determine the inflows for the PMF event, rainfall depths for the 21,400 square-mile downstream centered March and the 7,980 square-mile Bulls Gap centered March PMP events, as described in HMR 41 (Reference 7), were determined; hydrographs were developed using validated unit hydrographs (UHs) as well as UHs that were adjusted for non-linear basin response (Reference 37); and storage from potentially critical projects outside the model limits are identified and included.

The application of the following approach was adopted for inflow development (Reference 38) in sub-basins above Wheeler Dam for use in the subsequent Tennessee River routing model. Inflows to the sub-basins between Wheeler and Wilson Dams are conservatively based on a constant, peaks flow with no losses.

1. transform rainfall to runoff using available UHs and using UHs adjusted for non-linear basin response;

2. develop sub-basin surface runoff hydrographs both with no losses and using applicable loss rates;

3. include sub-basin monthly average constant baseflow (Reference 39);

4. develop total event inflow hydrographs for the sub-basins in the Tennessee Valley watershed above Wheeler Dam; and

5. as necessary, translate developed tributary sub-basin surface runoff hydrographs to model input points.

8.4.1 Hydrological Meterological Report

The applicability of the National Weather Service (NWS) HMR to the development of the PMF at TVA nuclear projects was reviewed and documented in Reference 40. This was done in accordance with the requested actions in the 50.54(f) letter requiring a reevaluation of flood causing mechanisms using present-day methodologies.

The NWS is no longer funded for PMP research and has not updated the HMRs since their publication. While the Bureau of Reclamation references indicate that updated PMP estimates are needed, no evidence was found of any published revision in the PMP estimates applicable to the TVA projects. In review, only one errata was noted in HMR-41 Table 7-2 where two values were assumed swapped and were noted in the calculation where used (Reference 7). The HMRs had a sound methodology and data basis at the time of the analysis.

TVA is currently reevaluating the PMP and developing a replacement for the HMRs. The process involves an expert panel review of the product in its entirety, with specific attention to storm selection and storm transposition. The replacement for HMRs was not completed at the time of this reevaluation but preliminary data supports the conservatism of the current HMRs. Therefore, the current HMRs meet the requirements of the 10 CFR Section 50.54(f), and are appropriate for use in the re-analysis.

8.4.2 Critical Storm Selection

The critical storm selection for the PMF event on the Tennessee River for BFN is reviewed and documented in Reference 41. As defined in Section 1.1 of the NUREG/CR-7046 (Reference 16), the probable maximum event is "…the event that is considered to be the most severe reasonably possible at the location of interest and is thought to exceed the severity of all historically observed events. For example, a PMF is the hypothetical flood generated in the drainage area by a PMP event." The PMP for the Tennessee Valley at the aforementioned nuclear projects is currently defined by the NWS HMR-41 (Reference 7).

The HMR-41 (Reference 7) guidance defines two general PMP configurations. The first configuration is a 21,400-square-mile PMP event with either an upstream or a downstream centering, which has a fixed location over the Tennessee Valley watershed. The second pattern type is a moveable 7,980-square-mile PMP event that may be slid roughly from southwest to northeast along the long axis of the published pattern.

As stated in Reference 42, currently analyzed storms are the 21,400-square-mile downstream centered event, the 7,980-square-mile Bulls Gap centered event and the 7,980-square-mile Sweetwater centered event. Selection of these events was based on the original TVA plant licensing analysis. The original analysis was based on the ANSI N170-1976/ANS-2.8, Section 5.2.6 guidance recommending downstream placement and use of NWS proposed methods. As expected, the previous and current TVA modeling efforts show that the critical PMP storm event producing the PMF when routed maximizes the rainfall volume over the total watershed. A re-analysis was performed using GIS software to allow direct comparison of the reviewed events.

The GIS analysis of the 21,400 square-mile PMP event showed that the downstream centered event produced higher rainfall depths at the locations reviewed. An analysis of the 7,980-square-mile PMP event was also performed. Additionally, two storms have previously been evaluated for BFN, the March 16,170 square-mile and March 12,030 square-mile events. The March 16,170 square-mile PMP is centered around Nickajack Reservoir. As specified in HMR 41 (Reference 7), the 21,400 square-mile PMP at BFN has a subsequent storm. Unlike the 21,400 square-mile PMP, the 16,170 square-mile PMP does not have a subsequent storm (Reference 8) and would cause fewer upstream tributary dam failures. The 12,030 square-mile storm is not documented in an HMR and was not considered for BFN.

It was determined that PMP depth is maximized at BFN by the 21,400-square-mile, downstream centered event. An independent analysis performed by Pacific Northwest National Laboratory confirmed that the critical storm centerings are identified for use in computing PMP rainfall (Reference 43).

8.4.3 National Inventory of Dams (NID) Inflows

The USACE maintains the National Inventory of Dams (NID), which provides characteristics for each dam (location, height, and volume). The guidance for assessment of flooding hazards due to dam failure (Reference 19) requires a screening process to identify all dams that are inconsequential. In order to identify the number of structures upstream of BFN the NID was queried for the Tennessee Valley watershed and approximately 1,100 dams were included in the analysis. As documented in Reference 44, rectangular-shaped hydrographs are used at existing inflow locations to account for the volume of these dams. These hydrographs are distributed across 6 days, from one day after the peak antecedent precipitation to one day after the peak main storm precipitation.

8.5 Seismic Inflows

Staff positions listed in Section 5.6 of Reference 19 specify that the coincident inflow from the 25-year flood be applied during the 10^{-4} annual exceedance probability seismic hazard and either the 500-year flood or the half PMF be applied as the coincident inflow during half the 10^{-4} ground motion. To develop these inflows, a methodology for production of scaled hydrographs was developed in Reference 45. The scaled hydrograph methodology used starts with the selection of streamflow event durations sufficient to allow maximization of the headwater elevation at the hypothetical failure location. These durations are then used in probabilistic analyses to develop the required return period volumes from historical streamflow data. A candidate historical or synthetic rainfall event sufficiently large to reflect the watershed translation of rainfall to runoff is then selected. This rainfall is distributed across the watershed, losses are applied and the surface runoff is generated based on available unit hydrograph data. The candidate surface runoff ordinates are then scaled to produce the calculated probabilistic volumes at the selected durations.

8.5.1 National Inventory of Dams (NID) Seismic Inflows

During postulated single and multiple project failure events, the concurrent failure of NID identified projects outside the model is considered possible. The NID volumes are located across the sub-basins with conveyances having differing sinuosity, length, slope, cross sectional and roughness characteristics. As a result, the postulated failure waves are expected to pass through a variety of supercritical, critical and subcritical flow regimes as they traverse the respective reaches starting at the failure location and ending at the respective model input points. The resulting translation reduces the peak flows and spreads the time base of the volume input. A simplified calculation approach, as described in Reference 46, is used to account for the NID volumes under these failure conditions.

8.6 Sunny Day and Watauga Project Specific PMF Inflows

Sunny day project failures are postulated to occur due to non-hydrologic and non-seismic causes as required by Reference 19. Simplified volume analyses are used and TVA projects having the potential to cause flooding at the plant sites are identified in Reference 46. The Watauga Project Specific PMF analysis is included in the sunny day failure analysis to provide a bounding scenario for the sunny day failures on the Holston tributary. Inflows for both the Sunny day failures and the Watauga Project Specific PMF are documented in Reference 45.

The development of two inflow scenarios for use in these model calculations is necessary. Inflows for use concurrent with sunny day failures identified in Reference 46 are included in Reference 45. Constant June baseflows from Reference 39 are applied for both. Watauga project PMP rainfall was taken from Reference 46 as recommended by Reference 19 and is convoluted in a spreadsheet using Soils Conservation Service (SCS) methodology. June curve numbers were taken from Reference 11 and unit hydrograph data were taken from Reference 38. NID inflows are not included in sunny day simulations.

9 APPLICABLE FLOOD CAUSING MECHANISMS

9.1 Local Intense Precipitation

9.1.1 Previous Analysis

Previous evaluation of the effect of LIP on water surface elevations in the BFN plant site area is described in Section 3.4.1.

PMP for the plant drainage systems is defined by HMR 56 (Reference 6). The underground drains are assumed clogged, and runoff is assumed to be equal to rainfall. Four flooding sources were identified; (1) the small unnamed stream northwest of the plant, (2) the area draining to the switchyard drainage channel, (3) the main plant area, and (4) the area draining to the cooling tower system of channels. The site was divided into four respective drainage areas.

9.1.2 Technical Approach

The reevaluation of the LIP on water surface elevations in the BFN plant site utilized HEC-HMS and HEC-RAS simulations and is documented in Reference 47. The LIP analysis is a measure of the extreme precipitation (high intensity and short duration) at a specific location, in this case BFN. According to Reference 16, LIP should be equivalent to the 1-hr, 1-mi^2, PMP at the location of the site. The analysis assumes fully functional site grading and partially blocked drainage channels. Inputs to the model are described in Section 8.

This analysis conservatively assumes that drainage features carrying offsite drainage toward BFN are fully functional and drainage features carrying on-site drainage away from BFN are not functional. Additionally, this analysis assumes partially, but significantly, blocked drainage channels. For the West diversion channel the old bridge on the abandoned road was assumed to fail, providing a 7-foot high obstruction for the entire width of the channel at the cross-section. Simultaneously for the East Switchyard PMP channel a 40-foot long, 10-foot obstruction in the bottom of the channel, approximating a trailer building, was applied at one cross-section near the north east corner of the switchyard and a 20-foot long 6-foot high rectangular obstruction, similar in size to a car, was applied at the location where the vehicle barrier system crosses the channel. For the West channel, two methods were employed in evaluation, the first with the originally determined Manning's n values, the second with increased Manning's n values to consider the effects of vegetative growth in the channel. Below the plant the model weir length was conservatively reduced.

The maximum water surface elevation (WSE) results of this analysis are presented in Table 9-1.

Table 9-1 Results of BFN LIP Analysis

Area	Maximum WSE (ft.)	Critical Elevation
West Channel (Station 16)	590.4	592.0*
West Channel (Station 6)	576.7	578.0*
East Switchyard	578.2	578.0
BFN Plant Area (Lower Mid)	565.2	565.0
BFN Plant Area (Lower West)	566.2	565.0
BFN Plant Area (Lower East)	566.6	565.0

*Elevation is a threshold elevation, if exceeded further evaluation would be required.

9.2 Flooding from Rivers and Streams

9.2.1 Previous Analysis

The results of the previous PMF analysis in Section 3.4.2 are presented in the BFN FSAR (Reference 2). This analysis incorporates updated channel geometry, updated dam rating curves, operational allowances and additional model refinements such as the Fort Loudoun-Tellico canal complex and the Dallas Bay-Lick branch-North Chickamauga Creek complex. The inflows for this analysis are determined using the FLDHYDRO and the API runoff methodology. The SOCH suite of programs was used to determine the PMF elevation.

9.2.2 Technical Approach

The reevaluation of the PMF is in accordance with the guidance in Reference 16. The PMP is applied to the drainage basin of the rivers and streams adjoining BFN. Inflows to the model were generated using industry standard codes and the (Soils Conservation Service) SCS runoff methodology. The SCS runoff parameters are calibrated to the inflow model presented in the FSAR (Reference 2). Curve numbers (CNs) used were validated against the TVA API method results for the 21,400-sq.-mi. March event. When compared to National Resources Conservation Service soils and USGS Multi-Resolution Land Characteristics data for sub-basins above the Wheeler project, the area weighted, validated CNs for the antecedent event are approximately 15.7% higher and for the main storm event are 4.3% higher. Therefore these CNs are considered conservative. Inflow development, storm selection, as well as additional inputs and assumptions are described in Section 8.

The verified HEC-RAS unsteady flow model of the Tennessee River System developed for the WBN LAR and approved in the NRC January 28, 2015 WBN SER (Reference 27) is used in the hazard reevaluation. The summary of differences between the BFN FSAR PMF analysis and the hazard reevaluation include:

a. The stream course model used was changed from the SOCH suite of software, including TRBROUTE, CONVEY, WWIDTH, and SOCH, to the USACE HEC-RAS model.

b. Twenty two critical TVA dams were evaluated for stability, including:

1. Apalachia
2. Blue Ridge
3. Boone
4. Chatuge
5. Cherokee
6. Chickamauga
7. Douglas
8. Fontana
9. Fort Loudoun
10. Fort Patrick Henry
11. Guntersville
12. Hiwassee
13. Melton Hill
14. Nickajack
15. Norris
16. Nottely
17. South Holston
18. Tellico
19. Tims Ford
20. Watauga
21. Watts Bar
22. Wheeler

c. Nine dams in the tributary system, that were not evaluated and are postulated to fail, were included:

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

d. Six dams that were evaluated for stability, but were not credited due to low margin are included:

1. Apalachia (during the 21,400 sq. mi. storm)
2. Boone
3. Fort Patrick Henry
4. Melton Hill (During the 7,980 sq. mi. storm)
5. Nickajack
6. Guntersville

e. A license condition for WBN is included in the NRC WBN SER Reference 26 that states modifications to dams will be completed. These modifications are credited in this analysis and are as follows:

1. Cherokee - Post-tensioning non-overflow dam and raising embankment overtopping elevation (removing HESCO® barriers)
2. Douglas - Post-tensioning non-overflow dam and raising embankment saddle dam overtopping elevation; adding saddle dam toe berms
3. Fort Loudoun - Post-tensioning non-overflow dam and replacing HESCO® barriers with permanent modifications. (approximately 1900 ft. of the HESCO® barriers will be replaced by the end of 2017 following installation of new bridge)
4. Tellico - Reinforcing the non-overflow dam "neck" and raising the embankments overtopping elevation (removing HESCO® barriers)
5. Watts Bar - Reinforcing the portions of the non-overflow and lock "necks"; raising the overtopping elevation of embankments and flood walls (removing HESCO® barriers); lowering the west saddle dam elevation to 752.0 ft.

Two HEC-RAS simulations were performed:

a. 21,400 square-mile downstream-centered March storm

b. 7,980 square-mile Bulls Gap-centered March storm

The PMF in rivers and streams adjoining BFN is determined by applying the PMP to the drainage basin of these rivers and streams. Inflows to the model are generated using industry standard codes and the SCS runoff methodology.

Results of the PMF analysis for the two candidate storms, the 21,400 sq.-mi. storm and the 7,980 sq.-mi. Bulls Gap centered storm, are presented in Table 9-2 (Reference 48). This analysis assumes overtopping and failure of Douglas Saddle Dams 1 and 3.

Table 9-2 PMF Elevations at BFN (TRM 294) Resulting from Reevaluation

PMF Event	Elevation (ft)	Discharge (cfs)
21,400 Sq.-Mi. Event	571.7	1,190,000
7,980 Sq.-Mi. Bulls Gap Event	572.1	1,220,000

9.3 Flooding from Dam Breach or Failures

Upstream dam failures by seismic events, structural defects, as well as other failure modes have the potential to impact BFN.

A simplified volume analysis identifies individual dams that have the potential to cause flooding at BFN if failed during a sunny day (Reference 46). The analysis conservatively assumes that gates at downstream dams are closed and inoperable providing a bounding scenario for cascading dam failures as a result of a sunny day dam failure. The results of the analysis identified six dams whose failures could lead to flooding at the plant. These dams are as follows:

1. Cherokee
2. Guntersville
3. Fontana
4. Norris
5. Nottely
6. South Holston
7. Watauga

The six dams identified were evaluated in one of three ways: under a project specific PMF event, single seismic dam failure combined with a flood event, or failure during a sunny day. A Watauga project specific PMF simulation bounds a sunny day failure of the South Holston dam. Guntersville Dam has low margin for seismic stability and are analyzed as single seismic dam failure simulations paired with a flooding event. These seismic simulations bound sunny day dam failure simulations for these dams. The remaining dams, Cherokee, Fontana, and Norris Dams are analyzed for failure in sunny day simulations. The Fontana Dam sunny day failure simulation bounds the Fontana seismic failure simulation because the seismic failure results in only a minor loss of the Fontana Dam.

9.3.1 Project Specific PMF

A project specific PMF at Watauga Dam serves as the upper bound of any sunny day failure at South Holston or Watauga dams. The postulated failure of Watauga Dam results in the cascading failure of Wilbur, Boone, Fort Patrick Henry, and John Sevier Dams. The results are presented in Table 9-3 (Reference 49)

Table 9-3 Elevations and Discharge at BFN Resulting from Project Specific Dam Failures

Dam Failure	Elevation (ft.)	Discharge (cfs)
Watauga Dam Project Specific PMF	556.5	234,000

9.3.2 Sunny Day Dam Failure

9.3.2.1 Previous Analysis

No previous analyses were completed on the failure of a dam upstream of BFN on a sunny day.

9.3.2.2 Technical Approach

Potential failure modes were evaluated for each dam to determine the sunny day simulations breach parameters.

a. Cherokee-The South Embankment is the longest and tallest embankment at Cherokee and is assumed to fail for the sunny day simulation.

b. Fontana–A total failure of Fontana Dam is assumed in the sunny day simulation. This failure will present the greatest potential for flooding downstream.

c. Norris–The Right Abutment is assumed to collapse and fail in the sunny day simulation.

The single seismic failure of Guntersville Dam bounds a sunny day failure. The single seismic failure results at Guntersville Dam are presented in Table 9-5. Results of the sunny day dam failure simulations (References 49 and 50) are presented in Table 9-4.

Table 9-4 Elevations and Discharge at BFN Resulting from Sunny Day Dam Failures

Dam Failure	Elevation (ft)	Discharge (cfs)
Cherokee Dam Sunny Day Failure	556.7	257,000
Fontana Dam Sunny Day Failure	557.5	354,000
Norris Dam Sunny Day Failure	556.8	280,000

9.3.3 Single Seismic Dam Failure

The results of the single seismic failures combined with a 500-year flood event are presented in Table 9-5. The failure of Chickamauga Dam is evaluated in the multiple seismic dam failure simulations in Section 9.4.2. The multiple seismic dam failures would bound a single seismic failure of Chickamauga Dam. The results of these simulations are shown in Table 9-8.

Table 9-5 Elevations and Discharge at BFN Resulting from Single Seismic Failure of Upstream Dams

Seismic Dam Failure Combination	Elevation (ft.)	Discharge (cfs)
Guntersville Dam Single Seismic Failure During a 500-Year June Flood Event	558.6	489,000

9.4 Flooding from Combined Effects

9.4.1 Floods Caused by Precipitation Events

9.4.1.1 Previous Analysis

In the previous flood evaluation, described in Section 3.4.8.1, coincident wind wave run-up was computed for and applied to the controlling PMF event. Coincident wind wave activity was determined based on a "reasonably severe windstorm producing 45 mile per hour sustained wind speeds" (Reference 2) and using the 1% wave of which 5 per hour will occur.

9.4.1.2 Technical Approach

In this reevaluation, this wind wave combination scenario evaluates the effects of wind-wave activity during floods at a site along the shore of an enclosed body of water. This combination is described in NUREG/CR-7046 (Reference 16) and includes the lesser of ½ PMF or 500-year flood, the worst windstorm with wind-wave activity, and the maximum controlled water level. The results of the analysis are the wind wave heights at the five critical structures to be added to the PMF elevation. Conservatively, the wind wave for the Reactor Building, Diesel Generator 1 & 2 Building, Diesel Generator 3 Building, and Radwaste Building are determined as a composite structure as the critical fetch for the composite structure bounds that of the individual structures.

The U.S. Nuclear Regulatory Commission's Guidance for Assessment of Flooding Hazards Due to Dam Failure (Reference 19) requires that wind wave activity be accounted for at dams. Wind Wave activity was calculated at Cherokee, Douglas, Fort Loudoun, Hiwassee, Melton Hill, Norris, Nottely, South Holston, Tellico, Tim's Ford, Watauga, and Watts Bar Dams. Wind wave activity was not calculated at Chickamauga, Nickajack, and Guntersville Dam because these dams were overtopped during the PMF. Wind wave activity was calculated at the previously mentioned dams because the top of dam elevations were such that they would not be overtopped by the stillwater flooding elevation, but could conceivably still be susceptible to wind wave damage. The wind wave at dams is presented in Table 9-6.

NUREG/CR-7046 (Reference 16) requires determination for combined-effects flooding to include wind wave height for the 2-year wind added to the PMF elevation. The wind wave heights to be added to the PMF elevation at critical structures (Reference 51) are displayed in Table 9-7 and in Figure 9-1.

Table 9-6 Wind Wave Results for Dams

Dam	Maximum Stillwater Elevation (ft.)	Maximum Wave Height, Hmax (ft.)	Dam Crest Elevation (ft.)
Cherokee[a]	1095.2	1.8	1095.8
Douglas Saddle Dam 1[b]	N/A	N/A	N/A
Fort Loudoun	834.6	1.1	837.0
Hiwassee	1536.7	0.8	1537.5
Melton Hill	812.2	0.6	805.0
Norris (Main)	1056.4	0.8	1061.0
Nottely	1782.9	1.3	1807.5
South Holston	1756.3	1.1	1765.0
Tellico	831.7	2.2	834.9
Watauga	1990.9	0.7	2012.0
Watts Bar	768.8	1.3	772.0

[a] The volume of water overtopping is evaluated in Reference 51 and does not result in damage to Cherokee embankments.

[b] Douglas Dam wind wave values are not applicable because Douglas Saddle Dams 1 and 3 is assumed overtopped an failed.

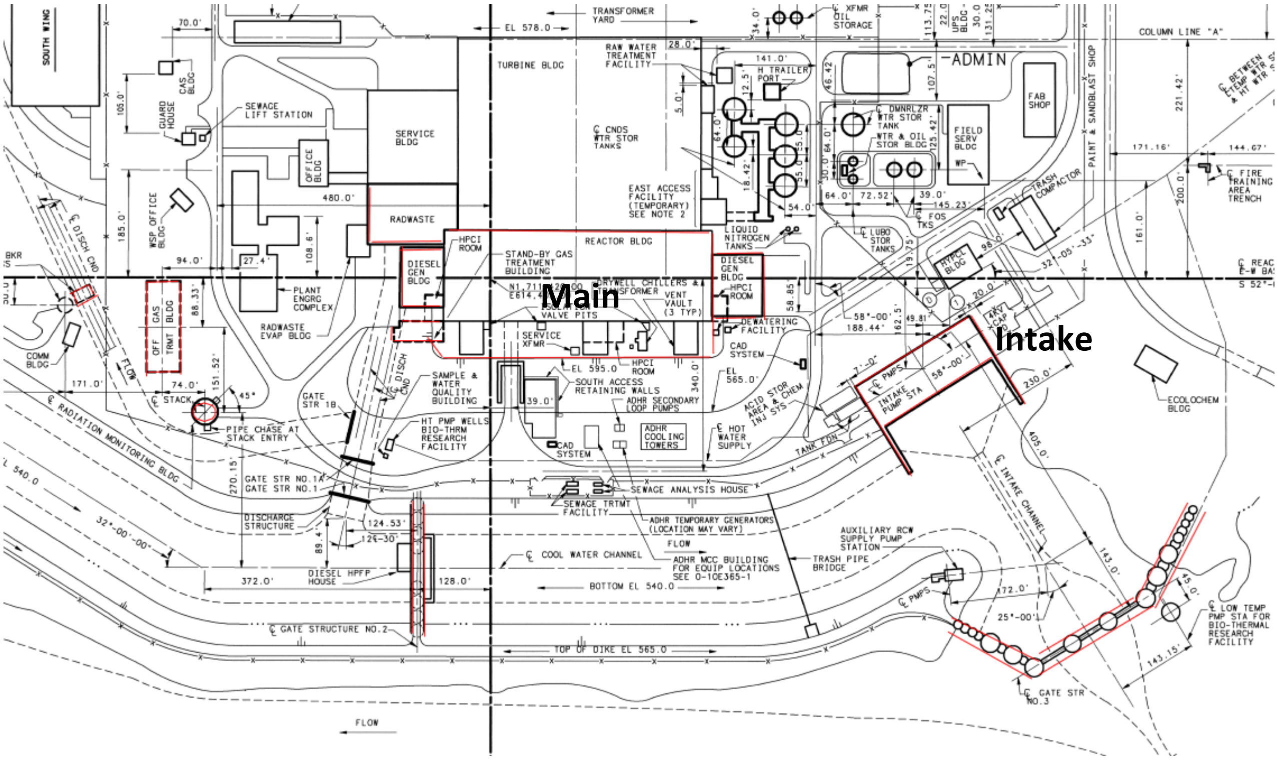


Figure 9-1 BFN General Site Plan - Structures

Table 9-7 Wind Wave Elevation Results at BFN Site (Reference 51)

Location	Stillwater PMF Elevation (ft.)	Total Wind Wave Height added (Wave run-up + wind setup) (feet)	Final PMF Elevation (ft.)
Intake	572.1	4.6	576.3
Diesel Generator 1 & 2 Building[a]	572.1	5.1	577.2
Diesel Generator 3 Building[a]	572.1	5.1	577.2
Radwaste Building[a]	572.1	5.1	577.2
Reactor Building[a]	572.1	5.1	577.2

[a] Included in the area marked "Main" in Figure 9-1

9.4.2 Multiple Seismic Dam Failures with Combined Flood Event

9.4.2.1 Previous Analysis

No previous analyses were completed to evaluate flooding due to seismically induced dam failure with combined flood event.

9.4.2.2 Technical Approach

The seismic dam failure reevaluation is in accordance with the guidance in References 16 and 19. Staff positions listed in Section 5.6 of Reference 19 specify that the coincident inflow from the 25-year flood be applied during the 10^{-4} exceedance probability seismic hazard and either the 500-year flood or the half PMF be applied as the coincident inflow during half the 10^{-4} exceedance probability seismic hazard ground motion. This applies to both single and multiple seismic dam failures. An analysis to support the screening of multiple dam failures due to a single seismic event is detailed in Reference 52. The analysis presents deaggregation results for the 10,000 year and ½ 10,000 year ground motion for both concrete and earthen embankments.

Using seismic stability results, a volume analysis was performed to determine which seismic centering would result in the largest volume of water that could be released. The results indicate that a Fort Loudoun centered seismic event results in multiple dam failures with the largest volume of water that could be released for a 10,000 year ground motion. A Douglas centered seismic event results in multiple dam failures with the largest volume of water that could be released for a ½ 10,000 year ground motion. In both centerings, Nickajack Dam fails due to overtopping.

The reevaluation considered the following scenarios:

a. 10^{-4} Fort Loudoun centered seismic event resulting in failures of Apalachia, Blue Ridge, Chatuge, Chickamauga, Fort Loudoun, Fontana, Melton Hill, Tellico and Watts Bar Dams and the Watts Bar West Saddle Dam coincident with a 25-year June flood event

b. ½ 10^{-4} Douglas centered seismic event resulting in failures of Apalachia, Blue Ridge, Chatuge, Chickamauga, Fort Loudoun, Fort Patrick Henry, Melton Hill, Tellico and Watts Bar Dams and the Watts Bar West Saddle Dam coincident with a 500-year June

Results of the seismic failure of upstream dam simulations (Reference 53) are presented in Table 9-8.

Table 9-8 Elevations and Discharge at BFN Resulting from Seismic Failure of Upstream Dams

Seismic Dam Failure Combination	Elevation (ft.)	Discharge (cfs)
10,000-Year Fort Loudoun Centered Seismic Failure During a 25-Year June Flood Event	559.4	626,697
Half-10,000 Year Douglas Centered Seismic Event During a 500-Year June Flood Event	560.9	678,817

10 EVALUATION OF UNCERTAINTIES

Inherent uncertainties exist in the analysis of the PMF in rivers and streams. Flooding simulations require many assumptions while determining input parameters for the analysis as well as during simulation. These assumptions are based on available data and industry accepted practice. NRC Interim Staff Guidance recommends several sensitivity analyses be performed to understand and account for this inherent uncertainty in key parameters of the flood hazard reevaluation (Reference 19). Recommended sensitives included evaluation of dam breach configuration, debris accumulation, gate failures, initial reservoir levels, reservoir inflow, and tailwater conditions. Sections 10.1 – 10.5 discuss the sensitivity analyses performed to support the flood hazard reevaluation for BFN.

10.1 100% Runoff

Simulations assuming no losses (100% runoff) were performed for the two PMF storm events (Reference 54), the 21,400 square-mile downstream centered March event and the 7,980 square-mile Bulls Gap centered March event. The 100% runoff simulations assumed no precipitation losses and quantified the effects on flood elevation and discharge at BFN due to the increased runoff volume. The purpose of the 100% runoff simulations was to present the upper bound for runoff volume for the two PMF storm events.

The results of the 100% Runoff sensitivity simulations produced an increase in elevation at BFN of 4.2 feet for the 21,400 square-mile storm event, and an increase of 5.7 feet for the 7,980 square mile Bulls Gap Storm event because of additional cascading dam failures.

These increases are not realistic and need not be considered in the re-evaluation. This conclusion is based on:

- Section 4.1 indicates the impervious area of the watershed above Guntersville Dam is extremely small (1.96%) and it is unrealistic to assume 100% runoff for the entire watershed during a nine day event based on land use.
- Rainfall to runoff transformation rates calculated are approximately 88%.
- Unit hydrographs used in determining inflow to streams have been calibrated to actual storm run-off levels for larger historical events.
- Reservoir flow models are biased to predict flood levels at or above historical storm levels.

10.2 Peaked and Lagged Unit Hydrographs

Peaking and lagging of the Unit Hydrographs was performed for the 21,400 square-mile downstream centered March event and the 7,980 square-mile Bulls Gap Centered March event (Reference 54). These simulations utilize unit hydrographs that were adjusted to reflect the non-linearity of the runoff generation process under field conditions. Adjustments to the unit hydrographs include increasing the peak discharge by 20% and decreasing the time-to peak by 1/3 as recommended in Appendix I of Reference 19.

The results of the peaked and lagged unit hydrographs sensitivity simulations produce an increase in elevation at BFN of 0.2 feet for the 21,400 square-mile storm event, and an increase of 0.1 feet for the 7,980 square-mile Bulls Gap Storm event. These increases are still bounded by the CLB elevation of 572.5 ft.

Adjusting the unit hydrographs for BFN is not required based on:

a. The unit hydrographs developed for the licensing basis PMF and used in the flood hazard reevaluation are developed from gage data and calibrated to historical events.

b. In the 2014 evaluation of the WBN LAR (Reference 26) and approved in the 2015 WBN SER (Reference 27) NRC performed a confirmatory check of TVA's unit hydrographs utilizing Snyder's unit hydrograph method. The NRC noted that the hydrographs computed by TVA have shorter peak time and higher peak flows when compared to the synthetic Snyder unit hydrographs. Based on reviewing the methodology and procedure used by TVA to develop the unit hydrographs the NRC staff concluded that TVA's unit hydrographs were conservative and acceptable. Therefore adjusting the peak and time to peak of the unit hydrographs is not appropriate.

10.3 Gate Operability/Blockage

The PMF analysis assumed each dam is able to fully open all gates and no debris blockage of the spillways occurs. To evaluate the sensitivity of this assumption, sensitivity to gate operability or gate blockage due to debris accumulation at dams credited for stability was performed for both the 21,400 square-mile downstream centered March event and the 7,980 square-mile Bulls Gap Centered March event. Multiple simulations were performed using the HEC-RAS unsteady flow model to compute peak water surface elevations at various dams whose outlet capacities were postulated to be reduced during the two PMF storm events. For each dam analyzed, the percentage of gate openings that may be blocked by debris and still pass the PMP without causing other failures at the dam was determined. The simulations for each dam did not provide results at BFN, only at the dam being analyzed. The dams considered include Blue Ridge, Chatuge, Cherokee, Douglas, Fontana, Fort Loudoun, Hiwassee, Norris, Nottely, South Holston, Tellico, Watauga, and Watts Bar Dams. Chickamauga, Nickajack, and Guntersville gate operability is not evaluated as these dams are assumed to fail. (Reference 54)

The dams analyzed above the BFN site have an outlet unavailability margin greater than 10% except Cherokee and Douglas in the 7,980 square-mile Bulls Gap centered event. Those two dams have an outlet unavailability margin less than 5% as measured by reaching a potential embankment overtopping elevation.

Spillway gate blockage due to maintenance issues or debris has been evaluated and determined to not represent a significant hazard for BFN. The conclusion is based on:

- TVA RO has a debris management program
- There is no history of debris blocking spillways at these dams in historically large flooding events
- There is no barge traffic above these two dams
- RO monitors gates daily for operation and the maintenance program for gates assures high reliability
- RO has the means and resources to resolve gate issues if needed to respond to flood events.
- The embankments at the dams are not threatened until floods reach PMF levels. At these levels, the flood waters at the larger reservoir dams significantly overtop the spillways gates by several feet. Floating debris would easily pass over the gates and continue downstream without blocking flow paths.

10.4 Breach Size

The embankment breach size and breach method sensitivity was evaluated for each dam with earthen embankments. The breach method selected, Von Thune and Gillette is the best suited for TVA based on the size of the dams in the system and the volume-driven nature of the Tennessee River System. Total failures of concrete structures are conservatively assumed for dam failures above Wheeler Dam when the dam stability evaluation demonstrates low margin (Reference 55).

10.5 Initial Reservoir Conditions

TVA controls and/or schedules the releases from all TVA dams and some non-TVA dams. Providing the scheduled releases from these dams enables TVA the opportunity to control the river system as an integrated system. TVA monitors the dams 24 hours a day, 7 days a week. The releases are based on the needs of the entire Tennessee River system while maintaining reservoir levels within the operating guide. The reservoir operating guidelines are implemented as prescribed operating ranges of reservoir levels throughout the year. Operating within the prescribed operating ranges provides consistency in the normal reservoir levels, therefore also providing consistency in the headwater and tailwater elevations at the dams. Reference 19 presents recommendations to evaluate the sensitivity of initial reservoir level, inflow, and tailwater conditions. In the case of the Tennessee River System, the reservoir level is strictly controlled and a conservative starting point based on known reservoir operating levels was used in the reevaluation. Tailwater variability is limited by the reservoir level relationships and is simulated within the HEC-RAS model. Thus no additional sensitivities for initial reservoir conditions were evaluated.

11 COMPARISON – CURRENT DESIGN BASIS ELEVATIONS VS. REEVALUATION RESULTS

Table 11-1 presents a comparison of the design basis flood elevations and the reevaluated results.

Table 11-1 Comparison of Current Design Basis Elevations and Reevaluation Results

Flood Causing Mechanism	Design Basis (ft.)	Reevaluation (ft.)	Bounded (Yes/No)	Comments
Local Intense Precipitation	<592	590.4	Yes	West Channel
	<578	578.2	No	East Switchyard
	<565	566.6	No	Lower Plant
Flooding from Rivers and Streams	572.5	572.1	Yes	7,980 sq-mi Bulls Gap centered event
Flooding from Dam Breaches or Failures	N/A	558.6	No	Resulting from a single seismic failure of Guntersville Dam
Flooding from Storm Surges or Seiches	N/A	N/A	N/A	Not a credible flood-causing mechanism at this site.
Flooding from Tsunamis	N/A	N/A	N/A	Not a credible flood-causing mechanism at this site
Flooding from Ice-Induced Events	N/A	N/A	N/A	Not a credible flood-causing mechanism at this site
Flooding From Channel Diversion or Migration Toward the Site	N/A	N/A	N/A	Not a credible flood-causing mechanism at this site
Flooding from Combined Effects	578.0	577.2	Yes	Design Basis was PMF plus wind waves
	N/A	560.9 ft	No	Controlling combination was the Half-10,000 Year Douglas Centered Seismic Event During a 500-Year June Flood Event.

N/A – Not applicable

12 IDENTIFICATION AND EVALUATION OF ANY INTERIM ACTIONS TAKEN TO MITIGATE HIGHER FLOOD HAZARD RELATIVE TO DESIGN BASIS

As identified in Table 11-1, reevaluation results for three flood causing mechanisms LIP, upstream dam breaches or failure, and combined effects of seismically-induced dam failure flooding are not bounded by the current design basis for BFN. The latter two mechanisms were not considered to be applicable in the CLB and thus do not have comparable CLB flood elevations.

Each of these mechanisms are evaluated below and interim actions defined if needed.

12.1 Local Intense Precipitation

The BFN LIP flooding critical elevation at the site is the 565.0 ft. floor elevation of the exterior doors leading to the Reactor Buildings, Diesel Generator Buildings, Intake Pumping Station, and Radwaste Building. In the CLB, the flood level of the LIP event did not exceed the 565.0 ft critical elevation. LIP flood water entering other site buildings will not impact safe operation of BFN.

The re-evaluation of the LIP event, as discussed in Section 9.1, determined that the flood water would potentially exceed the plant critical elevation for over an hour by as much as 1.6 feet. Because the reevaluated LIP flood hazard is not bounded by the CLB, an Integrated Assessment will be performed where the effect of the exceeded flood hazard on the plant's safety related SSCs will be examined in detail. Prior to completion of the Integrated Assessment, required interim actions needed to mitigate the potential impacts of the LIP flood hazard are evaluated as discussed in the following sections.

To evaluate the potential plant impacts due to the potential LIP flood elevation, access doors, as well as other openings at or below the LIP flood height that would allow water into the Reactor Buildings, Diesel Generator Buildings, Intake Pumping Station and Radwaste Building, were reviewed using site drawings. In addition, these potential flood water ingress paths were observed during a site walkdown performed in February 2015. Flood walls and penetrations were reviewed and determined to be acceptable for the PMF height of 578.0 ft and are acceptable for the maximum 566.6 ft LIP flood elevation.

The Reactor Building has a finished floor elevation of 565.0 ft. An equipment/personnel air lock and a personnel access door provides access on the south side of the building (Reference 56). The equipment airlock is a secondary containment boundary and consists of a large equipment door at either end (Doors 226 and 229). The equipment doors use inflatable seals to maintain an air seal and are interlocked such that only one door may be opened at a time (Reference 2). The LIP flood exceeds the finished floor elevation by only 0.2 feet (2.4 inches). Given the airtight design of the doors, the very limited flood height above the door, and the short duration of flood exposure, the amount of water leakage through an inadequate door seal will be very small and no water would be expected in the Reactor Building. A small side door (230A) provides personnel access. Door 230A is a normally closed watertight door and has a 3-1/2" threshold, which will not be exceeded by the LIP flood (Reference 57).

The Reactor Building doors on north side will potentially be exposed to flood water when the outside LIP flood levels exceed the Turbine Building floor elevation of 565.0 ft. from either the Lower East or Lower West areas. The personnel access doors (Doors 235 and 248) and equipment access doors (Doors 237 and 250) are normally closed, watertight doors designed for the PMF water height of 572.5 ft. (Reference 2). LIP flood water would not enter the Reactor Building through these doors. A small amount of water in leakage would be similar to the internal flooding analysis for the 565.0 general area. This analysis assumes rupture of a Reactor Water Cleanup System line break in the pipe trench located near the ceiling which spills over and floods the general area floor (Reference 58). Given the watertight design of the door, the limited flood height, and the short duration of exposure, the amount of leakage through an inadequate door seal will be small and would be bounded by the internal flooding analysis.

The Diesel Generator Buildings have a finished floor elevation of 565.5 feet. The LIP flood will exceed the finished floor elevation by 0.7 feet and 1.1 feet for the Unit 1/2 Diesel Generator Building and the Unit 3 Diesel Generator Building, respectively. Both buildings have five similar exterior doors (Doors 272-276 U1/U2, Doors 800-804 U2), each accessing one of the four diesel generator bays or the CO_2 room (References 59 and 60). These doors have removable seal plates extending 10 3/8 inches above the 565.0 ft. grade elevation. The external doors are normally closed, watertight doors designed for the PMF water height of 578.0 ft. (Reference 2). Little or no LIP flood water would enter the Diesel Generator Buildings through the doors. Given the watertight design of the door, the limited flood height, and the short duration of exposure, the amount of leakage through an inadequate door seal will be small and will not jeopardize diesel operation.

An emergency drain path exists in the corridor outside the diesel generator bays to drain water in the event of an 18” Emergency Equipment Cooling Water (EECW) system header break (References 61, 62, and 63). The drain path consists of a single 24” drain line for the Unit 1/2 building and two 18” drain lines for the Unit 3 building. The emergency drain lines have normally open shutoff valves (Reference 64) and are routed to valve pits just outside the buildings. The valve pits are located at site grade. LIP flood water may backflow through the emergency drain lines into the corridor and backflow through the floor drain piping into the diesel generator bay compartments. Backflow of LIP flood water into the diesel building would result in a Diesel Generator Floor Drain Sump Level High alarm in the control room. The alarm response procedure requires Operations personnel to be dispatched to investigate the cause of the alarm. If water backflows into the individual diesel compartment before the emergency drain line isolation valve is closed, accumulation of up to 6 inches could occur without impacting safety related components. The diesel generators sit on a skid 2 ft. above the floor. The diesel batteries are on a steel frame above an 8 inch concrete foundation pad and the diesel lube oil system is mounted below the skid at least 12 inches above the floor. Each diesel generator bay has a normally closed door between the corridor and the safety related components inside the diesel bay (Doors 280-284 U1/U2, Doors 805-809 U3), limiting the potential flow of water directly from the corridor into the diesel bays. For the LIP, interim actions, such as dispatch of plant operators to close the emergency drain isolation valves based on severe storm forecast will be evaluated. It is noted that for the PMF event, ample time is available and procedural direction is given to close the interior flooding drain isolation valves prior to flooding the diesel bays (Reference 9).

During the PMF, steps are taken to prevent flood water inflow to the 7-day diesel fuel tanks due to potential PMF damage to diesel fuel transfer piping located in the yard. The LIP associated effects, such as debris loads, hydrodynamic and hydrostatic loads, are expected to be negligible due to the low flow velocities and shallow water depths. Therefore, these steps are not required in response to LIP.

The truck fill boxes for the 7 day diesel fuel tanks are located outside the diesel generator building at approximately elevation 565.0 ft. (References 65 and 66). The truck load connection is rarely used, but operating procedures require the cap to be installed after any use. The truck fill boxes do not represent a potential for fuel oil contamination by flood water.

The Intake Pumping Station has a finished floor elevation of 565.0 ft. (Reference 67). The LIP flood will exceed the finished floor elevation by 1.6 feet. The Intake Pumping Station has four similar exterior doors, each accessing one of the four RHRSW pump compartments (Reference 68). The external doors are normally closed, watertight doors designed for the PMF water height of 578.0 ft. (Reference 2). Little or no LIP flood water would enter the pump compartments through the doors. The pump compartments are open at the roof and two sump pumps per compartment are provided to remove rain and other potential water inputs. A single sump pump is capable of removing the PMP with coincident RHRSW pump seal failure and gross EECW strainer leakage with margin (Reference 69). Given the watertight design of the door, the limited flood height, and the short duration of flood exposure, the amount of leakage through an inadequate door seal will be small and within the sump pump capacity margin. The LIP flood will not jeopardize RHRSW pump operation.

The Radwaste Building has a finished floor elevation of 565.0 ft. The LIP flood will exceed the finished floor elevation by 1.2 feet. There are two exterior doors to the Radwaste Building (Doors 182 and 183) and one exterior door to the Radwaste Evaporator Building (Door 383), which communicates directly to the Radwaste Building through unprotected openings (Reference 70). The external doors are watertight doors designed for the PMF water height of 578.0 ft. (Reference 2). Doors 182 and 183 are normally open and Door 383 is normally closed. There are two doors to the Radwaste Building (Doors 165 and 184) from the Service Building. Both doors are watertight and designed for the PMF water height of 572.5 ft. Door 184 is normally closed and Door 165 is normally open. Additional access to the Radwaste Building is via Turbine Building doors (Doors 202 and 203) at floor elevation 565.0 ft. and Door 25A at floor elevation 554.5 ft. (Reference 70). These doors are normally closed, watertight doors designed for the PMF water height of 572.5 ft. (Reference 2).

Given the watertight design of the doors, the limited flood height, and the short duration of exposure, little or no LIP flood water would enter the Radwaste Building through any of the closed doors. The open doors are in areas that are heavily travelled. It was concluded in a previous analysis that with Door 165 and Door 184 to the Service Building open during a LIP event, there was sufficient time to close both doors and that flood water entering the Radwaste Building would be handled by the floor drains without degrading flood safety (Reference 71). Equipment in the Radwaste Building is not considered essential to maintaining the reactors in a safe configuration.

When water enters the Turbine Building, the water will generally spread throughout the floor area, spilling into the lower elevations at 557.0 ft. and 551.0 ft. The Turbine Building does not contain safety related equipment and is allowed to flood during the PMF. Doors to the Turbine Building open outward and, although not water-tight, could be closed to severely restrict flow of water into the building.

The LIP re-evaluation of the BFN East Switchyard channel resulted in a flood elevation of up to 578.17 ft which exceeds the CLB LIP flood elevation of 578.0 ft. Overflow from the East Switchyard channel is fully contained in the Cooling Tower (CT) hot water discharge channel and in the switchyard area. There are no plant safety impacts due to this increase flow in the CT discharge channel. The tailwater effects of the increase in the discharge channels were considered in the re-evaluation of LIP floodwater run-off from the lower plant area. Overflow from the BFN East switchyard channel will also enter the switchyard area north of the plant main site. However, the elevation of the site north of the BFN turbine building is at least 578.6 which prevent the overflow from the East Switchyard channel from reaching the lower plant areas.

Therefore, no interim actions are required. Since the CLB elevation of 578.0 ft. was exceeded in the re-evaluation, an Integrated Assessment is required.

The following interim actions are for the BFN site:

a. TVA will revise existing procedures or develop new procedures to ensure that doors or other openings susceptible to a LIP flood event are maintained closed or controlled otherwise by procedure, compensatory measure, and/or work activity to protect ingress pathways in the event of a LIP. These actions will be implemented at the BFN site by July 31, 2015.
b. TVA will determine a resolution to the potential backflow through the Diesel Generator interior flooding drain lines into the diesel generator buildings during the LIP event by September 28, 2015.

12.2 Upstream Dam Breaches or Failures

The re-evaluation of upstream dam breaches or failure results in a maximum Browns Ferry site elevation of 558.8 ft. as a result of single failure of the Watts Bar dam during a 500-year flood event. Because this elevation is below the plant grade, this condition does not impact the safe operation of the Browns Ferry plant. No interim actions are required. The results associated with Section 12.3 below bound these results. Single seismic failures were not considered in the current licensing basis, therefore an Integrated Assessment will be performed.

12.3 Combined Effects Floods caused by Seismic Dam Failures

The re-evaluation of seismically induced flooding results in a maximum Browns Ferry site elevation of 560.9 ft. The combined effects of seismically induced dam failure, 500-year storm rainfall and maximum wind wave results in a flood elevation below plant grade. The safe operation of the Browns Ferry plant is not impacted. No interim actions are required. Combined effects floods caused by seismic dam failures were not considered in the current licensing basis, therefore an Integrated Assessment will be performed

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TVA Drawings 3-47E225-100, Harsh Environment Data Index, Note X, Revision 7

70. TVA Drawing 3-46W401-2, Architectural Plans 546.0 and 565.0, Revision 8

71. TVA Calculation NDQ0064920113, Flood Protection Effects of Leaving Radwaste Flood Building Doors and Reactor Building Flood Gate Normally Open, Revision 12.

ENCLOSURE 2

LIST OF COMMITMENTS

1. TVA will complete the March 12, 2012, 50.54(f) Request for Information required Integrated Assessment for Browns Ferry Nuclear Plant, Units 1, 2 and 3 and submit a report no later than March 12, 2017.

2. TVA will revise existing procedure actions or develop new procedures to ensure that doors or other openings susceptible to a Local Intense Precipitation (LIP) flood event are maintained closed or controlled otherwise by procedure, compensatory measure, and/or work activity to protect ingress pathways in the event of a LIP. These actions will be implemented at the BFN site by July 31, 2015.

3. TVA will determine a resolution to the potential backflow through the diesel generator interior flooding drain lines into the Diesel Generator Buildings during the Local Intense Precipitation (LIP) event by September 28, 2015.