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

CNL-14-228

December 5, 2014

10 CFR 50.4

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

Watts Bar Nuclear Plant Unit 1
Facility Operating License No. NPF-90
NRC Docket No. 50-390

Subject: **REQUEST FOR ADDITIONAL INFORMATION RELATED TO LICENSE AMENDMENT REQUEST TO UPDATED FINAL SAFETY ANALYSIS REPORT CHANGES ASSOCIATED WITH HYDROLOGIC ANALYSIS (TAC NO. ME9130)**

References:

1. Letter from TVA to NRC, "Application to Revise Watts Bar Nuclear Plant Unit 1 Updated Final Safety Analysis Report Regarding Changes to Hydrologic Analysis, TAC No. ME8200 (WBN-UFSAR-12-01)," dated July 9, 2012 (ADAMS Accession No. ML122360173)

2. Letter from TVA to NRC, "Supplement to Application to Revise Watts Bar Nuclear Plant Unit 1 Updated Final Safety Analysis Report Regarding Changes to Hydrologic Analysis (WBN-UFSAR-12-01)," dated March 1, 2013 (ADAMS Accession No. ML13067A393)

3. Electronic Mail from Andrew Hon (NRC) to Joseph W. Shea (TVA), "Watts Bar Nuclear Station, Unit 1 – RAI Related To LAR To Updated FSAR Changes Associated With Hydrologic Analysis From Mechanical And Civil Engineering Branch – Dam Stability Safety Factor (TAC No. ME913," dated May 9, 2014 (ADAMS Accession No. ML14129A316)

4. Electronic Mail from Siva Lingam (NRC) to Joseph W. Shea (TVA), "Watts Bar, Unit 1 Hydrology LAR with UFSAR Changes (TAC No, ME9130)," dated May 21, 2014

5. Letter from TVA to NRC, "Supplement To Application To Revise Watts Bar Nuclear Plant Unit 1 Updated Final Safety Analysis Report Regarding Changes To Hydrologic Analysis (WBN-UFSAR-12-01)" dated September 30, 2014 (ADAMS Accession Number ML14289A106)

6. Electronic Mail from Andrew Hon (NRC) to Joseph W. Shea (TVA), "Watts Bar Nuclear Station, Unit 1 – Request For Additional Information Related To License Amendment Request To Updated Final Safety Analysis Report Changes Associated With Hydrologic Analysis (TAC NO. ME9130)" dated November 18, 2014

By letter dated July 19, 2012 (Reference 1) to the Nuclear Regulatory Commission (NRC), the Tennessee Valley Authority (TVA) submitted a License Amendment Request (LAR) to revise the Watts Bar Nuclear Plant (WBN) Unit 1 Updated Final Safety Analysis Report (UFSAR) to reflect the results from new hydrologic analysis. By letter dated March 1, 2013 (Reference 2), TVA submitted a supplement to the Reference 1 LAR to reflect the identification of additional equipment that may be affected (partially submerged) during a design basis flood (DBF) event based on the updated hydrologic analysis. In addition, the March 2013 letter provided descriptions of appropriate compensatory measures and plant modifications to ensure safety-related systems, structures, and components identified in Regulatory Guide 1.29 would withstand the flood conditions associated with the updated DBF elevation, and would remain functional during external floods.

On April 23 and 24, 2014, the NRC performed an audit of the TVA dam stability calculations. As a result of the audit, on May 9, 2014, the NRC transmitted a request for additional information (RAI) by electronic mail (email) (Reference 3) to TVA requesting a technical basis to support the changes to UFSAR Section 2.4.3.4, “Probable Maximum Flood Flow,” under the “Concrete Section Analysis.” The RAI also requested that the technical basis for the proposed change include any analysis and calculations, or reference to industry standards, that supports the UFSAR change, and that using a factor of safety greater than 1.0, for sliding, provides an adequate basis that dam structures are considered safe against failure.

On May 21, 2014 (Reference 4), the NRC forwarded additional requests for information related to a preliminary TVA response to the RAI provided in Reference 3.

During subsequent discussions between TVA and the NRC staff, the NRC staff questioned actions taken by TVA to evaluate dam stability and furthermore indicated that the dams identified by TVA as requiring additional review (Boone, Cherokee, Fontana, Fort Patrick Henry, Melton Hill and Tellico) warranted additional changes to the licensing basis. As such, the dams affected are being modified or analyzed to new design and licensing basis criteria which are consistent with those outlined in the Standard Review Plan (NUREG-0800).

By letter dated September 30, 2014 (Reference 5), TVA submitted a supplement to the Reference 1 LAR to revise the WBN Unit 1 UFSAR to adopt a revised hydrologic analysis for the WBN site. The changes to the LAR incorporated updates to the hydraulic analysis methodology, including the use of the U.S. Army Corps of Engineers (USACE) Hydrology Engineering Center Hydrologic Modeling System (HEC-HMS) and River Analysis System (HEC-RAS) software, and updates to the TVA River Operations dam stability acceptance criteria for concrete and earthen dams.

On November 18, 2014 (Reference 6), the NRC forwarded additional requests for information (RAI) based on their ongoing audit and review of the LAR supplement (Reference 5).

Enclosure1 provides response to the Reference 6 RAIs, including proposed revisions to the WBN Unit 1 UFSAR resulting from the RAI responses. Enclosure 2 provides WBN Unit 1 LAR and UFSAR changes and clarifications that resulted from the NRC audit and TVA's internal review of the WBN Unit 2 FSAR Amendment.

Enclosure 3 provides proposed License Conditions related to ongoing modifications at Cherokee, Douglas, Fort Loudoun, Tellico, and Watts Bar dams, and the replacement of temporary HESCO barriers for 1900 feet of temporary HESCO barriers that has been delayed due to ongoing construction of the U.S. Highway 321 bridge across the Tennessee River to reroute traffic away from the Fort Loudoun Dam.

Please address any questions regarding this request to Gordon Arent at 423-365-2004.

I declare under penalty of perjury that the foregoing is true and correct. Executed on this 5th day of December 2014.

Respectfully,

J. W. Shea

Digitally signed by J. W. Shea
DN: cn=J. W. Shea, o=Tennessee Valley Authority, ou=Nuclear Licensing, email=jwshea@tva.gov, c=US
Date: 2014.12.05 17:12:23 -05'00'

J. W. Shea
Vice President, Nuclear Licensing

Enclosures:

1. Response to NRC Requests for Additional Information Related to License Amendment Request Associated With Hydrologic Analysis
2. LAR changes as a result of NRC Audit and TVA Identified UFSAR Changes
3. Proposed License Conditions

cc: See Page 4

cc (Enclosures):

NRC Regional Administrator - Region II
NRC Senior Resident Inspector – Watts Bar Nuclear Plant, Unit 1
NRC Senior Resident Inspector – Watts Bar Nuclear Plant, Unit 2
Director, Division of Radiological Health - Tennessee State Department of Environment and Conservation

ENCLOSURE 1

Response to NRC Requests for Additional Information Related to License Amendment Request Associated With Hydrologic Analysis

ENCLOSURE 1

NRC RAI 1.1 *Magnitude of Probable Maximum Precipitation (PMP) of same storm*

The NRC staff has found differences between the PMPs as shown on Table 2.4-11 of the September 30th submittal and other PMPs as shown on Table 2.4-5 of previous final safety analysis reports (FSARs). The NRC staff noted different values are now being used to determine the PMP, which is used as an input to the PMF. Please explain the reasons for the changes of PMPs in the current submittal while using the same 7,980 square mile storm at the same storm center as shown in the previous FSARs. To illustrate this, a part of the differing data is presented on Table 1.

Table 1: Example of comparing computed PMPs in the 2010 FSAR vs. 2014 submittal

	Antecedent Storm Rainfall Depth (inch)		*Main Storm Rainfall Depth (inch)*	
Sub-watershed name	*(Quoted from Table 2.4-5, precedent FSAR, 2010)*	*(Quoted from Table 2.4-11, September 30th 2014 Submittal)*	*(Quoted from Table 2.4-5, precedent FSAR, 2010)*	*(Quoted from Table 2.4-11, September 30th submittal, in 2014)*
Asheville	*6.44*	*6.0*	*17.40*	*10.98*
Newport, French Broad	*6.44*	*6.0*	*18.5*	*16.56*
Newport, Pigeon	*6.44*	*6.0*	*19.30*	*15.48*
Little Tennessee Local - Fontana to Chilhowee Dam	*6.44*	*6.0*	*24.00*	*15.30*
Little Tennessee Local- Chilhowee to Tellico Dam	*6.44*	*6.0*	*21.00*	*15.84*
Watts Bar Local Above Clinch River	*6.44*	*6.0*	*15.8*	*15.84*

TVA Response

Prior to submittal of the July 19, 2012 License Amendment Request (LAR), Table 2.4-5 presented 21,400 square mile precipitation data from a previous analysis. In the hydrology reconstitution project begun in 2008 and presented in the 2012 LAR, PMP data was reanalyzed with Calculation CDQ000020080053 (Reference 1). UFSAR Table 2.4-11 included in the 2012 LAR replaced UFSAR Table 2.4-5 and presented precipitation data for the 21,400 square mile downstream centered March event. In the 2014 LAR, Table 2.4-11 presents the calculated precipitation data for the 7,980 square mile Bulls Gap March event, which is the controlling event at Watts Bar Nuclear Plant.

NRC RAI 1.2 Antecedent Precipitation Index (API) method

The FSAR Section 2.4.3.1 of the September 30th submittal shows the PMP depth is 16.17 inches and the antecedent depth is 6.0 inches. FSAR Section 2.4.3.1 also states that the antecedent depth is 40 percent of the total PMP depth (6.47 inches). NRC staff requests that TVA clarify and explain the apparent discrepancy between the antecedent depth 6.0 inches and 6.47 inches, which is equal to 40 percent of the PMP depth. NRC also requests that TVA clarify the antecedent depth actually used.

In Section 2.4.3.2 of September 30th 2014 submittal, TVA states that the API numerical value was determined using the observed data from 1997 to 2007. Please provide supporting documents to illustrate use of the API method and its computed numerical values, including computational examples illustrating how the excess rainfall depth showed in Table 2.4-11 were determined.

TVA Response

As a result of this question, TVA proposes to revise the UFSAR as follows:

Revise the fifth paragraph of FSAR Section 2.4.3.1 from:

> All PMP storms are nine-day events. A three-day antecedent storm was postulated to occur three days prior to the three-day PMP storm in all PMF determinations. Rainfall depths equivalent to 40% of the main storm were used for the antecedent storms with uniform areal distribution as recommended in Report No. 41.[4]
>
> to:
>
> Storms evaluated were 9-day events with a 3-day antecedent storm postulated to occur 3-days prior to a 3-day PMP storm in all PMF determinations. As recommended in Hydrometeorological Report No. 41, the antecedent rainfall was applied using a uniform areal distribution to avoid compounding of probabilities. For the 7,980 square mile Bulls Gap centered March event, an antecedent rainfall depth of 6.00 inches was applied which is equivalent to 40% of the 14.95 inch main storm depth for the 24,452-square-mile watershed above Guntersville Dam. This meets the criteria as set forth in Hydrometeorological Report No. 41, Chapter VII.

ENCLOSURE 1

As discussed in Hydrometeorological Report No. 41 (HMR-41) (Reference 2), uniform areal distribution of antecedent rainfall was recommended. As a result, the combination of the critical basin shape and the storm pattern intensity determined the antecedent rainfall. As shown in CDQ000020080053, Appendix G (Reference 1), the antecedent rainfall applied was 40% of the sub-basin average main storm rainfall above Guntersville Dam.

In the UFSAR, a median antecedent precipitation index (API) as determined from past records was used at the start of the antecedent storm. As described in UFSAR Section 2.4.3 of the 2012 and 2014 LAR, 1997-2007 gage data was used to determine starting API values. These values represent initial mean soil moisture conditions for each sub-basin.

The following provides an illustrative example of the API method and how the computed numerical values are determined as documented in calculation CDQ000020080052 (Reference 3) :

Calculate API Example:

The week of the year and location inputs designate the appropriate API region curve to use in determining a runoff index (RI). This RI is used to forecast the surface runoff given a rainfall amount and runoff (RO) region. The United States Weather Bureau designated two RI versus RO regions for the Tennessee Valley identified as the TV and SE curves. The SE RI-RO curve is used only with the SE API region; all other API regions use the TV RI-RO curve. Data from these curves are used in the FLDHYDRO computer code in calculation of rainfall to runoff transformation.

Instructions:

1	Calculate API	Insert incremental rainfall data.
2	Calculate API	Calculate cumulative rainfall.
3	Calculate API	Calculate API at daily intervals. Values are truncated to 2 decimal places.
4	Select Antecedent RI*	Using starting antecedent API, select RI-Runoff relation from SE region API-RI table
5	Select Antecedent RI*	Using cumulative antecedent rainfall as input, use RI-Runoff table to determine cumulative runoff. Intermediate values are interpolated.
6	Select Main RI*	Using starting main API, select RI-Runoff relation from SE region API-RI table
7	Select Main RI*	Using cumulative main rainfall as input, use RI-Runoff table to determine cumulative runoff. Intermediate values are interpolated.
8	API Method	Shows Rainfall, API and runoff vs. time.

* RI = Runoff Index

Calculate API

French Broad River at Asheville

Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)	API
0	0	0.00	1.08
6	0.18	0.18	
12	0.18	0.36	
18	0.18	0.54	
24	0.18	0.72	2.77
30	1.02	1.74	
36	1.80	3.54	
42	0.72	4.26	
48	0.48	4.74	5.00
54	0.36	5.10	
60	0.36	5.46	
66	0.30	5.76	
72	0.24	6.00	5.00
78	0		
84	0		
90	0		
96	0		4.50
102	0		
108	0		
114	0		
120	0		4.05
126	0		
132	0		
138	0		
144	0	0	3.64
150	0.36	0.36	
156	0.36	0.72	
162	0.36	1.08	
168	0.42	1.50	5.00
174	2.04	3.54	
180	2.52	6.06	
186	1.38	7.44	
192	1.02	8.46	5.00
198	0.78	9.24	
204	0.66	9.90	
210	0.60	10.50	
216	0.48	10.98	5.00

Insert Incremental Rainfall

Calculate Cumulative Rainfall (previous hour + current hour)

Calculate API at daily intervals

Select Antecedent Runoff Index (RI)

Start API of 1.08	API for SE region weeks 2-11	Use RI of:
	0.12	29
	0.38	28
	0.68	27
	1.04	26
use →	1.50	25
	2.07	24
	2.75	23
	3.60	22
	4.60	21
	5.00	20

Using starting antecedent API, select RI-Runoff relation from SE region API-RI table

table values → (white cells)
interpolated values → (shaded cells)

Antecedent Event
RI = 25

	Cum. Rain (inches)	SRO
	0	0
t = 6 →	0.18	0.01
t = 12 →	0.36	0.02
t = 18 →	0.54	0.03
t = 24 →	0.72	0.04
	0.84	0.05
	1.04	0.10
	1.23	0.15
	1.42	0.20
	1.60	0.25
t = 30 →	1.74	0.29
	1.78	0.30
	1.95	0.35
	2.12	0.40
	2.28	0.45
	2.43	0.50
	2.58	0.55
	2.72	0.60
	2.86	0.65
	2.99	0.70
	3.11	0.75
	3.23	0.80
	3.35	0.85
	3.46	0.90
	3.54	0.94
t = 6 →	3.57	0.95

	Cum. Rain (inches)	SRO
t = 36 →	3.54	0.94
	3.57	0.95
	3.67	1.00
	3.77	1.05
	3.87	1.10
	3.97	1.15
	4.06	1.20
	4.15	1.25
	4.24	1.30
t = 42 →	4.26	1.33
	4.32	1.35
	4.40	1.40
	4.48	1.45
	4.56	1.50
	4.63	1.55
	4.70	1.60
t = 48 →	4.74	1.63
	4.77	1.65
	4.84	1.70
	4.90	1.75
	4.96	1.80
	5.02	1.85
	5.08	1.90
t = 54 →	5.10	1.92
	5.14	1.95

	Cum. Rain (inches)	SRO
	5.20	2.00
	5.26	2.05
	5.32	2.10
	5.38	2.15
	5.44	2.20
t = 60 →	5.46	2.22
	5.50	2.25
	5.56	2.30
	5.62	2.35
	5.67	2.40
	5.72	2.45
t = 66 →	5.76	2.49
	5.77	2.50
	5.82	2.55
	5.87	2.60
	5.92	2.65
	5.97	2.70
t = 72 →	6.00	2.74
	6.02	2.75
	6.07	2.80
	6.12	2.85
	6.17	2.90
	6.22	2.95
	6.27	3.00
	6.32	3.05

Cum. Rain (inches)	SRO
6.37	3.10
6.42	3.15
6.47	3.20
6.52	3.25
6.57	3.30
6.62	3.35
6.67	3.40
6.72	3.45
6.77	3.50
6.82	3.55
6.87	3.60
6.92	3.65
6.97	3.70
7.02	3.75
7.07	3.80
7.12	3.85
7.17	3.90
7.22	3.95
7.27	4.00
7.32	4.05
7.37	4.10
7.42	4.15
7.47	4.20
7.52	4.25
7.57	4.30
7.62	4.35

Cum. Rain (inches)	SRO
7.62	4.35
7.67	4.40
7.72	4.45
7.77	4.50
7.82	4.55
7.87	4.60
7.92	4.65
7.97	4.70
8.02	4.75
8.07	4.80
8.12	4.85
8.17	4.90
8.22	4.95
8.27	5.00
8.32	5.05
8.37	5.10
8.42	5.15
8.47	5.20
8.52	5.25
8.57	5.30
8.62	5.35
8.67	5.40
8.72	5.45
8.77	5.50
8.82	5.55
8.87	5.60

Cum. Rain (inches)	SRO
8.92	5.65
8.97	5.70
9.02	5.75
9.07	5.80
9.12	5.85
9.17	5.90
9.22	5.95
9.27	6.00
53.27	50.00

Using cumulative antecedent rainfall as input, use RI-Runoff table to determine cumulative runoff. Intermediate values are interpolated.

Select Main RI

Start API of 3.64	week 12	Use RI of:
	0.12	30
	0.40	29
	0.69	28
	1.02	27
	1.38	26
	1.87	25
	2.47	24
	3.24	23
use →	4.13	22
	5.00	21

Using starting main API, select RI-Runoff relation from SE region API-RI table

Main Event
RI = 22

table values →
interpolated →

	Cum. Rain (inches)	SRO
	0	0
	0.36	0.02
	0.72	0.05
	0.73	0.05
	0.90	0.10
	1.07	0.15
	1.08	0.15
	1.23	0.20
	1.39	0.25
	1.50	0.29
	1.54	0.30
	1.68	0.35
	1.82	0.40
	1.96	0.45
	2.10	0.50
	2.23	0.55
	2.36	0.60
	2.48	0.65
	2.60	0.70
	2.71	0.75
	2.82	0.80
	2.93	0.85
	3.04	0.90
	3.15	0.95
	3.25	1.00
	3.35	1.05
	3.44	1.10
	3.53	1.15

	Cum. Rain (inches)	SRO
t = 174 →	3.54	1.16
	3.62	1.20
	3.70	1.25
	3.78	1.30
	3.86	1.35
	3.94	1.40
	4.01	1.45
	4.08	1.50
	4.15	1.55
	4.22	1.60
	4.28	1.65
	4.34	1.70
	4.40	1.75
	4.46	1.80
	4.52	1.85
	4.58	1.90
	4.63	1.95
	4.68	2.00
	4.73	2.05
	4.78	2.10
	4.83	2.15
	4.88	2.20
	4.93	2.25
	4.98	2.30
	5.03	2.35
	5.08	2.40
	5.13	2.45
	5.18	2.50

	Cum. Rain (inches)	SRO
	5.23	2.55
	5.28	2.60
	5.33	2.65
	5.38	2.70
	5.43	2.75
	5.48	2.80
	5.53	2.85
	5.58	2.90
	5.63	2.95
	5.68	3.00
	5.73	3.05
	5.78	3.10
	5.83	3.15
	5.88	3.20
	5.93	3.25
	5.98	3.30
	6.03	3.35
t = 180 →	6.06	3.38
	6.08	3.40
	6.13	3.45
	6.18	3.50
	6.23	3.55
	6.28	3.60
	6.33	3.65
	6.38	3.70
	6.43	3.75
	6.48	3.80
	6.53	3.85

	Cum. Rain (inches)	SRO
	6.58	3.90
	6.63	3.95
	6.68	4.00
	6.73	4.05
	6.78	4.10
	6.83	4.15
	6.88	4.20
	6.93	4.25
	6.98	4.30
	7.03	4.35
	7.08	4.40
	7.13	4.45
	7.18	4.50
	7.23	4.55
	7.28	4.60
	7.33	4.65
	7.38	4.70
	7.43	4.75
t = 186 →	7.44	4.76
	7.48	4.80
	7.53	4.85
	7.58	4.90
	7.63	4.95
	7.68	5.00
	7.73	5.05
	7.78	5.10
	7.83	5.15
	7.88	5.20

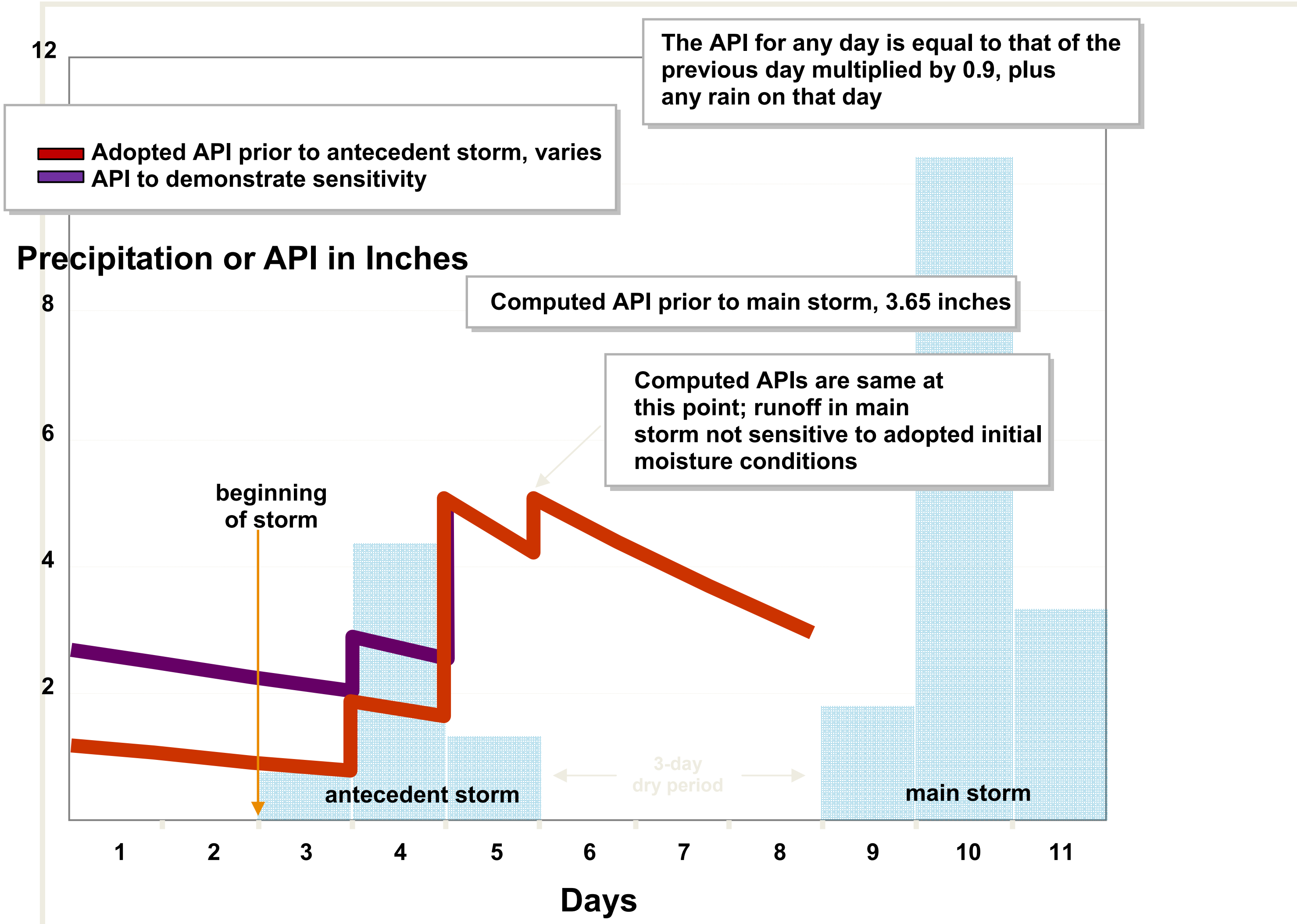
	Cum. Rain (inches)	SRO
	7.93	5.25
	7.98	5.30
	8.03	5.35
	8.08	5.40
	8.13	5.45
	8.18	5.50
	8.23	5.55
	8.28	5.60
	8.33	5.65
	8.38	5.70
	8.43	5.75
t = 192 →	8.46	5.78
	8.48	5.80
	8.53	5.85
	8.58	5.90
	8.63	5.95
	8.68	6.00
t = 198 →	9.24	6.56
t = 204 →	9.90	7.22
t = 210 →	10.50	7.82
t = 216 →	10.98	8.30
	52.68	50.00

Using cumulative antecedent rainfall as input, use RI-Runoff table to determine cumulative runoff. Intermediate values are interpolated.

API Method – Shows Runoff, API, and Runoff v. Time

French Broad River at Asheville					
Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)	API	RI	Cumulative Runoff (inches)
0	0	0.00	1.08	25	0.00
6	0.18	0.18		25	0.01
12	0.18	0.36		25	0.02
18	0.18	0.54		25	0.03
24	0.18	0.72	2.77	25	0.04
30	1.02	1.74		25	0.31
36	1.80	3.54		25	0.94
42	0.72	4.26		25	1.31
48	0.48	4.74	5.00	25	1.63
54	0.36	5.10		25	1.92
60	0.36	5.46		25	2.22
66	0.30	5.76		25	2.49
72	0.24	6.00	5.00	25	2.73
78	0				
84	0				
90	0				
96	0		4.50		
102	0				
108	0				
114	0				
120	0		4.05		
126	0				
132	0				
138	0				
144	0	0	3.64	22	
150	0.36	0.36		22	0.02
156	0.36	0.72		22	0.05
162	0.36	1.08		22	0.15
168	0.42	1.50	5.00	22	0.29
174	2.04	3.54		22	1.16
180	2.52	6.06		22	3.38
186	1.38	7.44		22	4.76
192	1.02	8.46	5.00	22	5.78
198	0.78	9.24		22	6.56
204	0.66	9.90		22	7.22
210	0.60	10.50		22	7.82
216	0.48	10.98	5.00	22	8.30

The following figure illustrates the computed API results prior to the main storm and demonstrates that runoff in the main storm is not sensitive to initial moisture conditions:



NRC RAI 1.3 Hydrologic Engineering Center-Hydrologic Modeling System C-(HEC-HMS) model for PMF event

In Section 2.4.3 on page E1-5 of 38, Probable Maximum Flood on Streams and Rivers, TVA indicated that the HEC-HMS model was applied in the modeling of Probable Maximum Flood. But, in the October 8th meeting, TVA verbally indicated that the HEC-HMS model was not used in the PMF simulation. NRC staff requests that TVA clarifies Section 2.4.3 on page E1-5 of 38 if the HEC-HMS was not used in the PMF event. If HEC-HMS is not used in the PMF, TVA needs to elaborate on the other methods that are adopted to convert the precipitation to surface runoff for the PMF event.

TVA Response:

In response to NRC's request, TVA proposes to revise LAR Enclosure 1, Attachment 1, Evaluation of Proposed Changes, Section 2.4.3 on page E1-5 of 38, Probable Maximum Flood on Streams and Rivers, to read:

The following technical changes are proposed for WBN Unit 1 UFSAR Section 2.4.3 to update the discussion of PMF on streams and rivers to reflect the most current information available as inputs, and to use updated methodologies such as the USACE HEC-HMS and USACE HEC-RAS software for elements of the hydrologic analysis for determining the PMF for streams and rivers for WBN Unit 1. Only HEC-RAS is used for computing the PMF elevation. Use of HEC-HMS is limited to compute the outflow hydrograph from the postulated failure of tributary dams in Section 2.4.4 for seismic dam failures. HEC-HMS was also used to validate the unit hydrographs. UNITGRPH was used to develop the unit hydrographs. Effective rainfall is obtained from observed rainfall data with the FLDHYDRO program. TRBROUTE is only used in Section 2.4.4 for routing tributaries in the seismic dam failure simulations. The SOCH Model is used for seismic dam failure simulations and warning time analysis. Additional editorial changes are shown in the mark-ups provided in Attachment 1.

NRC RAI 1.4 Unit Hydrograph

As stated in the markup version of the FSAR Section 2.4.3, page 2.4-29, there is unpublished work by TVA that supports the assumption of linearity of unit hydrographs for large, out-of-bank floods produced by major, basin-wide storms. Please explain why this statement is being removed from the current FSAR (see Pages 2.4-29 through 2.4-30).

Please include in your response the quantitative justification that the watershed responds linearly at WBN when unit hydrograph peaking is applied. Please provide the results to show whether 25% peaking and reduction of one-third to the time to peak of unit hydrographs has any impact on the computed PMF hydrograph at WBN.

TVA Response

During the TVA 2008 hydrology reconstitution project, non-linearity of the unit hydrographs (UHs) was reviewed. The sub-basins delineated for this analysis were based on gage locations to use observed data. This model basis reduces unnecessary complexity in areas where intermediate results are not needed and avoids the compounding of errors that would be present in synthetic approximations. The end result is a better model approximation of observed events. Sub-basin sizes ranged from a minimum of 29.3-square-miles to a maximum of 2912.8-square-miles with an average area of 483.6-square-miles and are primarily forested in upland areas with considerable vegetation on the floodplain areas.

A literature review shows that most of the research done on non-linearity is based on small scale watersheds of homogeneous land use. Research reported in References 1 and 2 are very often used as the basis for nonlinear basin response in any condition. Both references used the same data from an 11 hectare (27.2 acre) agricultural watershed of uniform character with little or no floodplain. Reference 2 states in General Considerations, "*It must also be stressed that the application of the NLUH* (nonlinear unit hydrograph) *technique to medium and large watersheds was not a primary objective of this paper ... Furthermore, the difficulty in derivation of UH for large watersheds is partly due to uncertainties associated with spatial variability of precipitation and infiltration which makes determination of excess rainfall a major challenge for future extension of nonlinear UH concept.*" This statement acknowledges the methodology should be limited to basins having a size and land use similar to those used in the study.

Reference 3 analysis postulates that overland flow effects dominate in smaller watersheds while the channelized conveyance system with turbulent flow controls the response of larger drainage basins. The study concludes that while the reviewed measure of nonlinearity could not clearly be related strictly to either basin area or geomorphic watershed characteristics, the nonlinear effects in the transformation of rainfall to runoff decrease with increasing basin size in the watersheds reviewed. This is supported by Reference 4 which states, "*In drainage basins with large floodplains and vegetation or other obstructions within the high banks and on overbank areas, average velocities are likely to remain fairly constant or even decrease to some extent as flow increases."*

Because the Tennessee River watershed is so large, the stacking of unknown uncertainties was minimized. Reference 4 states that "Choice of form and degree of nonlinearity involves considerable uncertainty," and that "…the use of an adequate base of observed data may lead to greater accuracy and is more important than using a complex model." The sub-basin input data development followed these ideas by developing UHs from observed gage data from the larger available gaged events.

As previously provided in the 2009 expert panel workshop (NRC Adams ML091910253) and 2010 NRC Hydrology Workshop presentation information, the peaking of the UHs was reviewed to validate the above hypotheses. The results showed that the calculated model differences were negligible.

For the calculations supporting TVA Watts Bar Unit 2:

- Model sub-basin sizes are very large compared to those used in non-linearity studies,
- Studies indicate that non-linearity decreases with increasing basin size,
- Use of a peaking factor not determined or supported by watershed data would introduce unknown uncertainties,
- Observed data from gaged events were used in UH development, and
- Internal model runs with peaked UHs showed negligible differences.

Therefore, adjustments for non-linearity were judged unnecessary and inappropriate for use in development of inflows for the PMF analysis.

References:

1. Minshall, N. E., Predicting storm runoff on small experimental watersheds, J. Hydraul Div. Am. Soc. Civ. Eng., 86(HY8), 17-38, 1960.
2. Saghafian, Bahram, "Nonlinear Transformation of Unit Hydrograph", *Journal of Hydrology*, Vol. 330 (2006): 596-603.
3. Wang, C. T., V. K. Gupta, and E. Waymire, A geomorphologic synthesis of nonlinearity in surface runoff, Water Resour. Res., 17(3), 545– 554, 1981.
4. Pilgrim D.H. and I. Cordery. 1993. "Flood Runoff." Chapter 9 in *Handbook of Hydrology*, D.R. Maidment (ed.). McGraw-Hill Book Company. New York, NY.

NRC RAI 1.5 Selected flood event

In FSAR Section 2.4.2.1, there is a table of historic floods. Examination of this table shows the maximum flood in May 8, 1984 had a Water Surface Elevation of 698.23 feet at the tail-water of the Watts Bar Dam. Explain why this storm was not included in calibration data set.

TVA Response

The model calibrations were used not only to compare the model calculated flow and stage values with the gaged values for the larger events of record, but also to validate the local unit hydrographs indirectly against the gaged data. Consequently, the larger inflow events are selected for use in validating these local unit hydrographs. Review of the Estimated Local Flow (ELF) data for the Chickamauga local inflows shows the one-day volumes for the 1973 and 2003 storms used were ranked 1 and 3, respectively. For the two and three-day volumes, the 2003 and 1973 storms were ranked 1 and 2, respectively. The May 8, 1984 storm one-day volume was ranked 29th and the two-day and three-day volumes were ranked 20th. Therefore, the recorded May 1984 Watts Bar Dam tailwater elevation is likely due to a large upstream event with resulting high Watts Bar Dam discharges, and not due to large inflows within the Chickamauga local watershed.

NRC RAI 2.1 Hydrologic Engineering Center-River Analysis System (HEC-RAS) model calibration for segment between Watts Bar Dam and Fort Loudoun Dam

As presented in Section 2.4.3.3.5.1 and Figures 2.4-12 through 2.4-20, TVA selected 1973 and 2003 flood events to calibrate HEC-RAS model for the Tennessee River system. For calibration purposes, the river system was divided into many river segments. Each segment contains dams at upstream and downstream boundaries. For example, the Watts Bar segment was assigned for calibrating the river hydraulic condition between Fort Loudoun Dam and Watts Bar Dam. Since TVA set the observed flood elevations at the headwater of the Watts Bar Dam, the HEC-RAS model cannot compute the flood elevation at the dam. Therefore, the calibration results cannot show the difference between the computed and observed flood elevations at the Watts Bar Dam (Figure 2.4-17, Sheet 1 of 2).

Please provide validation for the computed flood elevation at the headwater of Watts Bar Dam using the observed flow rates of 1973 and 2003 flood events as the downstream boundary condition. The validation should be able to assure the computed flood elevations at the Watts Bar Dam are conservative within 1 foot to meet the calibration rule as indicated in Section 2.4.3.3.5.1.

TVA Response

TVA has historically used and continues to use observed elevation hydrographs as downstream boundaries for unsteady flow model calibration for the following reasons:

- This approach allows calibration to upstream gaged (observed elevation only) locations without the need to rectify computed downstream elevation issues due to potentially suspect observed data.
- Elevations modeled in the immediate reach upstream of a rating curve controlled section are relatively insensitive to Manning's N values, but are very dependent on accurate storage reach modeling (water volume balancing) of a relatively flat water surface profile.
- Focus of effort is the Manning's N calibration of upper reservoir reaches due to:
  - need to accurately model higher sloped profile storage (since off-channel reach storage is already accounted for with engineered ineffective flow areas).
  - accuracy needed for tailwater modeling due to potentially high upstream dam submergence during PMF modeling.

A volume check computation indicates that the recorded Watts Bar Dam discharges during the March 1973 storm event are suspect. An investigative analysis of the potential adverse effects of using these suspect discharges for downstream boundary calibration leads to the following:

- Manning's N values that are physically unreasonable for the channel.
- Modeled Watts Bar Dam 1973 elevations that are still considerably below the observed elevations even with "extreme" Manning's N adjustments.
- Modeled Watts Bar Dam 2003 elevations that deviate further from the observed elevations.

The approach of using the observed elevation hydrographs as the downstream boundary for the 1973 and 2003 unsteady flow model calibration produces results that meet the acceptance criteria indicated in Section 2.4.3.3.5.1.

NRC RAI 2.2 Fluctuation of computed flow rates shown in the calibration results

Please explain a deviation as shown on Figure 2.4-14 (Sheet 4 of 5), which appears to indicate a bias between the observed data and the HEC-RAS simulation results. Also, explain why the sharp variation of the flow rates at Watts Bar Dam results in smooth variation of the flood elevations as shown on Figure 2.4-17 (Sheet 1 of 2 and Sheet 2 of 2).

TVA Response

During the 2003 flood event, Cherokee was pulsing turbines between zero and 2,000 cubic feet per second (cfs). Using this inflow as an upstream boundary in the calibration resulted in model instability as a result of the channel running dry. The Cherokee discharge was augmented by several thousand cfs to keep the model stable. This augmented flow was removed further downstream to maintain the correct inflow volume.

The sharp variation in computed discharge at Watts Bar Dam shown on UFSAR Figure 2.4-17 is a result of HEC-RAS performing a water balance of the inflows and the observed stage which is set as the downstream boundary.

NRC RAI 2.4 Calibrated geometry

In CDQ000002014000018 HEC-RAS Tributary Model Calibration of Enclosure 11, there are references to calibrated geometry. Please explain and clarify what this means.

TVA Response

In the HEC-RAS Tributary Geometry Calculation, CDQ000002014000018 (Reference 4), the reference to calibrated geometry is inclusive of a number of previously developed HEC-RAS geometry files. Cross sections and initial Manning's n-values calibrated to steady state flows were derived from the SOCH Geometry Verification Calculations. Calibrated storage areas were derived in the HEC-RAS Main Stem Geometry Calculation, CDQ0000002012000004 (Reference 5), and HEC-RAS Tributary Geometry Calculation.

NRC RAI 3.1 Safety Factor 1.52 for Slope Failure at the Embankment of Watts Bar Dam

In Section 2.4.4.1 of the 2012 license amendment request submittal, TVA revised the safety factor from 1.52 to 1.0 for the embankment of Watts Bar Dam under an operating basis earthquake (OBE) combined with ½ PMF. In the September 30th submittal in 2014, TVA changed the safety factor of greater than 1.0 back to 1.52 as the same safety factor from 1977's condition. This 1.52 safety factor remains as one of the unresolved issues without updated loadings and embankment configurations.

The NRC staff requests that TVA update the computations according to the OBE concurrent with ½ PMF condition. TVA needs to present the updated slope failure analysis for Watts Bar dam embankment to NRC Staff for review. The updated safety factor needs to be addressed in Section 2.4.4.1.

TVA Response

For the 2012 LAR submittal, TVA assessed the Watts Bar east embankment dam for the slightly increased headwater and tailwater elevations. The increase in headwater and tailwater elevations was judged to not degrade the factor of safety to the point where the acceptance criteria would no longer be met; i.e., the margin in the original factor of safety of 1.52 is large enough to judge the updated headwater and tailwater elevation would not cause the factor of safety to go below 1.0 which is the acceptance criteria for loading during the earthquake. The proposed wording in the 2012 LAR submittal was to reflect the acceptance criteria only and not a quantitatively assessed factor of safety.

To address recent Staff questions, TVA recently completed simplified analyses on the current embankment configuration with the revised headwater and tailwater elevations. These calculations were completed to a different methodology than the licensing basis but are used for a more quantitative demonstration of the factor of safety rather than the qualitative assessment that TVA performed to support the 2012 LAR submittal.

The licensing basis methodology is the standard slip circle method which is also referred to as the Swedish slip circle or ordinary method of slices. The original calculations were manually completed using an analytical-graphical procedure. The earthquake amplification was increased linearly from the toe of the dam to the crest by a factor of 2 and the acceleration at each slice was different. In addition, this method also ignores interslice normal and shear forces.

Calculations recently performed for this dam were completed by the computer program GeoStudio's SLOPE/W which uses the Bishop, Ordinary and Janbu methodology to approximate the original methodology. Some differences to note are the maximum acceleration for the crest of the embankment dam is used for each slice instead of linearly varying the acceleration. Taking the maximum acceleration throughout the embankment is conservative. The current configuration of the embankment was evaluated with the updated headwater and tailwater elevations. The lowest factor of safety determined from these simplified studies was 1.22 therefore demonstrating the factor of safety for the Watts Bar dam is greater than the acceptance criteria (FS=1.0) for loading during the earthquake. Therefore, Watts Bar dam is considered stable in the OBE seismic load case when combined with a 1/2 PMF.

TVA proposes to revise the last sentence of the first paragraph of Section 2.4.4.1, OBE Concurrent With One-Half the Probable Maximum Flood, Watts Bar Dam from:

> The original slip circle analysis of the earth embankment section results in a factor of safety of 1.52, and the embankment is judged not to fail.

to:

> The original slip circle analysis of the earth embankment section results in a factor of safety of greater than 1.2, and the embankment is judged not to fail since the factor of safety is above the acceptance criteria factor of safety of 1.0 during the earthquake.

NRC RAI 3.2 Usage of SOCH model

On Page E1-6 and 7 of 38, TVA stated that SOCH is still used and provided SOCH documentation in Appendix 2.4A. The SOCH model is used for potential flood levels due to seismically induced dam failures, loss of downstream dams, and warning times. NRC staff requests TVA confirm that the HEC-RAS is only used for computing PMF elevation, and that the SOCH is only used for computing ½ PMF and 25-year flood elevation concurrences with seismic events.

TVA Response

TVA confirms that HEC-RAS is only used for computing PMF elevation. SOCH is used for computing ½ PMF and 25-year flood elevation concurrences with seismic events, loss of downstream dams, and warning times**.**

References:

1. CDQ000020080053, PMF Inflow Determination, Revision 1
2. Hydrometeorological Report No. 41, Probable Maximum and TYA Precipitation over the Tennessee River Basin above Chattanooga
3. CDQ000020080052, API and Rain Runoff Relationship for Tennessee River Watershed, Revision 2
4. CDQ000002014000018, HEC-RAS Tributary Model Calibration, Revision 0
5. CDQ0000002012000004, HEC-RAS Main Stem Geometry Development - Main Stem, Revision 1

ENCLOSURE 2

LAR changes as a result of NRC Audit and TVA identified UFSAR Changes

TVA proposes to revise LAR Enclosure 1, Attachment 1, Evaluation of Proposed Changes, Section 2.4.14 Flooding Protection Requirements, fifth paragraph, Page E1-29 of 38, to read:

The Intake Pumping Station (IPS) is designed to have the Essential Raw Cooling Water (ERCW) System and the High Pressure Fire Protection (HPFP) System remain fully function for the DBF. The revised DBF elevation for the critical face of the IPS results in the possibility of flooding of the IPS impacting ERCW equipment required for flood mode operation located on elevation 722 ft. The IPS structure contains various equipment required to support the ERCW and HPFP systems. The IPS contains the ERCW and HPFP pumps, travelling water screens and support equipment including screen wash pumps, ERCW strainers and support equipment including backwash valves and pressure indicators, and HPFP strainers and support equipment including backwash valves and pressure indicators. During a DBF event, surge is accounted for by considering the sum of the wind wave and runup on the critical face of the IPS combined with the PMF stillwater elevation, which conservatively results in an internal flood elevation of 741.7 ft for the IPS. While this does not wet any flood-sensitive equipment on elevation 741.0 ft, the ERCW strainers and support equipment are located on elevation 722.0 ft of the IPS, connected to elevation 741.0 ft via stairwells and doors W001 and W002 at elevation 741.0 ft. The critical elevation of flood-sensitive equipment located on elevation 722.0 ft is approximately 18 inches above the floor elevation. Doors W001 and W002 both have 0.5 ft concrete berms at the opening to elevation 741.0 ft, which raises the critical elevation for floodwaters to be capable of wetting elevation 722.0 ft to elevation 741.5 ft. A plant modification to install passive flood barriers that protects the equipment in the IPS to design basis flood levels has been completed. ~~As a result of this increase, a compensatory measure of staged sandbags to be constructed into a berm at any time prior to or during the event of a Stage I flood warning has been implemented. These sandbags will be constructed into a berm at least 12 inches in height to prevent water intrusion to elevation 722.0 ft.~~ Additionally, two non-safety related sump pumps in each of the ERCW Train A and B strainer rooms, connected to safety-related power sources, are available to expel water leakage to this elevation outside the structure. ~~TVA's established corrective action program requirements are being implemented to address the need for additional compensatory measures necessary to provide flood protection for the IPS internal systems and components. A plant modification to install passive flood barriers will protect the equipment in the IPS to design basis flood levels.~~

UFSAR Changes:

TVA proposes to revise UFSAR Section 2.4, first paragraph, to read:

2.4 HYDROLOGIC ENGINEERING

Watts Bar Nuclear Plant (WBN) is located on the west bank of Chickamauga Lake at Tennessee River Mile (TRM) 528 with plant grade at elevation 728.0 ft MSL. The plant has been designed to have the capability for safe shutdown in floods up to the computed maximum water level, in accordance with regulatory position 2 of Regulatory Guide 1.59, Revision 2, August 1977, as described in Section 2.4.14.

TVA proposes to revise UFSAR Section 2.4.2.2, first paragraph, revise to read:

2.4.2.2 Flood Design Considerations

TVA has planned the Watts Bar project to conform with Regulatory Guide 1.59 including position 2 as described herein and in Section 2.4.14.

TVA proposes to revise UFSAR Section 2.4.3.3 Runoff and Stream Course Model, page 2.4-16 to read:

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

TVA proposes to revise UFSAR APPENDIX 2.4A - Runoff and Stream Course Model, to read:

Unit hydrographs were developed for each unit area for which discharge records were available from maximum flood hydrographs either recorded at stream gaging stations or estimated from reservoir headwater elevation, inflow, and discharge data using the procedures described by Newton and Vineyard Reference 1. For non gaged unit areas ~~synthetic~~ unit graphs were developed from relationships of unit hydrographs from similar watersheds relating the unit hydrograph peak flow to the drainage area size, time to peak in terms of watershed slope and length, and the shape to the unit hydrograph peak discharge in cfs per square mile. Unit hydrograph plots are provided in Figure 2.4-10 (11 Sheets). Table 2.4-13 contains essential dimension data for each unit hydrograph.

TVA proposes to revise the first sentence of first paragraph of UFSAR Section 2.4.14.2.2 from:

> Fuel in the spent fuel pool is cooled by the Spent Fuel Pool Cooling and Cleanup System (SFPCCS), the active components of which are located above flood waters”

to:

> Fuel in the spent fuel pool is cooled by the Spent Fuel Pool Cooling and Cleanup System (SFPCCS), the active components of which are either located above flood waters or are physically protected from the flood waters.

TVA proposes to revise UFSAR Section 2.4.14.8.1, second paragraph, from:

> The WBN flood warning plan provides a minimum of 27 hours to prepare for operation in the flood mode, 3 hours more than the 24 hours needed.

to:

> The WBN flood warning plan provides a minimum of 27 hours to prepare for operation in the flood mode, 10 hours for Stage I and 17 hours for Stage II preparations.

TVA proposes to revise Revise the table in UFSAR Section 2.4.14.1.1 as noted:

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

TVA proposes to Add footnotes to UFSAR Table 2.4-11 as shown:

Table 2.4-11 Probable Maximum Storm Precipitation and Precipitation Excess
(Page 1 of 2)

		Antecedent Storm		Main Storm	
Sub-Basin[a]	Name	Rainfall (inches)	Excess[b] (inches)	Rainfall (inches)	Excess[c] (inches)
1	French Broad River at Asheville	6.00	2.73	10.98	8.30
2	French Broad River, Newport to Asheville	6.00	3.51	16.56	14.57
3	Pigeon River at Newport	6.00	2.73	15.48	12.80
4	Nolichucky River at Embreeville	6.00	3.51	15.42	13.43
5	Nolichucky local, Embreeville to Nolichucky Dam	6.00	3.51	21.06	19.07
6	Douglas Dam local	6.00	4.26	26.70	25.48
7	Little Pigeon River at Sevierville	6.00	3.65	20.22	18.23
8	French Broad River local	6.00	3.65	24.00	22.01
9	South Holston Dam	6.00	4.43	16.86	15.64
10	Watauga Dam	6.00	3.51	16.26	14.27
11	Boone local	6.00	3.65	19.68	17.69
12	Fort Patrick Henry	6.00	4.43	23.34	22.12
13	North Fork Holston River near Gate City	6.00	4.43	17.64	16.42
14&15	Cherokee and Holston River below Fort Pat & Gate City	6.00	4.43	24.30	23.08
16	Holston River local, Cherokee Dam to Knoxville gage	6.00	4.43	21.66	20.44
17	Little River at mouth	6.00	3.65	20.16	18.17
18	Fort Loudoun local	6.00	3.65	20.16	18.17
19	Little Tennessee River at Needmore	6.00	2.55	11.58	8.90
20	Nantahala	6.00	2.55	11.70	9.02
21	Tuckasegee River at Bryson City	6.00	2.73	13.50	10.82
22	Fontana local	6.00	2.73	14.76	12.08
23	Little Tennessee River local, Fontana Dam to Chilhowee Dam	6.00	2.73	15.30	12.62
24	Little Tennessee River local, Chilhowee Dam to Tellico Dam	6.00	2.73	15.84	13.16
25	Watts Bar local above Clinch River	6.00	3.65	15.84	13.85
26	Clinch River at Norris Dam	6.00	4.43	16.50	15.28
27	Melton Hill local	6.00	4.10	18.00	16.59
33	Clinch River local above mile 16	6.00	4.26	16.62	15.21
34	Poplar Creek at mouth	6.00	4.26	16.26	14.85
35	Emory River at mouth	6.00	4.26	12.24	10.83
36	Clinch River local, mouth to mile 16	6.00	4.26	15.48	14.07
37	Watts Bar local below Clinch River	6.00	4.26	13.20	11.79
38	Chatuge Dam	6.00	2.73	10.50	7.82
39	Nottely Dam	6.00	2.73	10.14	7.46
40	Hiwassee River local below Chatuge and Nottely	6.00	2.55	12.18	9.50

Table 2.4-11 Probable Maximum Storm Precipitation and Precipitation Excess (Page 2 of 2)

		Antecedent Storm		Main Storm	
Sub-Basin	Name	Rainfall (inches)	Excess (inches)	Rainfall (inches)	Excess (inches)
41	Apalachia local	6.00	3.65	12.42	10.43
42	Blue Ridge Dam	6.00	2.73	9.48	6.80
43	Ocoee No. 1 local, Ocoee No. 1 to Blue Ridge Dam	6.00	2.73	11.40	8.72
44A	Hiwassee River local, Charleston gage at mile 18.9 to Apalachia and Ocoee No. 1 Dams	6.00	3.65	12.78	10.79
44B	Hiwassee River local, mouth to Charleston gage at mile 18.9	6.00	4.10	12.00	10.59
45	Chickamauga local	6.00	4.10	11.52	10.11

a. Unit area corresponds to Figure 2.4-9 numbered areas.
b. Adopted antecedent precipitation index prior to antecedent storm varies by unit area, ranging from 0.78-1.29 inches
c. Computed antecedent precipitation index prior to main storm, 3.64 inches.

TVA proposes to delete UFSAR Table 2.4-13

ENCLOSURE 3

Watts Bar Nuclear Plant, Unit 1
Proposed License Conditions

TVA proposes to add the following license conditions to the WBN, Unit 1 Facility Operating License (NPF-90):

2\. C. This license shall be deemed to contain and is subject to the conditions specified in the Commission's regulations set forth in 10 CFR Chapter I and is subject to all applicable provisions of the Act and to the rules, regulations, and orders of the Commission now or hereafter in effect; and is subject to the additional conditions specified or incorporated below:

(9) Permanent Dam Modifications

(a) The Tennessee Valley Authority (TVA) will take actions to ensure the stability of the Tellico Dam, Watts Bar Dam, Watts Bar West Saddle Dike, Fort Loudoun Dam, Cherokee Dam, Douglas Dam, and required Douglas Saddle Dams under nuclear probable maximum flood conditions, consistent with TVA's River Operations acceptance criteria. These actions shall be completed prior to implementing the revised hydrologic analysis for the WBN site, including changes to the hydraulic analysis methodology and updates to the TVA River Operations dam stability acceptance criteria by May 31, 2015.

(b) TVA shall implement permanent modifications to prevent overtopping of the embankments of the Fort Loudoun Dam due to the Probable Maximum Flood by February 1, 2017.