ATTACHMENT 1

FLOODING HAZARD REEVALUATION REPORT FOR RESOLUTION OF FUKUSHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1: FLOODING

March 2013

VIRGINIA ELECTRIC AND POWER COMPANY
NORTH ANNA POWER STATION UNITS 1 AND 2

North Anna Power Station Units 1 & 2 Flooding Hazard Reevaluation Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Flooding

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Introduction

Following the accident at the Fukushima Dai-ichi nuclear power plant resulting from the 2011 Great Tohoku Earthquake and tsunami, the NRC established the Near-Term Task Force (NTTF) and tasked it with conducting a systematic and methodical review of NRC processes and regulations to determine whether improvements were necessary.

The NTTF report concluded that continued U.S. nuclear plant operation did not pose an imminent risk to public health and safety, and provided a set of recommendations to the Commission. The Commission directed the Staff to determine those recommendations that should be implemented without unnecessary delay (Staff Requirements Memorandum (SRM) on SECY-11-0093).

The NRC issued its request for information pursuant to 10 CFR 50.54(f) on March 12, 2012, based upon the following NTTF flood-related recommendations:

- Recommendation 2.1: Flooding
- Recommendation 2.3: Flooding

Enclosure 2 to the NRC 50.54(f) letter addressed Recommendation 2.1 and requested a written response from licensees:

1 To gather information with respect to NTTF Recommendation 2.1, as amended by SRM on SECY-11-0124 and SECY-11-0137, and the Consolidated Appropriations Act, for 2012, Section 402, to reevaluate seismic and flooding hazards at operating reactor sites.

2 To collect information to facilitate NRC's determination if there is a need to update the design basis and systems, structures, and components (SSCs) important to safety to protect the updated hazards at operating reactor sites.

3 To collect information to address Generic Issue (GI) 204 regarding flooding of nuclear power plant sites following upstream dam failures.

This report is prepared in response to the March 12, 2012, 50.54(f) letter to provide information on the reevaluation of external flooding hazards at North Anna Nuclear Station Units 1 & 2 using present day methodologies, data and guidance. Flooding hazards from external sources for the North Anna site and vicinity have been evaluated recently in support of the Early Site Permit (ESP) and Combined License Application (COLA) for a future unit (Unit 3) to be located adjacent to Units 1 & 2 within the plant's property boundary. The approach and methods used for the North Anna Units 1 & 2 external flooding reevaluation are the same as the ESP and COLA analyses, which are consistent with the standards and requirements of present-day regulatory and industry guides, in particular, NUREG/CR-7046, NUREG 0800 and ANSI/ANS-2.8-1992. The results of the flooding hazard reevaluation, augmented by recent site specific information, are compared to the current design basis for the plant, which is documented in the Updated Final Safety Analysis Report (UFSAR).

1.0 Site Information Related to Flood Hazard

1.1 Detailed Site Information

The North Anna Power Station (NAPS) is located in the northeastern portion of Virginia in Louisa County as shown on Figure 1.1-1. The site is on a peninsula on the southern shore of Lake Anna at the end of State Route 700. The earth dam that creates Lake Anna is about 5 miles southeast of the site. The North Anna River flows southeasterly, joining the South Anna to form the Pamunkey about 27 miles southeast of the site.

Regionally, the site is approximately 40 miles north-northwest of Richmond, Virginia; 36 miles east of Charlottesville, Virginia; 22 miles southwest of Fredericksburg, Virginia; and 70 miles southwest of Washington, D.C. Highways U.S. Route 1 and Interstate 95, the two principal highways joining Richmond with the rest of the eastern corridor, pass within 15 and 16 miles, respectively, east of the site.

The plant property comprises 1803 acres, of which about 760 acres are covered by water. Virginia Electric and Power Company (VEPCO) et al. owns and controls all of the land within the site boundary, both above and beneath the surfaces, including those portions of the North Anna Reservoir and Waste Heat Treatment Facility that lie within the site boundary. VEPCO et al. also owns all land outside the site boundary that forms the North Anna Reservoir and the Waste Heat Treatment Facility up to their expected high-water marks. The station and all supporting facilities including the North Anna Reservoir, Waste Heat Treatment Facility, earthen dam, dikes, railroad spur, and roads constitute approximately 13,775 acres out of a total land allocation of some 18,643 acres.

The topography in the site region is characteristic of the central Piedmont Plateau with a gently undulating surface varying from 200 to 500 ft above sea level. The surrounding region is covered with forest and brushwood interspersed with an occasional farm. The land adjacent to Lake Anna is becoming increasingly residential as the land is developed.

Lake Anna was constructed to serve the needs of the North Anna Power Station. The lake is approximately 17 miles long, with an irregular shore line of approximately 272 miles. Lake Anna is divided into two major portions, the North Anna Reservoir and the Waste Heat Treatment Facility (WHTF). The lake covers a surface area of 13,000 acres and contains approximately 100 billion gallons of water. The largest segment, the North Anna Reservoir, consists of approximately 9600 acres and functions as a storage impoundment to ensure adequate water for condenser cooling. The smaller segment, the Waste Heat Treatment Facility, has an area of about 3400 acres and is separated from the North Anna Reservoir by dikes. The first of the Waste Heat Treatment Facility's three cooling lagoons receives the heated condenser cooling water after its passage through the units. The heated water transfers most of its heat to the atmosphere as it moves, via canals, to the second and third cooling lagoons. The cooled water is discharged from the third cooling lagoon to the North Anna Reservoir at a point immediately upstream of the dam.

The principal tributaries of Lake Anna include the North Anna River, Pamunkey Creek, and Contrary Creek. Several smaller tributaries drain to the lake as well. Only two of the tributaries draining into Lake Anna are gauged: Pamunkey Creek at Lahore, Virginia (USGS 01670180), and Contrary Creek near Mineral, Virginia (USGS 01670300). The Pamunkey Creek station gauges a drainage area of 40.5 square miles. The daily streamflow record extends from August 1989 through July 1993 (Reference 1.1-1). The Contrary Creek station gauges a drainage area of 5.53 square miles. The daily streamflow record for this station extends from October 1975 through January 1987 (Reference 1.1-1). The remaining 297 square miles of the 343 square mile Lake Anna watershed are not gauged and cannot be characterized accurately for inflows to the impoundment. Inflows can be estimated, however, from records obtained from the North Anna River near Doswell, Virginia, which has a record that measures streamflow from April 1929 through September 1988. This gauging station is located approximately 15 miles downstream of the dam and gauges a drainage area of 441 square miles (Reference 1.1-1).

Using the portion of the Doswell, Virginia record preceding dam closure (i.e., April 1929 through December 1971), inflows to Lake Anna were estimated. The flows at Doswell are larger than the flows at the dam due to the larger contributing drainage area. Thus, these flows were adjusted by multiplying by the ratio of the drainage area at the dam to the drainage area at Doswell, Virginia. Table 1.1-1 summarizes the observed and estimated mean monthly inflows to Lake Anna estimated as described above.

Outflows from Lake Anna have been measured on the North Anna River near Partlow, Virginia, which is located just downstream of the dam at the Virginia Route 601 bridge. The drainage area at this stream gauge is 344 square miles. The daily streamflow record for this gauging station extends from October 1978 through September 1995. The discharge at this station reflects the regulated outflow from Lake Anna for the entire period of record since the dam was completed in 1972 (Reference 1.1-1). Table 1.1-1 summarizes the mean monthly outflows from the Lake Anna impoundment using streamflow data from the U.S. Geological Survey (USGS). Note that the period of record for the estimated total inflow precedes the closure of the North Anna Dam whereas the period of record for the outflow occurs after dam closure. Mean monthly outflows may, therefore, exceed the mean monthly inflows for some months.

Lake Anna water levels have been recorded since the existing units began operating. Mean monthly water levels for the period of record from August 1978 through March 2003 are summarized in Table 1.1-1. The North Anna Power Station Units 1 & 2 adopts the mean sea level (msl) as the plant's reference vertical datum, which is also referred to as the National Geodetic Vertical Datum of 1929 (NGVD 29) in this report. Directions are specified relative to true north in this report, unless otherwise stated. The plot plan for the North Anna Power Station is shown on Figure 1.1-2.

References

1.1-1 North Anna Early Site Permit Application, Site Safety Analysis Report, Revision 9, Dominion, September 2006.

Table 1.1-1. Mean Monthly Hydrologic Statistics for Lake Anna

Month	Pamunkey Creek Inflow[1] (cfs)	Contrary Creek Inflow[2] (cfs)	Estimated Total Inflow[3] (cfs)	Outflow[4] (cfs)	Water Level[5] (ft NGVD 29)
January	61.2	7.97	411	401	249.79
February	37.5	9.37	449	507	249.89
March	49.0	8.92	497	601	249.95
April	62.0	8.36	454	485	249.91
May	43.0	4.33	286	330	249.88
June	23.9	2.46	171	215	249.77
July	19.3	1.34	161	133	249.59
August	9.72	3.40	228	134	249.43
September	14.5	1.20	125	109	249.12
October	31.8	3.16	174	138	248.97
November	31.8	5.05	218	244	249.14
December	47.6	5.46	298	265	249.49

[1] USGS 01670180 Pamunkey Creek at Lahore, Virginia, September 1989 – April 1993 (Reference 1.1-1).

[2] USGS 01670300 Contrary Creek near Mineral, Virginia, October 1975 – December 1986 (Reference 1.1-1).

[3] USGS 01671000 North Anna River near Doswell, Virginia, January 1929 – December 1971 (Reference 1.1-1), scaled to Lake Anna drainage area.

[4] USGS 01670400 North Anna River near Partlow, Virginia, October 1978 – September 1995 (Reference 1.1-1).

[5] August 1978 – March 2003.

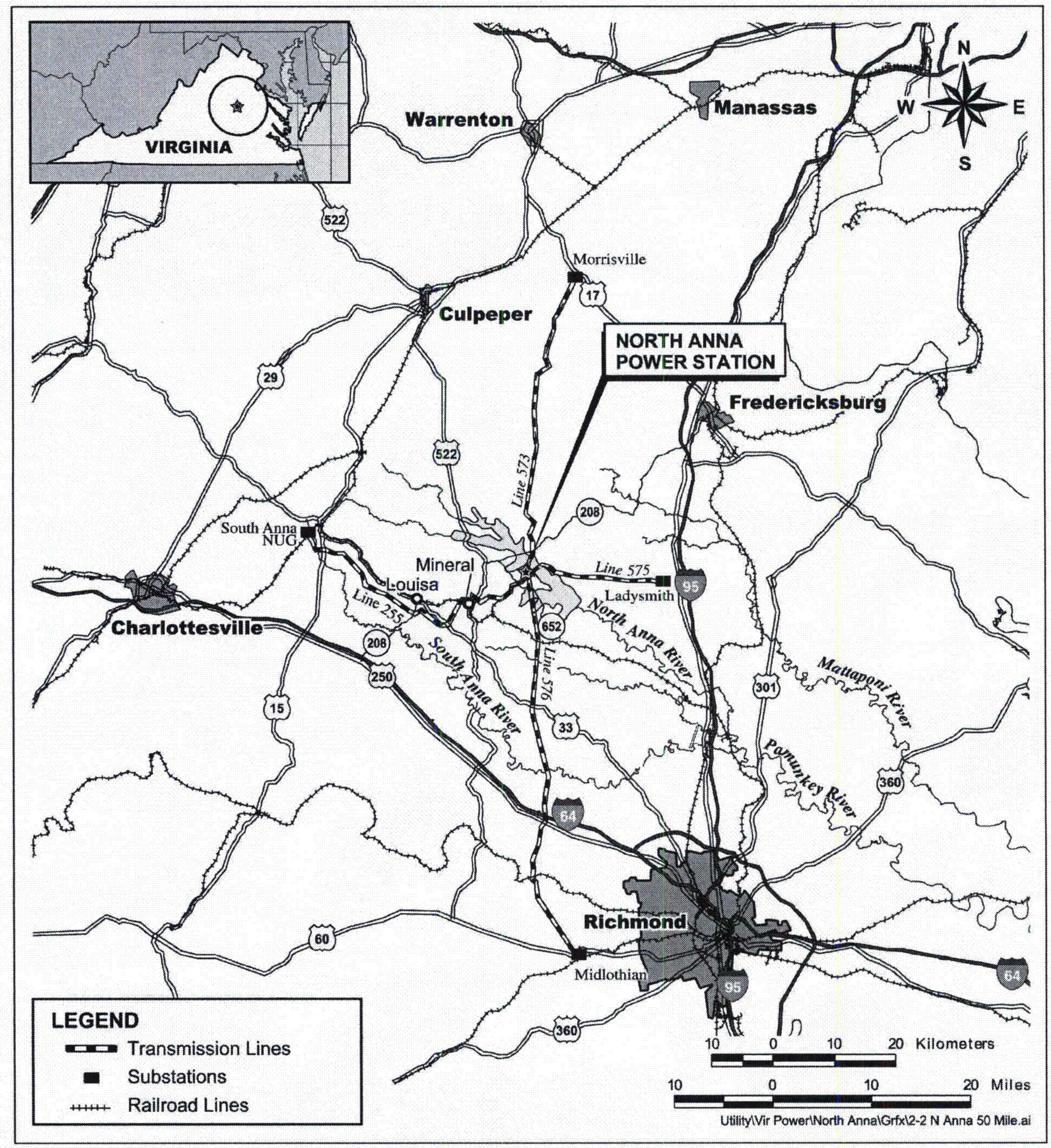

Figure 1.1-1. Vicinity Map

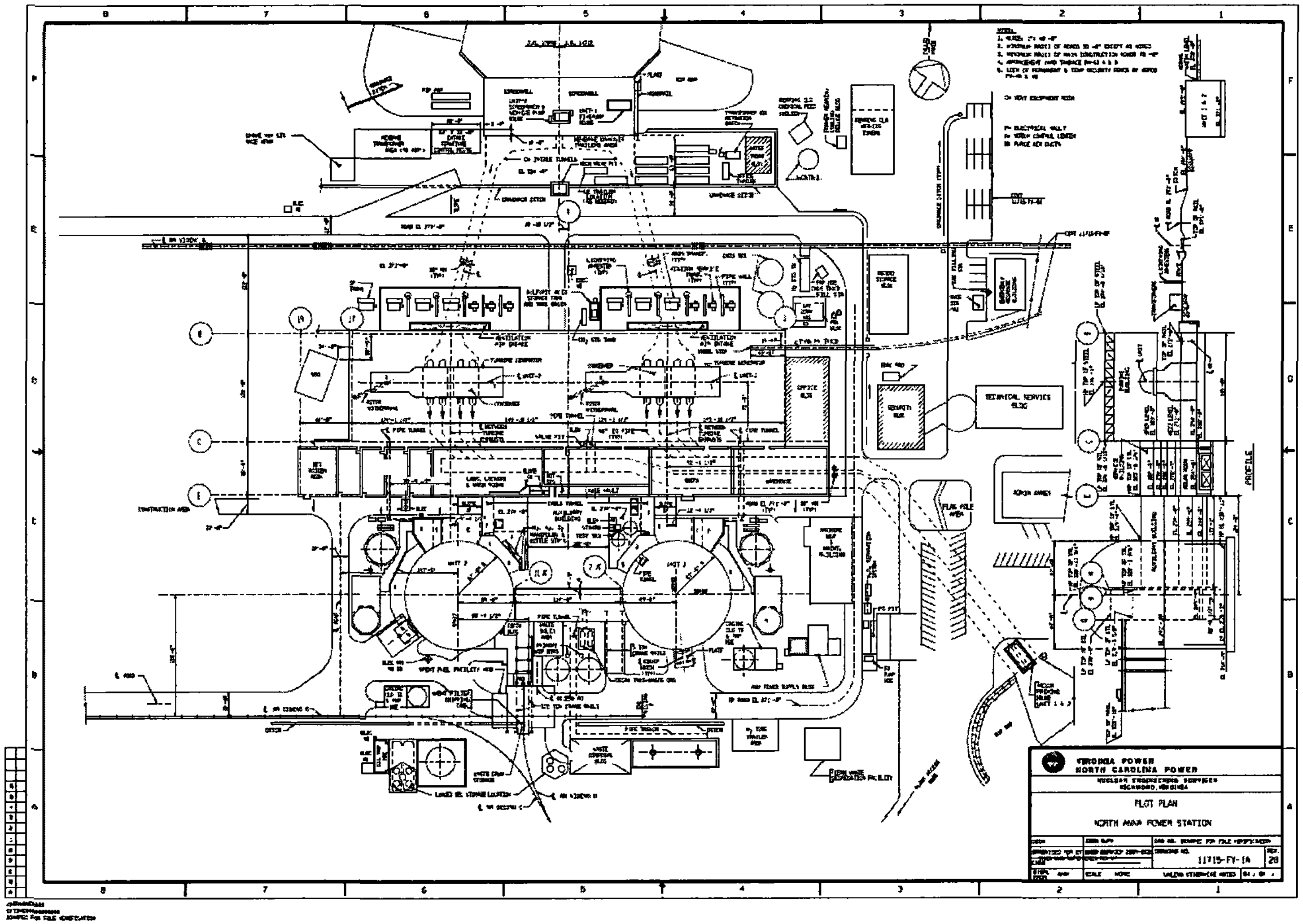

Figure 1.1-2. North Anna Power Station Units 1 & 2 Plot Plan

1.2 Current Design Basis Flood Elevations

Current design and licensing documents, including flood response procedures and flooding walkdown report (Reference 1.2-1), were reviewed to identify site-specific features credited for protection and mitigation against external flooding events. NAPS Units 1 & 2 were evaluated and determined to be in accordance with the General Design Criterion (GDC) for flooding GDC 2, *Design Bases for Protection Against Natural Phenomena*; as well as Regulatory Guide (RG) 1.59, *Design Basis Floods for Nuclear Power Plants*; and RG 1.102, *Flood Protection for Nuclear Power Plants*. 10 CFR 50 Appendix A, GDC 2, states:

"Systems, structures, and components important to safety shall be designed to withstand the effects of natural phenomena, such as earthquakes, tornados, hurricanes, floods, tsunami, and seiches without the loss of capability to perform their safety functions. The design basis for these systems, structures, and components shall reflect:

1. Appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2. Appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena.

3. The importance of the safety functions to be performed."

NAPS was evaluated and determined to be in accordance with RG 1.59 and RG 1.102 to prevent the loss of safety-related functions resulting from the most severe flood conditions that can reasonably be predicted to occur at the site as a result of severe hydrometeorological conditions, seismic activity, or both.

The NAPS Updated Final Safety Analysis Report (UFSAR) (Reference 1.2-2), Chapter 2, in addition to Sections 3.1 and 3.4, provides a description of external flooding events and the resultant flooding levels. Using the NAPS UFSAR as the basis, the probable maximum flood (PMF) is the only flood source event that is discussed in detail. Although not specifically evaluated as a major flooding event in the UFSAR, the local intense precipitation event is being addressed as part of the 50.54(f) letter response.

1.2.1 Probable Maximum Flood with Associated Probable Maximum Precipitation Event

The PMF was established for the NAPS site based on the hydrometeorological conditions for the site area. The effects of probable maximum precipitation (PMP), backwater, wind surge, and wave runup are considered to determine the PMF flood stage for the facility site. The PMF water elevation was conservatively calculated to be a maximum of elevation 268.6 ft NGVD 29 at the circulating water intake structure and elevation 267.3 ft NGVD 29 along the shore leading up to the plant. For the purposes of the flooding walkdowns, the higher flood level at the intake structure, elevation 268.6 ft NGVD 29, was used as the reference PMF level to add conservatism in determining available margin. Systems, structures, and components (SSCs) necessary to maintain the plant in a safe condition are unaffected as plant grade is at elevation 271 ft NGVD 29.

The PMP flood mechanism was evaluated for NAPS and determined to be the primary contributor to the PMF as discussed in UFSAR Section 2.4.3 and Appendix 2A. The PMF is generated from watershed drainage using a calibrated unit hydrograph for the Lake Anna watershed and the 48-hour PMP of 27.04 inches. The PMP was developed based on Hydrometeorological Report (HMR) No. 33 (Reference 1.2-3) using the processes outlined in UFSAR Appendix 2A. Similarly, the calibrated unit hydrograph for the Lake Anna watershed was developed based on historical storm data using the processes outlined in UFSAR Appendix 2A. PMP direct rainfall and runoff coincident with the associated dam discharge yield a conservative still-water PMF upper-bound level of elevation 264.2 ft NGVD 29 at the North Anna Dam.

Several assumptions were used to simplify and add conservatism to PMF still-water level calculations. The standard project storm of 13.54 inches in 48 hours (approximately half of the PMP) was assumed for antecedent precipitation of the PMP. The antecedent precipitation was also assumed to occur 5 days before the PMP storm, with 3 rainless days between storms. Because of the limited flow between the WHTF and North Anna Reservoir, it was conservatively assumed that the PMP rainfall and runoff is routed to the North Anna Reservoir until the stage reaches elevation 260 ft NGVD 29 (top of the dividing dikes). It was also conservatively assumed that only the water storage above elevation 260 ft NGVD 29 in the WHTF is available. This is equivalent to assuming that the WHTF is full to elevation 260 ft NGVD 29 at the start of the PMF. Additionally, a conservative backwater allowance of 0.2 ft was added to the still-water flood level at the North Anna Dam to determine the still-water level at the NAPS site, as calculated in UFSAR Section 2.4.3.5.

The PMF analysis also included transient water level increases caused by wind surge and wave runup at the time of the maximum still-water flood-water level. Calculations in UFSAR Appendix 2A.2.7 used a sustained 40 mph wind blowing in the most critical direction to determine the maximum wave height at the station in accordance with RG 1.59, Revision 1. Along the shoreline leading up to plant grade and at the face of the circulating water intake structure, wind surge and wave runup effects were conservatively calculated to increase the PMF water elevation by a maximum of 2.9 ft and 4.2 ft respectively.

The flood protection elevation for safety-related SSCs is based on the maximum North Anna Reservoir level and wave runup associated with the PMF. Analysis resulted in a conservative upper-bound value for the still-water level of elevation 264.2 ft NGVD 29 at the North Anna Dam. As calculated in UFSAR Appendix 2A, the plant site would experience up to an additional 0.2 ft of backwater effects and 2.9 ft of wind surge and wave runup effects. Therefore, the maximum flood level at the plant site totals elevation 267.3 ft NGVD 29, 3.7 ft below plant grade of elevation 271 ft NGVD 29. Similarly, wave runup effects at the circulating water intake structure were conservatively calculated to be a maximum of 4.2 ft. Using the same still-water level and backwater allowance, the PMF totals elevation 268.6 ft NGVD 29 at the circulating water intake structure. The results of this conservative analysis show that the flood stage associated with the PMF is below plant grade such that the SSCs necessary to maintain the plant in a safe condition are unaffected.

1.2.2 Local Intense Precipitation Event

The local intense precipitation event is only generally considered in the licensing basis for flood protection features at NAPS. As discussed in UFSAR Section 2.4.2.2, the site is relatively flat, and no concentration of runoff is expected on the flat areas. The site is graded to cause surface runoff to flow away from the turbine buildings, reactor containments, and safety-related facilities to the North Anna Reservoir or WHTF via the storm drain systems. The site drainage system was designed to carry the runoff from a 10-year storm of 5-minute duration with rainfall intensity of 7 inches per hour.

According to UFSAR Table 2.4-5, no accumulation is expected on the site at any point during the 10-, 50-, and 100-year storms with the storm drain systems available to remove water from the site. The drainage area to the west of the site, however, receives runoff from approximately 35 acres. The drainage swales, ditches, and culverts in this area have been designed for a 50-year storm. The area west of the site has the potential to be flooded from rainfall in excess of the 50-year design storm. Because this area is west of the main site facilities, potential flooding has no effect on the site or the integrity of the safety-related facilities. When the 100-year storm exceeds the capacity of the drainage swales, ditches, and culverts, water runs off the site and into the North Anna Reservoir and the WHTF to the north and east. According to UFSAR Section 2.4.2.2, flooding associated with this degree of precipitation does not interfere with the capability to safely shut down the station, nor does it affect the safety-related facilities.

The UFSAR does not address specific on-site flood water depths or elevations for local intense precipitation events. Therefore, for the purpose of this report, a precipitation event flood level was established based on a review of applicable UFSAR sections and calculations. The precipitation event flood level was established to be ground level with no accumulation or significant ponding at each site location around power block structures. No other key assumptions are mentioned in the UFSAR associated with the physical state or performance characteristics of the flood protection and mitigation features associated with this flooding event.

1.2.3 Potential Flooding in West Basin Area

As discussed in UFSAR Section 3.8.6, a flood protection dike, known as the "west dike," is located at the west end of the Unit 2 Turbine Building to protect the west side of the Unit 2 Turbine Building from rising lake water resulting from a PMF event. The west dike was built to the Unit 2 site grade and forms a depressed area between the dike and the Unit 2 Turbine Building, known as the "West Basin." Because this area is subject to water accumulation during intense precipitation events, a drain pipe was installed through the base of the west dike to allow water to drain to the lake. The drain pipe is designed to be isolated by valves to prevent rising lake water (during a PMF event) from penetrating the dike. Closing these valves causes rainwater falling within the west dike and yard surrounding the Unit 2 Turbine Building to accumulate in the West Basin. The west wall of the Unit 2 Turbine Building bounds the West Basin and is protected from flooding up to elevation 257 ft NGVD 29.

When the west dike was constructed, a conservative calculation was performed to determine the potential volume of water that could accumulate in the West Basin. The calculation applied a PMP with intensity and duration based on HMR No. 33 localized to the site. Conservative assumptions regarding the contributing "footprint" included the actual square footage formed by the West Basin and the contribution of structure walls which bound the area and capture "driving" rain into the West Basin area.

The Station Blackout (SBO) Diesel Building was subsequently constructed within the West Basin area displacing a volume of flood water storage previously available to accommodate the accumulation of precipitation. The potential water accumulation and storage in the West Basin was revised to account for this reduction of volume and demonstrate the acceptability of the design and ensure continued compliance with GDC 2.

Assuming the west dike drain pipe is closed, the volume available in the West Basin area up to elevation 257 ft NGVD 29 is sufficient to contain precipitation resulting from the 10-year storm. However, with the drain pipe closed, storms in excess of the 10-year storm accumulate sufficient water to raise the water level in the west bowl above elevation 257 ft NGVD 29 and begin to enter and cause flooding of the Turbine Building basement below elevation 257 ft NGVD 29.

The North Anna Turbine Building basement areas are also subject to internal flooding events in the design and licensing basis of the station. Barriers are in place to protect vital equipment from such an internal flooding event up to elevation 257 ft NGVD 29. The designed height of these barriers considered the volume from the floor elevation (elevation 254 ft NGVD 29) to the top of the barrier height to accommodate flood water as well as the large volume available in the sumps and pits below the Turbine Building basement floor. This same volume is available for external flood waters that enter the Unit 2 Turbine Building through the west wall openings. The total volume of accumulated precipitation from the PMP was calculated to be less than the combined volume of the West Basin up to elevation 257 ft NGVD 29 (subtracting the SBO Diesel Building volume) and the volume in the Turbine Building basement up to the internal flood barrier protection elevation. Thus, with the west dike drain pipe closed, the maximum water level in the Turbine Building basement due to external flooding from a precipitation event was calculated to be elevation 256.1 ft NGVD 29, i.e., 0.9 ft below the internal flood barriers.

1.2.4 IPEEE Evaluation

A review and reevaluation of external flooding was also performed in response to NRC Generic Letter (GL) 88-20, *Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities*, and resolution of NRC generic issue GI-103, *Design for Probable Maximum Precipitation (PMP)*. This IPEEE evaluation was beyond design basis and is not included in the CLB for NAPS.

1.2.5 Service Water Reservoir Overflow Event

The Service Water Reservoir was designed and constructed with an emergency overflow dike and intercepting channel as discussed in UFSAR Section 3.8.4.7.5. Regardless of the initiating event, i.e., seismic, wind, or precipitation, the emergency dike controls and diverts potential

Service Water Reservoir overflow or breach flow to the WHTF, thereby preventing water from entering site structures. This event has no calculated flood level.

1.2.6 Probable Maximum Hurricane and Storm Surge

Because the site is not located on an estuary or open coast, flooding from hurricane storm surge does not produce the maximum still-water level at the site. Instead, the maximum still-water level at the site is caused by the PMF; however, the PMP may be from a hurricane. Additionally, surge effects and wave runup are added to the maximum water level at the station site within the PMF.

The probable maximum hurricane (PMH), as discussed in UFSAR Section 2.4.11.2, is a contributing factor in low Lake Anna water level calculations. A PMH wind directing water away from the intake yields an estimated low-water surge of 0.3 ft below the still-water surface.

1.2.7 Groundwater Ingress

The groundwater ingress mechanism was not included as a credible flooding event at the NAPS. As discussed in UFSAR Section 3.4, groundwater protection is a design consideration such that below-grade walls are designed for the maximum anticipated hydrostatic loadings and the containment foundation incorporates a waterproof membrane for groundwater corrosion protection.

1.2.8 Tsunami and Seiche

The tsunami flood mechanism was not included as a design consideration at NAPS as discussed in UFSAR Section 2.4.6. The inland site location of NAPS does not invoke the possibility of flooding due to tsunami-related incidents. Seiche-related flooding is not addressed in the UFSAR and is not a design consideration.

1.2.9 Ice-Related Flooding

Flooding from ice formation is not deemed to be a credible event at NAPS as discussed in UFSAR Section 2.4.7. Studies have indicated that monthly average natural temperatures of Lake Anna would have reached the freezing point only once during the entire period of record. With full generating capacity of the station providing heat, it is estimated that the minimum temperature of Lake Anna would be approximately 40°F.

References

1.2-1 North Anna Power Station Units 1 and 2 Report in Response to March 12, 2012 Information Request Regarding Flooding Aspects of Recommendation 2.3, Virginia Electric and Power Company (Dominion) letter to NRC, Serial No. 12-207G, November 27, 2012.

1.2-2 Updated Final Safety Analysis Report, Rev. 46.06, North Anna Power Station, Virginia Power, Updated Online 05/12/2011.

1.2-3 Hydrometeorological Report No. 33, Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1000 Square Miles and Durations of 6, 12, 24, and 48 Hours, U.S. Weather Bureau, 1956.

1.3 Licensing Basis Flood-Related and Flood Protection Changes

There have been no flood-related changes or changes to flood protection measures beyond the measures in place for the current design basis. Any new flood protection measures planned for NAPS Units 1 & 2 will be addressed by the licensee after review of this Flooding Hazard Reevaluation Report.

1.4 Watershed and Local Area Changes

Lake Anna is formed by an embankment dam across the North Anna River. The North Anna River originates in the eastern slopes of the Southwestern Mountains in the Appalachian mountain range near Gordonsville, Virginia, and follows a southeasterly course to its confluence with the South Anna River 5 miles northeast of Ashland, Virginia, where the Pamunkey River is formed. The Pamunkey continues on a general southeasterly course to West Point, Virginia, where it is joined by the Mattaponi River to form the York River. The York River flows into the Chesapeake Bay about 15 miles north of Hampton, Virginia (Reference 1.4-1). The North Anna River drains a watershed area of 343 square miles above the dam, which is located about 4 miles north of Bumpass, Virginia, and about a half mile upstream of Virginia Route 601.

Lake Anna is about 17 miles long and inundates several small tributaries; thereby, resulting in an irregular shape having a shoreline length of approximately 272 miles. To provide optimum thermal performance for the existing units, Lake Anna is separated into two segments by a series of dikes and canals. The larger segment of about 9600 acres is referred to as the North Anna Reservoir and functions as a storage impoundment to ensure adequate water supplies for condenser cooling. The smaller segment, called the Waste Heat Treatment Facility (WHTF), has an area of about 3400 acres and functions primarily as a heat exchanger for transferring most of the NAPS units heat rejection to the atmosphere. The principal tributaries of Lake Anna include the North Anna River, Pamunkey Creek, and Contrary Creek. Several smaller tributaries drain to the lake as well.

The upstream watershed lies in three Virginia counties; Louisa, Spotsylvania and Orange. The watershed is predominantly rural with residential areas in the immediate surroundings of Lake Anna. Of the acreage in the Lake Anna watershed, 57 percent is forest, 38 percent is covered with cropland and pasture, and 3 percent is developed for residential use. The comprehensive plans for each county indicate past growth and predict future growth in each county. However, the designated growth centers are not located within the Lake Anna watershed (Reference 1.4-2). Development near Lake Anna since operations at NAPS Units 1 & 2 began has been limited to residential development in areas adjacent to the lake. The development primarily consists of large residences on large lots where much of the land remains vegetated with only a small increase in impervious surfaces. Because the percentage of residential area in the watershed remains small the impact of residential growth around Lake Anna on the runoff volume to Lake Anna during flooding events has been negligible.

As discussed in Section 2.2 precipitation and Lake Anna water level data from three storm events that occurred after construction of NAPS Units 1 & 2 have been used to demonstrate that the Lake Anna unit hydrograph developed for the flood analyses used in the NAPS Units 1 & 2 UFSAR is still valid and does not need adjustment due to changes in the watershed. Additionally, if the North Anna Unit 3 is constructed and begins operation, the normal pool elevation for Lake Anna will be raised by 3 inches from elevation 250.00 ft NGVD 29 to elevation 250.25 ft NGVD 29, to meet environmental permit requirements.

With respect to the drainage area affecting the local PMP runoff, vehicle security barriers have been placed around the site as described in Section 2.1. These barriers restrict runoff flow from the eastern and southern borders of the NAPS Units 1 & 2 protected area, channelizing it to openings in the vehicle barrier system. The reevaluation analysis for local intense precipitation described in Section 2.1 accounts for the placement of the vehicle barrier system.

References

1.4-1 Updated Final Safety Analysis Report, Rev. 46.06, North Anna Power Station, Virginia Power, Updated Online 05/12/2011.

1.4-2 "Lake Anna Special Area Plan," Lake Anna Special Plan Committee, March 2000.

1.5 Current Licensing Basis Flood Protection and Mitigation Features

1.5.1 Probable Maximum Flood Event

As discussed in Section 1.2, flood protection from PMF high lake water is inherent because the plant grade is above the maximum expected lake surface elevation, including coincident wind and wave activity.

In addition, a flood protection dike to the west of the Unit 2 Turbine Building, known as the "west dike," provides flood protection from high Lake Anna flood waters. A drainpipe with closure valves was installed within the west dike in order to provide drainage to the low area between the dike and Unit 2 Turbine Building known as the "West Basin." To prevent rising lake water from penetrating the west dike, the dike drainage pipe valve is closed upon a North Anna Reservoir level of elevation 252 ft NGVD 29.

Upon a reservoir level of elevation 256 ft NGVD 29, both units are manually shut down and the circulation water valves are closed. This removes the possibility of flooding the Turbine Building basement up to the North Anna Reservoir surface elevation upon a coincident failure of the circulation water system pressure boundary. Abnormal Procedure 0-AP-40, *Abnormal Level in North Anna Reservoir (Lake)* (Reference 1.5-1), initiates these actions through Technical Requirements Manual (TRM) (Reference 1.5-2), Section 3.7.16, at the conservative North Anna Reservoir level of elevation 254 ft, NGVD 29 as discussed below.

1.5.2 Precipitation Event

Flood protection from the precipitation event is provided by the yard storm and sanitary sewer system and by local site grading as discussed in UFSAR Section 2.4.2. The yard storm and sanitary sewer system was designed to carry the runoff from a 10-year storm of 5-minute duration with a rainfall intensity of 7 inches per hour. The drainage area that contributes runoff to the yard drainage system is not much larger than the site protected area. The site is graded to cause surface runoff to flow away from the turbine buildings, reactor containments, and the safety-related facilities to Lake Anna or the storm drains.

The area west of the site and outside the protected area receives runoff from approximately 35 acres. The drainage swales, ditches, and culverts in this area have been designed for a 50-year storm.

The yard within the protected area, however, is quite flat, and the installation of the vehicle barrier system since the plant was built has limited the ability of water to leave the site by surface runoff. As a result, there is now a higher dependence on the storm drains to remove water during a precipitation event.

As discussed above, the west dike drainpipe valve is closed when North Anna Reservoir water levels reach elevation 252 ft NGVD 29 in accordance with Abnormal Procedure 0-AP-40. This prevents rising lake water from penetrating the west dike. Closing this valve causes rainwater falling on the West Basin and the yard surrounding the Unit 2 Turbine Building to accumulate in the West Basin. The west wall of the Unit 2 Turbine Building provides flood protection from water accumulating in the West Basin to elevation 257 ft NGVD 29. If precipitation water accumulates higher than elevation 257 ft NGVD 29, water may enter the Turbine Building basement (floor elevation 254 ft NGVD 29).

The station is designed to withstand flooding in the Turbine Building up to elevation 257 ft NGVD 29 without impacting safety-related equipment. Internal flood protection is provided by dike-type barriers at various locations throughout the Turbine Building basement. These barriers prevent the intrusion of water into areas containing safety-related equipment and are required by station Technical Specification TS LCO 3.0.9 (Reference 1.5-3). An assessment of the local PMP runoff has determined that water from a precipitation event may fill the West Basin to elevation 257 NGVD 29 and the Turbine Building basement (including sumps and pits) to a maximum of elevation 256.1 ft NGVD 29. Therefore, a freeboard of 0.9 ft exists at internal barriers to safety-related equipment within the Turbine Building.

1.5.3 Service Water Reservoir Overflow Event

The Service Water Reservoir was designed and constructed with an emergency overflow dike and intercepting channel as discussed in UFSAR Section 3.8.4.7.5. Regardless of the initiating event, i.e., seismic, wind, or precipitation event, the emergency dike controls and diverts potential Service Water Reservoir overflow or breach flow to the WHTF, thereby preventing water from entering site structures.

1.5.4 Abnormal Procedures

Corporate Hurricane Response Plan

Dominion corporate hurricane response plan HRP-NUCLEAR, *Hurricane Response Plan (Nuclear)* (Reference 1.5-4), provides a corporate-level assessment of station operational status and delineation of corporate responsibilities and support staff requirements. Although not subject to surge flooding due to its inland location, severe wind and precipitation due to a hurricane can impact the North Anna site. The plan provides for an assessment of pre-storm preparedness and implementation of associated contingency activities. The plan also establishes post-storm guidelines and addresses emergency staffing in terms of management, supervision, and support personnel. The plan is intended to provide general guidelines for management to prepare for and recover from a hurricane. The plan contains activity checklists developed to expedite preparations for impending severe weather, as well as post-storm response actions. A management decision to implement the plan would be made approximately 36 hours prior to the projected arrival of hurricane-force winds onsite.

Emergency Plans

Station and Corporate Emergency Plans are designed to be activated for certain severe weather conditions. A Notice of Unusual Event (NOUE) is declared at NAPS when onsite wind speed is greater than 80 mph. If these winds cause damage or affect systems, an Alert is declared. Upon notification that weather conditions are deteriorating, the emergency response organization would activate emergency response facilities to the extent determined by senior management.

NAPS Site-Specific Procedures

PMF events would occur in the presence of severe weather. NAPS Abnormal Procedure 0-AP-41, *Severe Weather Conditions* (Reference 1.5-5), is initiated either by severe weather indications from monitoring the National Weather Service, Virginia Power Weather Center, or actual site conditions. This procedure provides instructions to review severe weather bulletins; close, replace, or install temporary measures for manholes, blocks, and missile barriers; monitor the intake structure for debris; and evaluate the weather-related risks associated with suspended processes such as maintenance and fuel handling.

Additionally, Abnormal Procedure 0-AP-40 and TRM Sections 3.7.4 and 3.7.16, provide instructions for when the North Anna Reservoir level exceeds elevation 251 ft NGVD 29. To prevent rising lake water from penetrating the west dike, the dike drainage pipe valve is closed upon a North Anna Reservoir level of elevation 252 ft NGVD 29. Upon a North Anna Reservoir level of elevation 256 ft NGVD 29, both reactors are manually shut down and the circulation water valves are closed. This removes the possibility of flooding the Turbine Building basement up to the North Anna Reservoir surface elevation upon a coincident failure of the circulation water system pressure boundary.

Adverse weather conditions concurrent with the PMF event or precipitation event do not affect the operator's ability to perform the required procedural steps stated above.

1.5.5 Warning Systems to Detect the Presence of Water

The North Anna Reservoir water level is used to initiate flooding protection procedures in Abnormal Procedure 0-AP-40 and in the TRM. The North Anna Reservoir water level is measured at the dam and the intake structure. The measurement at the dam is read locally and communicated verbally to the control room by telephone. The measurement at the intake structure is read and can be trended in the control room from the plant computer.

There are no room water level warning systems that are credited as a flood protection feature for an external flooding event.

Sump alarms activate when sump water levels reach setpoints throughout safety-related structures. However, these alarms and their associated pumps are not credited for flood protection or mitigation in response to CLB external flooding hazard events. Several water level detection systems are located throughout the Turbine Building and Auxiliary Building for detection of internal flooding. These water level warning systems are available but are not credited for flood protection in the external flooding licensing basis.

References

1.5-1 Abnormal Procedure 0-AP-40, Abnormal Level in North Anna Reservoir (Lake), Revision 18, North Anna Power Station, Dominion.

1.5-2 Technical Requirements Manual (TRM), Revision 78, North Anna Power Station, Dominion.

1.5-3 Technical Specifications, Units 1 and 2, North Anna Power Station, Dominion.

1.5-4 Procedure HRP-NUCLEAR, Hurricane Response Plan (Nuclear), Revision 11, Dominion.

1.5-5 Abnormal Procedure 0-AP-41, Severe Weather Conditions, Revision 54, North Anna Power Station, Dominion.

2.0 Flooding Hazard Reevaluation

Flooding hazards from various flood causing mechanisms were evaluated for the NAPS Units 1 & 2 in accordance with Enclosure 2 of the NRC's March 12, 2012 50.54(f) Request for Information Letter (Reference 2.0-1) that identifies the requirements for the flooding hazard reevaluations associated with Near Term Task Force (NTTF) Recommendation 2.1. The flooding hazard reevaluation for NAPS Units 1 & 2 follow the hierarchical hazard assessment (HHA) process described in NUREG/CR-7046 (Reference 2.0-2). As explained in Attachment 1 to Enclosure 2 of the NRC's 50.54(f) letter, HHA is a progressively refined, stepwise estimation of the site-specific hazards that evaluates the safety of the site with the most conservative plausible assumptions consistent with available data. Consistent with the HHA approach, flooding mechanisms that are determined to not be the controlling factors of the design basis flood will be screened out using order of magnitude analysis or qualitative assessments, where appropriate, with conservative assumptions and physical reasoning based on the physical, hydrological and geological settings of the site. For the flooding mechanism(s) that will potentially control or affect the design basis, detailed analyses will be performed based on present-day methodologies and standards. The NAPS Units 1 & 2 flooding reevaluation applies the flooding hazard analysis approaches, regulatory guidance, and methodologies used in support of the preparation of the Combined License Application (COLA) for a future unit at the site (NAPS Unit 3) (Reference 2.0-3), which is augmented by recent site specific information. The principal regulations and guidance related to flooding hazard evaluations and the determination of design basis floods include: 10 CFR 50 Appendix A (Reference 2.0-4); 10 CFR 52.79 (Reference 2.0-5); 10 CFR 100.20 (Reference 2.0-6); Regulatory Guides 1.59 (Reference 2.0-7), 1.102 (Reference 2.0-8), 1.206 (Reference 2.0-9); Standard Review Plan (NUREG 0800) Sections 2.4.1 to 2.4.7, 2.4.9 to 2.4.10 (Reference 2.0-10); NUREG/CR-7046 (Reference 2.0-2); NUREG/CR-6966 (Reference 2.0-11); and ANSI/ANS-2.8-1992 (Reference 2.0-12).

This chapter describes in details the reevaluation effort for each plausible flooding mechanism and the potential impacts to the safety-related and important to safety SSCs of the plant: flooding impacts due to local intense precipitation (Section 2.1), flooding in streams and rivers (Section 2.2), dam breaches and failures (Section 2.3), storm surge (Section 2.4), seiche (Section 2.5), tsunami (Section 2.6), ice induced flooding (Section 2.7), channel migration or diversion (Section 2.8), and combined effect flood (Section 2.9).

The North Anna Power Station Units 1 & 2 adopts the Mean Sea Level (msl) as the plant's reference vertical datum, which is also referred to as the National Geodetic Vertical Datum of 1929 (NGVD 29) in this report. Directions are specified relative to true north in this report, unless otherwise stated.

References

2.0-1 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, ADAMS Accession No. ML12053A340, U.S. Nuclear Regulatory Commission, March 12, 2012.

2.0-2 Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7046, U.S. Nuclear Regulatory Commission, November 2011.

2.0-3 North Anna 3 Combined License Application, Final Safety Analysis Report, Revision 5, Dominion, March 2012.

2.0-4 Title 10 of Code of Federal Regulations, Appendix A to Part 50—General Design Criteria for Nuclear Power Plants, 10 CFR 50, Appendix A, U.S. Nuclear Regulatory Commission, December 2012.

2.0-5 Title 10 of Code of Federal Regulations, Part 52.79 Contents of applications; technical information in final safety analysis report, 10 CFR 52.79, U.S. Nuclear Regulatory Commission, December 2012.

2.0-6 Title 10 of Code of Federal Regulations, Part 100.20 Evaluation Factors for Stationary Power Reactor Site Applications on or After January 10, 1997, 10 CFR 100.20, U.S. Nuclear Regulatory Commission, December 2012.

2.0-7 Design Basis Floods for Nuclear Power Plants, Regulatory Guide 1.59, Revision 2, U.S. Nuclear Regulatory Commission, August 1977.

2.0-8 Flood Protection for Nuclear Power Plants, Regulatory Guide 1.102, Revision 1, U.S. Nuclear Regulatory Commission, September 1976.

2.0-9 Combined License Applications for Nuclear Power Plants, Regulatory Guide 1.206, U.S. Nuclear Regulatory Commission, June 2007.

2.0-10 Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition — Site Characteristics and Site Parameters (NUREG-0800, Chapter 2), Sections 2.4.1 through 2.4.10, U.S. Nuclear Regulatory Commission, March 2007.

2.0-11 Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America - Final Report, Pacific Northwest National Laboratory, NUREG/CR-6966, U.S. Nuclear Regulatory Commission, Prasad, R., March 2009.

2.0-12 Determining Design Basis Flooding at Power Reactor Sites, ANSI/ANS-2.8-1992, Nuclear Standard 2.8, American National Standards Institute/American Nuclear Society, July 1992.

2.1 Local Intense Precipitation

A reevaluation analysis of flooding due to local intense precipitation on the North Anna Power Station (NAPS) Units 1 & 2 was performed for this report. Guidelines detailed in U.S. Nuclear Regulatory Commission (NRC) NUREG/CR-7046 (Reference 2.1-1), NRC Regulatory Guide 1.59 (Reference 2.1-2), and ANSI/ANS-2.8-1992 (Reference 2.1-3) are the basis for the methodology used in this reevaluation. In accordance with the Hierarchical Hazard Assessment (HHA) approach outlined in Reference 2.1-1, the analysis is performed assuming underground storm drains and culverts, as well as roof drains are clogged and not functioning during the local probable maximum precipitation (PMP) storm event. Less conservative assessments (e.g., storm drains partially clogged) were not performed for this report.

Natural Resources Conservation Service (NRCS) methods simulated in the US Army Corps of Engineers (USACE) computer program HEC-HMS (Reference 2.1-4) are used to determine runoff hydrographs and peak discharges along identified flow paths. Water surface elevations are determined using the USACE computer program HEC-RAS (Reference 2.1-5).

Elevations shown on the topography on Figures 2.1-1 through 2.1-4 are based on a 2006 LIDAR survey and are referenced to the North American Vertical Datum of 1988 (NAVD 88). The LIDAR data are supplemented by additional spot elevations, also referenced to NAVD 88, from a ground survey to determine cross section geometry along some of the flow paths. Elevations presented in the NAPS Units 1 & 2 UFSAR (Reference 2.1-8) are referenced to mean sea level (msl), which is equivalent to the National Geodetic Vertical Datum of 1929 (NGVD 29). For direct comparison to the current design basis as documented in the UFSAR, elevations presented in this section, with the exception of the topography shown on Figures 2.1-1 through 2.1-4, are presented in the NGVD 29 datum. The datum conversion from NAVD 88 datum elevations to NGVD 29 datum elevations is +0.86 ft (Reference 2.1-9), such that elevation 270.1 ft NAVD 88 is equivalent to elevation 271.0 ft NGVD 29 after rounding.

2.1.1 Site Description

The site topography and drainage sub-basins are shown on Figures 2.1-1 through 2.1-3. Identified flow paths and cross sections are shown on Figure 2.1-4. The protected area is the area inside the security fence and is the area primarily shown on Figures 2.1-1 and 2.1-4. The protected area is essentially completely asphalt or concrete paved, and the Unit 3 excavation and independent spent fuel storage installation (ISFSI) areas are essentially completely grass covered with the exception of the paved ISFSI pads.

As there are no drainage ditches in the NAPS Units 1 & 2 protected area, four flow paths (labeled CW, CE, SW, and SE) along the western, southern, and eastern sides of the protected area are identified based on locations where flows from the protected area can exit the site through the vehicle barrier system or over the drainage divide into the future Unit 3 excavation located west of Unit 2. The top of the vehicle barriers, which are constructed of Jersey Barriers, on the south and east sides of the protected area are located 32 inches above grade, based on standard Jersey Barrier dimensions. The vehicle barriers on the west side of the protected area are located at the bottom of the slope to the future Unit 3 excavation. Vehicle barriers on the north side of the protected area act as a retaining wall with the top of the barrier wall located at protected area grade elevation and the bottom at the NAPS Units 1 & 2 intake area grade elevation.

Flow paths CW and SW receive runoff from the western, southwestern and southern portions of the site and both discharge into the Unit 3 excavation west of the protected area as shown on Figure 2.1-4. Runoff collected in the Unit 3 excavation discharges to the portion of Lake Anna behind the existing cofferdam in the future Unit 3 intake area through a 14 ft by 14 ft concrete box tunnel as shown on Figure 2.1-2. During the local PMP storm event, it is conservatively assumed that the Lake Anna water level, and consequently, the water level in the Unit 3 excavated area, will be equivalent to the top of North Anna Dam at elevation 265.0 ft NGVD 29, which is above the Lake Anna probable maximum flood level (PMF) as discussed in Section 2.2. At this lake water level, the cofferdam, in front of the future Unit 3 intake area with a top at elevation 255.9 ft NGVD 29, will be overtopped.

As shown on Figures 2.1-1 and 2.1-4, Flow Path CE receives runoff from the eastern portion of the protected area and discharges through the opening in the vehicle barrier system along the eastern edge of the protected area. Flow Path SE receives runoff from the southeast portion of the protected area and discharges through a portion of the vehicle barrier system that consists of bollards (allowing water to flow through) along the southeastern edge of the protected area at the new sally port entrance as shown on Figure 2.1-1. At the new sally port entrance, Flow Path SE also receives runoff from a small sub-basin (SE4) south of the protected area and discharges to the east over the access road and into the Waste Heat Treatment Facility (WHTF) discharge canal.

Runoff from the northern portions of the site (sub-basins N1 through N3) sheet flows towards the vehicle barrier along the northern edge of the protected area and south of the NAPS Units 1 & 2 intake area. The vehicle barrier acts as a weir and the sheet flows fall over the vehicle barrier wall and eventually to Lake Anna.

Runoff in NAPS Units 1 & 2 ISFSI area sheet flows to the southwest to a collection ditch that runs along the southern and western edges of the ISFSI area (see Figure 2.1-3). This ditch normally discharges through a culvert to natural drainage to the west. However, during the local PMP runoff analysis, this culvert is assumed to be clogged and does not discharge. The southern and western banks of the ditch are constructed of a berm which has a top at elevation 310.9 ft NGVD 29. During the PMP storm event, flows will overtop the berm, which will act as a weir, and flow down to existing drainage to the south and west.

2.1.2 Runoff Analyses

The local PMP depths for the future Unit 3 site, adjacent to NAPS Units 1 & 2, have been determined for durations up to 6-hours and published in the North Anna Unit 3 Site Safety Analysis Report in the North Anna Early Site Permit Application (Reference 2.1-6, Section 2.4.2). These PMP depths were originally obtained from National Oceanic and Atmospheric Administration (NOAA) publications Hydrometeorological Report Nos. 51 and 52 (References 2.1-10 and 2.1-11). The PMP depths for durations ranging from 5 minutes to 6 hours obtained from these publications are summarized in Table 2.1-1. A plot of the PMP depths versus duration from Table 2.1-1 is shown in Figure 2.1-5. An examination of official rainfall records for the central Virginia area since the publication of the HMRs has not identified any events with precipitation depths of magnitudes approaching the PMP depths listed in Table 2.1-1. Thus, the use of the PMP values from the HMRs is considered applicable for the NAPS Units 1 & 2 site.

Modeling the PMP storm event in HEC-HMS is accomplished using the Frequency Storm option in the HEC-HMS Meteorological Model component. The 6-hour frequency storm option requires precipitation depths for durations of 5 and 15 minutes and 1, 2, 3, and 6 hours. The 5 and 15 minute PMP depths as well as the 1 and 6 hour PMP depths are determined from the HMR values presented in Table 2.1-1. The 2 and 3 hour PMP depths are determined from the plot in Figure 2.1-5. The values input into the HEC-HMS model are summarized in Table 2.1-2. The rainfall depths are temporally distributed with the highest intensity occurring at the storm center and the next highest intensities being distributed on either side of the peak intensity.

The sub-basins delineated for the runoff analysis are shown on Figures 2.1-1 through 2.1-3. The measured drainage area for each individual sub-basin is listed in Table 2.1-3. A schematic of the HEC-HMS sub-basins is shown in Figure 2.1-6. Runoff flows from sub-basins are combined at junctions where flows are entered into the flow path cross sections in the HEC-RAS analysis as shown in Figure 2.1-4. In accordance with NRCS methodologies (Reference 2.1-12), a runoff curve number is estimated for each sub-basin. Typically, the runoff curve number is estimated based on a weighted average of the soil types and land cover conditions for the entire sub-basin. However, for the purposes of this reevaluation, which considers a saturated soil condition as a result of a recent heavy rainfall (40% PMP event) per the combined events requirements in ANSI/ANS-2.8-1992 (Reference 2.1-3), a runoff curve number representing impervious areas is used for each sub-basin. The curve number for impervious surfaces is 98 (Reference 2.1-12) for all soil types. Time of concentration values are estimated for each sub-basin using NRCS methodologies (Reference 2.1-12). To account for non-linear response for large storms such as the PMP, the estimated time of concentration values are reduced by 25% per guidance from the U.S. Army Corps of Engineers (Reference 2.1-13). However, the minimum time of concentration value used in the analysis is 5 minutes, after the 25% reduction. The reduced time of concentration values for each sub-basin are listed in Table 2.1-3. Lag times equal to 0.6 times the time of concentration value are input into the HEC-HMS model (Reference 2.1-4).

Flow paths CW and SW both discharge to the large existing excavated area where the future Unit 3 area will be placed just west of the Units 1 & 2 site. This area is labeled "Reservoir U3" in

Figure 2.1-6. Additionally, Sub-basin U3 discharges to Reservoir U3. Discharge from Reservoir U3 flows through a single 14 ft by 14 ft concrete tunnel underneath the Units 1 & 2 access road to the area behind the cofferdam in front of the future Unit 3 intake area as shown on Figure 2.1-2.

The Lake Anna water level and, consequently, the level for Reservoir U3 during the PMP storm event are conservatively assumed to be at elevation 265.0 ft NGVD 29. This elevation is equivalent to the top of the North Anna Dam and is higher than the Lake Anna PMF still water level of 264.3 ft NGVD 29 as described in Section 2.2.

A stage-surface area and stage-discharge rating for Reservoir U3 are developed and input into the HEC-HMS computer program. HEC-HMS uses the supplied stage-surface area rating data to develop stage-storage rating data within the program. The stage-discharge relationship is developed assuming that only 50% cross sectional area of the 14 ft by 14 ft concrete tunnel is available (the top half of the tunnel) during the PMP storm to account for possible debris accumulation in the tunnel. The top of the concrete tunnel is located at elevation 264.9 ft NGVD 29 and will be submerged during the PMP storm event. Reservoir U3 stage-surface area and stage-discharge relationships are shown in Tables 2.1-4 and 2.1-5. Note that rating data in the HEC-HMS model uses elevations based on the NAVD 88 datum. For consistency with elevations presented in this report, the elevation data presented in the rating tables have been converted to the NGVD 29 datum.

Sub-basin B1, which represents the Turbine Building roof drainage for both units as well as the yard area west of the Turbine Building, discharges to an excavated area around the Station Blackout Building (SBO) (Figure 2.1-1). This area is about 20 ft lower in elevation than the rest of the yard area for NAPS Units 1 & 2 and is referred to as the West Basin. The Turbine Building roof has 2.0 ft high parapet walls on the north, east, and south sides. The wall along the west side of the roof is only 6 inches high. Therefore, during a PMP storm event, when roof drains are assumed to be inoperable, all of the runoff from the Turbine Building roof and the area surrounding the West Basin will discharge to the West Basin. During the PMP event, there will be no discharge from the West Basin. The drain pipe which discharges to the Unit 3 excavation area is conservatively assumed to be blocked during the PMP storm event. During the PMP storm event, runoff collected in the West Basin will be stored in the West Basin and Turbine Building basement until it can be released through the existing drain or pumped out after the PMP storm event.

The drainage divide between the yard area near Flow Path CW and the West Basin ranges between elevation 271.4 ft NGVD 29 and 271.9 ft NGVD 29 as shown on Figure 2.1-4. If water levels along Flow Path CW at Cross Section 46 (See Figure 2.1-4) are higher than elevation 271.4 ft NGVD 29 then some runoff from Sub-basins CW1 and CW2 will also enter the West Basin. To determine the amount of runoff entering the West Basin from Flow Path CW a diversion component labeled "SBO Bowl Diversion" is added to the HEC-HMS model downstream of Junction CW2. The diversion component uses a discharge diversion rating table to determine how much flow is diverted into the West Basin. A diverted flow rate to the West Basin is estimated using the broad crest weir equation for selected flow rates at Cross

Section 46 based on water levels determined in HEC-RAS. The West Basin/SBO Bowl Diversion rating data is shown in Table 2.1-6.

With the sub-basin and rating data described above, runoff hydrographs are determined for each sub-basin and junction using the HEC-HMS model. Additionally, peak water levels in Reservoir U3 and the volume of diverted flow into the West Basin at the SBO Bowl Diversion are determined. A summary from the HEC-HMS model is shown in Table 2.1-7. Flow hydrographs at selected cross section locations along each flow path are shown on Figures 2.1-7 through 2.1-10. Section 2.1.1.3 describes the distribution of the HEC-HMS discharges at each of the HEC-RAS model cross sections.

The HEC-HMS model results for Reservoir U3 indicates that the peak water level is at elevation 268.6 ft NGVD 29. This water level is below the elevation of the divide between the Unit 2 yard area and the future Unit 3 excavation, which ranges from elevation 271.0 ft NGVD 29 to 271.9 ft NGVD 29 and does not affect the water levels in Flow Paths CW and SW.

The HEC-HMS model results for the SBO Bowl Diversion indicates that the total volume of runoff diverted into the West Basin from Flow Path CW during the PMP event is about 1300 ft^3, with the peak diverted flow being 3.3 cfs lasting for approximately 23 minutes.

The total runoff volume into the West Basin from Sub-basin B1 is not determined from the HEC-HMS model. The PMP precipitation storm evaluated for the other sub-basins is a 6-hour PMP storm, where peak discharges control the maximum water levels. Because the West Basin has no outlet during the PMP and acts as a storage area, the runoff volume from a longer duration storm is more appropriate and conservative for a water level assessment. Thus, the 72-hour PMP storm depth is used to determine the runoff volume into the West Basin. While the overall precipitation depth and runoff volume for a 72-hour storm are greater than those of a 6-hour duration storm, peak discharges for small local drainage areas are determined by short duration rainfall depths (i.e., 5 minute duration depths). These depths are the same for both the 6-hour duration and the 72-hour duration storms. Thus, the peak discharges for these two storm durations would be the same.

From the NOAA publication HMR 51 (Reference 2.1-10, Figure 22) a 72-hour, 10-mi^2 PMP depth of 43 inches is determined for the site. In addition to the 72-hour PMP depth, the flooding analysis also considers a 40% PMP storm preceding the PMP event with 3 to 5 dry days in between the storms (Reference 2.1-3). As Sub-basin B1 is considered impervious, all of the precipitation is treated as runoff. It is possible that the runoff from the 40% PMP event could be released or pumped out of the West Basin and Turbine Building basement during the 3 to 5 dry days in between the two storm events. Therefore, the West Basin runoff volume is calculated for two cases: Case 1 includes the 40% PMP antecedent storm event runoff, and Case 2 does not include the 40% PMP antecedent storm event runoff. In addition to the sub-basin B1 runoff, the 1,300 ft^3 from the SBO Diversion is added to the West Basin for each case. The West Basin runoff volumes for each case are listed below.

Case 1 Runoff Volume = 545,450 ft^3

Case 2 Runoff Volume = 389,980 ft^3

Once flood levels in the West Basin reach elevation 257.0 ft NGVD 29, flood waters enter the Turbine Building and begin to flood the Turbine Building basement and sumps. There is a sizeable volume of storage in the Turbine Building below elevation 257.0 ft NGVD 29, so water levels in the West Basin remain just above this elevation until the water level inside the Turbine Building reaches elevation 257.0 NGVD 29. Additionally, there is a flood protection wall located inside the Turbine Building that protects equipment necessary for plant operation. The top of this wall is also located at elevation 257.0 ft NGVD 29. The total storage volume available in the West Basin area and in the Turbine Building basement below the crest of the flood protection wall is 274,131 ft^3. Thus, in both cases, the maximum flood levels during a PMP storm event in the Turbine Building are above the top of the flood protection wall.

2.1.3 Water Level Determination

The four flow path locations in the NAPS Unit 1 & 2 protected area are shown on Figure 2.1-4. Geometric data used in the HEC-RAS analysis to determine cross section geometry and compute water surface elevations is obtained from the topographic information shown on Figure 2.1-4 and from ground survey spot elevations along flow paths CE, CW, and SW. Geometric data is obtained at the cross section locations shown on Figure 2.1-4 along each flow path. Because there are no ditches or swales, the cross sections reflect the geometry of the yard grade elevations and building placement throughout the yard. There are several small rectangular buildings shown on Figure 2.1-4 south and west of Unit 2. These are temporary trailers that were in place when the topographic information was obtained in 2006. These trailers are no longer in place and are not included in the PMP analysis.

As the flow paths are over asphalt paved areas, a Manning's roughness coefficient of 0.016 would normally be selected (Reference 2.1-14). As an added measure of conservatism a Manning's roughness coefficient of 0.018 is selected for the channel and overbank areas to account for debris and wear in the asphalt covering over the site.

Sub-basin discharges are conservatively assigned to the furthest upstream cross section in each sub-basin. Table 2.1-8 describes the HEC-HMS elements that provide the HEC-RAS discharges used at the indicated flow path cross sections.

The peak discharges for all sub-basins contributing to the flow paths in the protected area occur at time 3:04 (from the beginning of the storm). Thus, the peak discharges at time 3:04 are used to determine the peak water levels along the flow paths using the steady state routing option in HEC-RAS. In addition to determining the maximum water level profile along each flow path for the peak discharges, water surface profiles are also developed for four other times. The four additional hydrograph times selected are at times 2:35, 2:50, 2:55, and 3:00. The discharges at these selected times represent the rising limb of the hydrographs as shown on Figures 2.1-7 through 2.1-10. Similar discharges are also seen on the falling limb of the hydrographs. Discharges before the first time (2:35) are relatively small in comparison to the peak discharges.

For instance, the peak discharge at Junction SW4 at time 2:30 is 8.7 cfs compared to 23.2 cfs at time 2:35. Thus, the flood producing discharges occur from time 2:35 to about time 3:35 (See Figures 2.1-7 through 2.1-10). The discharges for each of these selected times at the HEC-HMS elements listed in Table 2.1-8 are determined from the HEC-HMS element hydrographs (Figures 2.1-7 through 2.1-10). A listing of the discharges input into HEC-RAS for all five profiles at the cross sections indicated in Table 2.1-8 are listed in Table 2.1-9.

The HEC-RAS computed water surface elevations and the input discharges for each profile and at each flow path cross section are listed in Table 2.1-10. The flood elevations in Table 2.1-10 correspond to the range of PMP hydrograph discharges from about time 2:35 to about time 3:35. As can be seen in Figures 2.1-7 through 2.1-10, flood hydrograph discharges before and after these times are small, and thus, flood levels outside these times are below the flood levels listed in Table 2.1-10 at each cross section.

Runoff from Sub-basins N1, N2, and N3 sheet flows to the north away from the plant buildings towards the vehicle barrier wall, which acts as a retaining wall between the protected area and the lower NAPS Units 1 & 2 intake area. The runoff then falls over the vehicle barrier wall as weir flow into the NAPS Units 1 & 2 intake area. The depth of sheet flow in each of these sub-basins is estimated based on normal depth using Manning's equation. The width of the channel used in Manning's equation is equal to the length of vehicle barrier wall on the northern edge of the sub-basin minus an arbitrary 10% to account for obstructions that may exist in the yard area between the Turbine Building and the vehicle barrier. As in the HEC-RAS analysis, a Manning's roughness coefficient of 0.018 is used to represent the asphalt surface. The average slope of the yard towards the vehicle Barrier wall is measured to be about 0.003. The results of the Manning's normal depth calculation are summarized in Table 2.1-11. Because these depths are shallow and the runoff flows away from the Turbine Building to the intake area, safety-related facilities are not adversely impacted.

The ISFSI area is shown in Figure 2.1-3. As mentioned previously, runoff from the ISFSI area is collected in a ditch along the southern and western edges of the ISFSI area. The outside bank of the ditch is formed by an embankment berm. During the PMP storm event, when the culvert is assumed to be clogged, the runoff collected in the ditch will overflow the outside ditch embankment and spill as weir flow to natural drainage to the south and west. The length of the weir overflow area from the ditch and ISFSI area is approximately 1040 ft as shown on Figure 2.1-3. A broad crested weir equation is used to calculate the water level required to discharge the PMP runoff over the ditch outer embankment. The crest of the outer berm is at elevation 310.9 ft NGVD 29. The water level required to discharge the PMP flow will essentially be the water level in the ISFSI yard area during the PMP storm event as the grade is nearly flat. The ISFSI area PMP water levels are summarized in Table 2.1-12. Note that the peak discharge in the ISFSI area occurs at time 3:10.

2.1.4 Conclusion

Maximum flood water levels as a result of the local PMP event have been reevaluated for the NAPS Units 1 & 2 site in accordance with NUREG/CR-7046 (Reference 2.1-1). This reevaluation uses the PMP depths derived from HMRs 51 and 52 (References 2.1-10 and 2.1-11) and assumes that all passive storm drainage and roof drainage systems are inoperable. The analysis also uses runoff conditions in accordance with an antecedent storm with an equivalent precipitation depth of 40% of the PMP occurring 3 to 5 days prior to the PMP storm event (Reference 2.1-3). The results of this reevaluation indicate the following:

- The reevaluated flood elevations in the NAPS Units 1 & 2 protected area from local intense precipitation ranges from 271.3 ft to 274.5 ft NGVD 29.

- Using the discharge and water level information in Table 2.1-10 along with the hydrograph plots in Figures 2.1-7 through 2.1-10, a duration for flood levels above a desired elevation at any cross section location can be estimated. It is estimated that the flood levels indicated in Table 2.1-10 along the four designated flow paths last about 65 minutes, with the maximum water level less than 4 minutes at each cross section.

- Runoff generated from the PMP storm event north of the Turbine Building flows to the north away from the Turbine Building and out of the protected area as sheet flow. The maximum water depths during the PMP range from 1.8 to 2.8 inches with durations of less than 8 minutes. Flood depths more than 1.0 inch last less than 30 minutes.

- As the protected area is flat and is essentially covered with an impervious surface, there is no potential for hydraulic jumps or scour. The potential for sediment build up is also small and is accounted for in the assumption that all passive drainage systems are non-operational.

- The runoff volumes for the existing condition into the West Basin for the 72-hour PMP event (389,980 ft^3) and for the 40% 72-hour PMP plus the full 72-hour PMP events (545,450 ft^3) exceed the storage capacity (274,131 ft^3) of the West Basin and Turbine Building basement below the crest of the flood protection wall in the Turbine Building at elevation 257.0 ft NGVD 29.

- The maximum PMP water level in the ISFSI yard area is at elevation 311.2 ft NGVD 29.

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2.1-12 Technical Release 55, Urban Hydrology for Small Watersheds, Soil Conservation Service (Now known as Natural Resources Conservation Service), U.S. Department of Agriculture, June 1986.

2.1-13 EM 1110-2-1417, Flood Runoff Analysis, U.S. Army Corps of Engineers, August 1994.

2.1-14 Chow, V.T., Open Channel Hydraulics, McGraw-Hill Book Co., 1959.

Table 2.1-1. Probable Maximum Precipitation Depths (References 2.1-10 and 2.1-11)

Duration	PMP Depth (in)
5 min	6.1
15 min	9.6
30 min	13.7
1 hr	18.3
6 hr	27.9

Table 2.1-2. HEC-HMS Probable Maximum Precipitation Depths

Duration	PMP Depth (in)
5 min	6.1
15 min	9.6
1 hr	18.3
2 hr	20.2
3 hr	22.1
6 hr	27.9

Table 2.1-3. Sub-basin Drainage Areas

Sub-basin	Drainage Area (ft^2)	Drainage Area (acres)	Drainage Area (Mi^2)	Time of Concentration (min)
N1	11,596	0.27	0.00042	5.0
N2	39,501	0.91	0.00142	5.0
N3	66,028	1.52	0.00237	5.0
B1	108,469	2.49	0.00389	5.0
CW1	42,049	0.97	0.00151	5.0
CW2	20,627	0.47	0.00074	5.0
CE1	44,472	1.02	0.00160	5.0
CE2	18,386	0.42	0.00066	5.0
CE3	64,673	1.48	0.00232	5.0
SW1	22,264	0.51	0.00080	5.0
SW2	28,814	0.66	0.00103	5.0
SW3	27,666	0.64	0.00099	5.0
SW4	23,104	0.53	0.00083	5.0
SW5	23,288	0.53	0.00084	5.0
SW6	14,309	0.33	0.00051	5.0
SW7	19,757	0.45	0.00071	5.0
SE1	40,222	0.92	0.00144	5.0
SE2	24,308	0.56	0.00087	5.0
SE3	41,826	0.96	0.00150	5.0
SE4	30,123	0.69	0.00108	5.0
U3	1,474,070	33.84	0.05287	8.9
ISFSI	450,231	10.34	0.01615	14.6

Table 2.1-4. Reservoir U3 Stage-Surface Area Rating

Elevation (ft NGVD 29)	Area (ft^2)	Area (acre)
250.9	77175	1.77
255.9	186740	4.29
260.9	215088	4.94
265.9	237509	5.45
270.9	303801	6.97

Table 2.1-5. Reservoir U3 Stage-Discharge Rating

Elevation (ft NGVD 29)	Discharge (cfs)
265.0	0.00
265.9	432.06
266.4	543.33
266.9	635.41
267.4	715.73
267.9	787.91
268.4	854.02
268.9	915.36
269.4	972.84
269.9	1027.10
270.4	1078.64
270.9	1127.83

Table 2.1-6. HEC-HMS SBO Bowl Diversion Rating Data

Junction CW 2 Discharge (cfs)	SBO Diversion Discharge (cfs)
2.8	0
14.2	0
28.4	0.08
42.6	0.68
56.8	1.41
71.0	2.10
85.2	2.68
99.4	4.01
113.6	5.32
127.8	6.83
142.0	8.50
156.2	10.32
170.4	12.28
184.6	13.65
198.8	15.08
213.0	17.31
227.2	18.87
241.4	20.47
255.6	22.28
269.8	24.28

Table 2.1-7. HEC-HMS Summary Output

Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)
Subbasin-U3	0.05287	1761.6	03:06	77.79
Subbasin-SW1	0.00080	32.5	03:04	1.18
Subbasin-SW2	0.00103	41.9	03:04	1.52
Junction -SW1	0.00183	74.4	03:04	2.69
Subbasin-SW3	0.00099	40.2	03:04	1.46
Junction-SW2	0.00282	114.6	03:04	4.15
Subbasin-SW5	0.00084	34.1	03:04	1.24
Subbasin-SW4	0.00083	33.7	03:04	1.22
Junction-SW3	0.00449	182.5	03:04	6.61
Subbasin-SW6	0.00051	20.7	03:04	0.75
Subbasin-SW7	0.00071	28.9	03:04	1.04
Junction-SW4	0.00571	232.1	03:04	8.40
Subbasin-CW1	0.00151	61.4	03:04	2.22
Subbasin-CW2	0.00074	30.1	03:04	1.09
Junction-CW2	0.00225	91.5	03:04	3.31
SBO Bowl Diversion	0.00225	88.2	03:04	3.28
Reservoir-U3	0.06083	882.4	03:14	89.47
Subbasin-CE1	0.00160	65	03:04	2.35
Subbasin-CE2	0.00066	26.8	03:04	0.97
Junction-CE2	0.00226	91.9	03:04	3.33
Subbasin-CE3	0.00232	94.3	03:04	3.41
Junction-CE3	0.00458	186.2	03:04	6.74
Subbasin-SE1	0.00144	58.5	03:04	2.12
Subbasin-SE2	0.00087	35.4	03:04	1.28
Junction-SE2	0.00231	93.9	03:04	3.40
Subbasin-SE3	0.00150	61	03:04	2.21
Subbasin-SE4	0.00108	43.9	03:04	1.59
Junction-SE4	0.00489	198.8	03:04	7.19
Subbasin-N3	0.00237	96.3	03:04	3.49
Subbasin-N2	0.00142	57.7	03:04	2.09
Subbasin-B1	0.00389	158.1	03:04	5.72
Subbasin-N1	0.00042	17.1	03:04	0.62
Subbasin-ISFSI	0.01615	422.3	03:10	23.76

Table 2.1-8. Flow Path Cross Sections and Their Corresponding HEC-HMS Elements

Flow Path	HEC-RAS Cross Section	HEC-HMS Discharge Element(s)
CE	438	Sub-basin CE1
	287	Junction CE2
	207	Junction CE3
CW	299	Sub-basin CW1
	136	Junction CW2
SE	431	Junction SE2
	268	Sub-basin SE3 + Junction SE2
	222	Junction SE4
SE Trib	243	Sub-basin SE3
SW	636	Sub-basin SW1
	535	Junction SW1
	386	Junction SW2
	243	Sub-basin SW5 + Junction SW2
	195	Junction SW3
	118	Sub-basin SW6 + Junction SW3
	68	Junction SW4

Table 2.1-9. HEC-RAS Discharges

Flow Path	Cross Section	HEC-HMS Discharge Element(s)	Discharges Input to HEC-RAS (cfs)				
			Profile 1 (2:35)	Profile 2 (2:50)	Profile 3 (2:55)	Profile 4 (3:00)	Profile 5 (3:04)
CE	438	Sub-basin CE1	6.5	14.6	18.3	32.1	65.0
	287	Junction CE2	9.2	20.7	25.8	45.4	91.9
	207	Junction CE3	18.6	41.8	52.4	92.0	186.2
CW	299	Sub-basin CW1	6.1	13.8	17.3	30.3	61.4
	136	Junction CW2	9.1	20.6	25.7	45.2	91.5
SE	431	Junction SE2	9.4	21.1	26.4	46.4	93.9
	268	Sub-basin SE3 + Junction SE2	15.5	34.8	43.5	76.5	154.9
	222	Junction SE4	19.8	44.7	55.9	98.2	198.8
SE Trib	243	Sub-basin SE3	6.1	13.7	17.1	30.1	61.0
SW	636	Sub-basin SW1	3.2	7.3	9.1	16.1	32.5
	535	Junction SW1	7.4	16.7	20.9	36.7	74.4
	386	Junction SW2	11.4	25.8	32.2	56.6	114.6
	243	Sub-basin SW5 + Junction SW2	14.8	35.5	41.8	73.5	148.7
	195	Junction SW3	18.2	41.0	51.3	90.1	182.5
	118	Sub-basin SW6 + Junction SW3	20.3	45.7	57.1	100.3	203.2
	68	Junction SW4	23.2	52.2	65.3	114.6	232.1

Table 2.1-10. HEC-RAS Results

Flow Path	Cross Section	Profile	Total Discharge (cfs)	Water Surface Elevation (ft NGVD 29)
Flow Path SW	636	PF 1	3.2	272.1
		PF 2	7.3	272.3
		PF 3	9.1	272.4
		PF 4	16.1	272.7
		PF 5	32.5	273.2
	577	PF 1	3.2	272.1
		PF 2	7.3	272.3
		PF 3	9.1	272.4
		PF 4	16.1	272.7
		PF 5	32.5	273.2
	535	PF 1	7.4	272.1
		PF 2	16.7	272.3
		PF 3	20.9	272.4
		PF 4	36.7	272.7
		PF 5	74.4	273.2
	473	PF 1	7.4	272.1
		PF 2	16.7	272.3
		PF 3	20.9	272.4
		PF 4	36.7	272.7
		PF 5	74.4	273.2
	433	PF 1	7.4	272.1
		PF 2	16.7	272.3
		PF 3	20.9	272.4
		PF 4	36.7	272.7
		PF 5	74.4	273.2
	386	PF 1	11.4	272.1
		PF 2	25.8	272.3
		PF 3	32.2	272.4
		PF 4	56.6	272.6
		PF 5	114.6	273.1

Table 2.1-10 (Continued). HEC-RAS Results

Flow Path	Cross Section	Profile	Total Discharge (cfs)	Water Surface Elevation (ft NGVD 29)
Flow Path SW	345	PF 1	11.4	272.1
		PF 2	25.8	272.3
		PF 3	32.2	272.4
		PF 4	56.6	272.6
		PF 5	114.6	273.0
	288	PF 1	11.4	272.1
		PF 2	25.8	272.3
		PF 3	32.2	272.4
		PF 4	56.6	272.6
		PF 5	114.6	273.0
	243	PF 1	14.8	272.1
		PF 2	35.5	272.3
		PF 3	41.8	272.4
		PF 4	73.5	272.6
		PF 5	148.7	273.0
	195	PF 1	18.2	272.1
		PF 2	41	272.3
		PF 3	51.3	272.3
		PF 4	90.1	272.5
		PF 5	182.5	272.8
	173	PF 1	18.2	272.1
		PF 2	41	272.3
		PF 3	51.3	272.3
		PF 4	90.1	272.5
		PF 5	182.5	272.7
	118	PF 1	20.3	272.0
		PF 2	45.7	272.1
		PF 3	57.1	272.1
		PF 4	100.3	272.2
		PF 5	203.2	272.5

Table 2.1-10 (Continued). HEC-RAS Results

Flow Path	Cross Section	Profile	Total Discharge (cfs)	Water Surface Elevation (ft NGVD 29)
Flow Path SW	68	PF 1	23.2	271.7
		PF 2	52.2	271.8
		PF 3	65.3	271.9
		PF 4	114.6	272.0
		PF 5	232.1	272.3
	34	PF 1	23.2	271.5
		PF 2	52.2	271.6
		PF 3	65.3	271.6
		PF 4	114.6	271.8
		PF 5	232.1	272.0
Flow Path CW	299	PF 1	6.1	274.1
		PF 2	13.8	274.2
		PF 3	17.3	274.2
		PF 4	30.3	274.3
		PF 5	61.4	274.5
	244	PF 1	6.1	271.4
		PF 2	13.8	271.5
		PF 3	17.3	271.6
		PF 4	30.3	271.8
		PF 5	61.4	272.0
	198	PF 1	6.1	271.3
		PF 2	13.8	271.4
		PF 3	17.3	271.4
		PF 4	30.3	271.5
		PF 5	61.4	271.7
	136	PF 1	9.1	271.3
		PF 2	20.6	271.4
		PF 3	25.7	271.4
		PF 4	45.2	271.5
		PF 5	91.5	271.6

Table 2.1-10 (Continued). HEC-RAS Results

Flow Path	Cross Section	Profile	Total Discharge (cfs)	Water Surface Elevation (ft NGVD 29)
Flow Path CW	85	PF 1	9.1	271.2
		PF 2	20.6	271.3
		PF 3	25.7	271.4
		PF 4	45.2	271.5
		PF 5	91.5	271.6
	46	PF 1	9.1	271.2
		PF 2	20.6	271.3
		PF 3	25.7	271.4
		PF 4	45.2	271.5
		PF 5	91.5	271.6
	0	PF 1	9.1	271.2
		PF 2	20.6	271.2
		PF 3	25.7	271.2
		PF 4	45.2	271.3
		PF 5	91.5	271.4
Flow Path SE Trib	243	PF 1	6.1	272.4
		PF 2	13.7	272.7
		PF 3	17.1	272.8
		PF 4	30.1	273.2
		PF 5	61	273.8
	180	PF 1	6.1	272.4
		PF 2	13.7	272.7
		PF 3	17.1	272.8
		PF 4	30.1	273.2
		PF 5	61	273.8
	98	PF 1	6.1	272.4
		PF 2	13.7	272.7
		PF 3	17.1	272.8
		PF 4	30.1	273.2
		PF 5	61	273.8

Table 2.1-10 (Continued). HEC-RAS Results

Flow Path	Cross Section	Profile	Total Discharge (cfs)	Water Surface Elevation (ft NGVD 29)
Flow Path SE Trib	35	PF 1	6.1	272.4
		PF 2	13.7	272.7
		PF 3	17.1	272.8
		PF 4	30.1	273.2
		PF 5	61	273.8
Flow Path SE	431	PF 1	9.4	272.4
		PF 2	21.1	272.7
		PF 3	26.4	272.8
		PF 4	46.4	273.2
		PF 5	93.9	273.8
	387	PF 1	9.4	272.4
		PF 2	21.1	272.7
		PF 3	26.4	272.8
		PF 4	46.4	273.2
		PF 5	93.9	273.8
	352	PF 1	9.4	272.4
		PF 2	21.1	272.7
		PF 3	26.4	272.8
		PF 4	46.4	273.2
		PF 5	93.9	273.8
	316	PF 1	9.4	272.4
		PF 2	21.1	272.7
		PF 3	26.4	272.8
		PF 4	46.4	273.2
		PF 5	93.9	273.8
	268	PF 1	15.5	272.3
		PF 2	34.8	272.6
		PF 3	43.5	272.7
		PF 4	76.5	273.1
		PF 5	154.9	273.6

Table 2.1-10 (Continued). HEC-RAS Results

Flow Path	Cross Section	Profile	Total Discharge (cfs)	Water Surface Elevation (ft NGVD 29)
Flow Path SE	222	PF 1	19.8	272.2
		PF 2	44.7	272.3
		PF 3	55.9	272.4
		PF 4	98.2	272.7
		PF 5	198.8	273.2
	184	PF 1	19.8	272.2
		PF 2	44.7	272.4
		PF 3	55.9	272.5
		PF 4	98.2	272.7
		PF 5	198.8	273.1
	142	PF 1	19.8	272.2
		PF 2	44.7	272.4
		PF 3	55.9	272.5
		PF 4	98.2	272.7
		PF 5	198.8	273.1
	72	PF 1	19.8	272.2
		PF 2	44.7	272.4
		PF 3	55.9	272.4
		PF 4	98.2	272.6
		PF 5	198.8	272.9
	39	PF 1	19.8	272.1
		PF 2	44.7	272.2
		PF 3	55.9	272.2
		PF 4	98.2	272.4
		PF 5	198.8	272.7
	0	PF 1	19.8	270.7
		PF 2	44.7	270.9
		PF 3	55.9	270.9
		PF 4	98.2	271.1
		PF 5	198.8	271.3

Table 2.1-10 (Continued). HEC-RAS Results

Flow Path	Cross Section	Profile	Total Discharge (cfs)	Water Surface Elevation (ft NGVD 29)
Flow Path CE	438	PF 1	6.5	274.0
		PF 2	14.6	274.1
		PF 3	18.3	274.1
		PF 4	32.1	274.3
		PF 5	65	274.5
	384	PF 1	6.5	271.7
		PF 2	14.6	272.0
		PF 3	18.3	272.1
		PF 4	32.1	272.5
		PF 5	65	273.2
	331	PF 1	6.5	271.7
		PF 2	14.6	272.0
		PF 3	18.3	272.1
		PF 4	32.1	272.5
		PF 5	65	273.2
	287	PF 1	9.2	271.7
		PF 2	20.7	272.0
		PF 3	25.8	272.1
		PF 4	45.4	272.5
		PF 5	91.9	273.2
	237	PF 1	9.2	271.6
		PF 2	20.7	271.9
		PF 3	25.8	272.1
		PF 4	45.4	272.4
		PF 5	91.9	273.1
	207	PF 1	18.6	271.5
		PF 2	41.8	271.7
		PF 3	52.4	271.8
		PF 4	92	272.0
		PF 5	186.2	272.5

Table 2.1-10 (Continued). HEC-RAS Results

Flow Path	Cross Section	Profile	Total Discharge (cfs)	Water Surface Elevation (ft NGVD 29)
Flow Path CE	160	PF 1	18.6	271.5
		PF 2	41.8	271.7
		PF 3	52.4	271.8
		PF 4	92	272.1
		PF 5	186.2	272.6
	120	PF 1	18.6	271.5
		PF 2	41.8	271.7
		PF 3	52.4	271.8
		PF 4	92	272.0
		PF 5	186.2	272.4
	72	PF 1	18.6	271.3
		PF 2	41.8	271.5
		PF 3	52.4	271.5
		PF 4	92	271.7
		PF 5	186.2	272.0
	49	PF 1	18.6	271.2
		PF 2	41.8	271.3
		PF 3	52.4	271.4
		PF 4	92	271.5
		PF 5	186.2	271.7
	0	PF 1	18.6	271.0
		PF 2	41.8	271.1
		PF 3	52.4	271.2
		PF 4	92	271.3
		PF 5	186.2	271.5

Table 2.1-11. Sheet Flow Depths in Sub-basins N1, N2, and N3

Sub-basin	Discharge (cfs)	Weir Length (ft)	Sheet Flow width (ft)	Sheet Flow Slope (ft/ft)	Depth (ft)	Depth (in)	Water Elevation (ft NGVD29)
N1	1.7	105	94.5	0.003	0.04	0.5	271.15
	3.8	105	94.5	0.003	0.06	0.7	271.17
	4.8	105	94.5	0.003	0.07	0.8	271.18
	8.4	105	94.5	0.003	0.10	1.2	271.21
	17.1	105	94.5	0.003	0.15	1.8	271.26
N2	5.8	260	234	0.003	0.04	0.5	270.90
	13.0	260	234	0.003	0.07	0.8	270.93
	16.2	260	234	0.003	0.08	1.0	270.94
	28.5	260	234	0.003	0.11	1.3	270.97
	57.7	260	234	0.003	0.18	2.2	271.04
N3	9.6	275	247.5	0.003	0.06	0.7	270.92
	21.7	275	247.5	0.003	0.09	1.1	270.95
	27.1	275	247.5	0.003	0.11	1.3	270.97
	47.6	275	247.5	0.003	0.15	1.8	271.01
	96.3	275	247.5	0.003	0.23	2.8	271.09

Table 2.1-12. ISFSI Area PMP Water Surface Elevations

Time	PMP Discharge (cfs)	Required Head (ft)	Water Surface Elevation (ft NGVD 29)
2:35	28.6	0.05	310.91
2:50	95.2	0.11	310.97
2:55	129.4	0.13	310.99
3:00	173.0	0.16	311.02
3:05	319.1	0.24	311.10
3:10	422.3	0.29	311.15
3:15	327.4	0.24	311.10

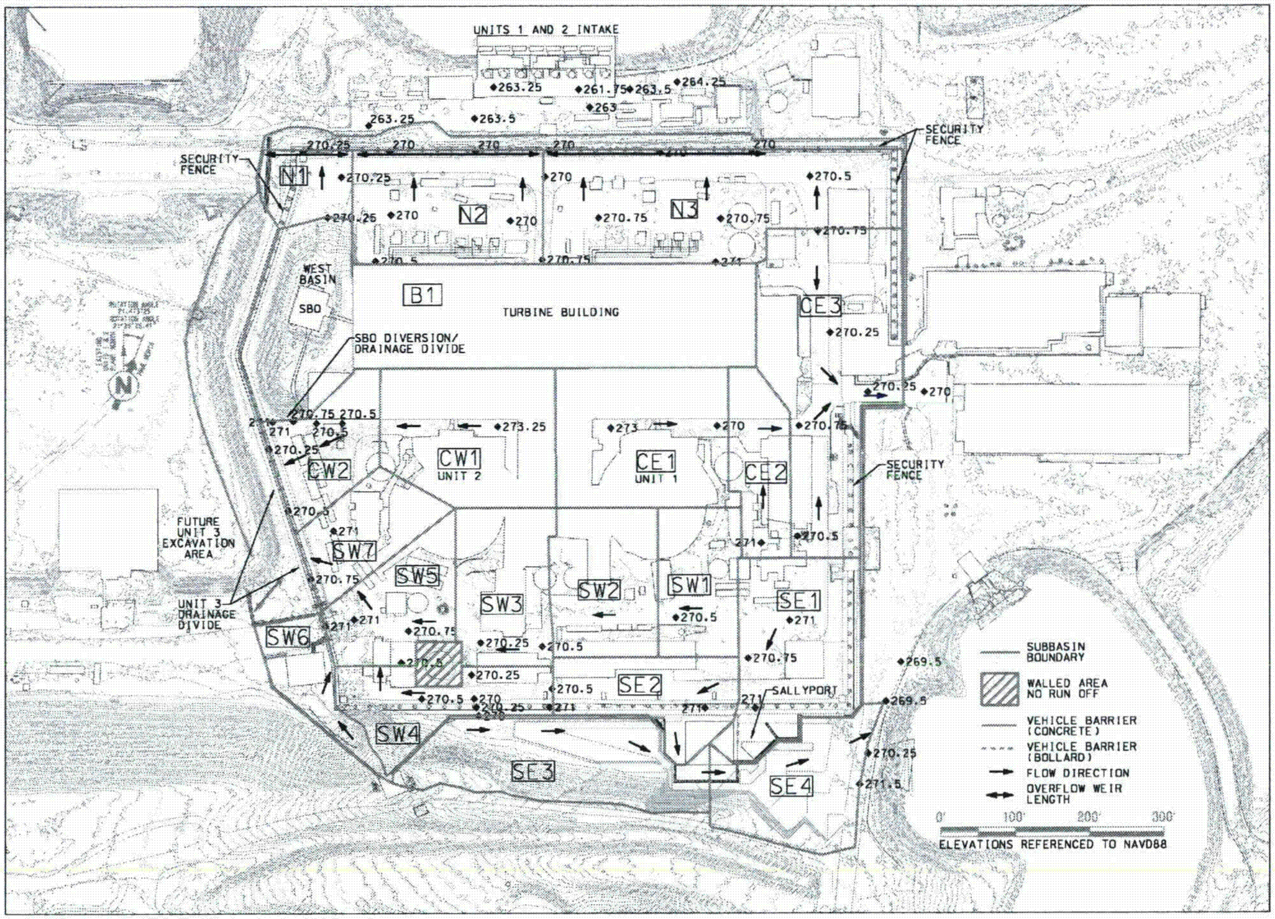

Figure 2.1-1. NAPS Units 1 & 2 Protected Area Sub-basin Drainage Boundaries

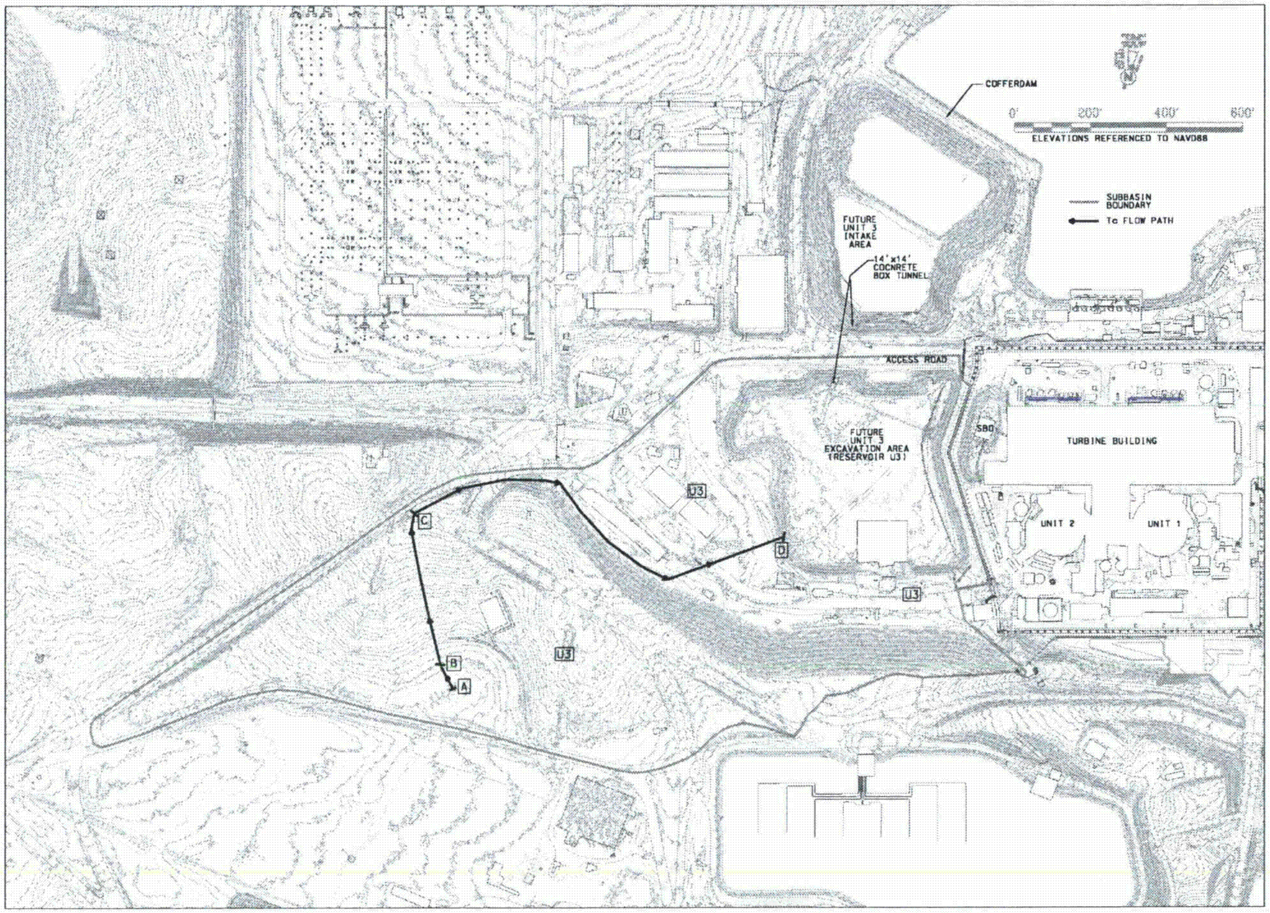

Figure 2.1-2. NAPS Units 1 & 2 Reservoir U3 Sub-basin Drainage Boundary

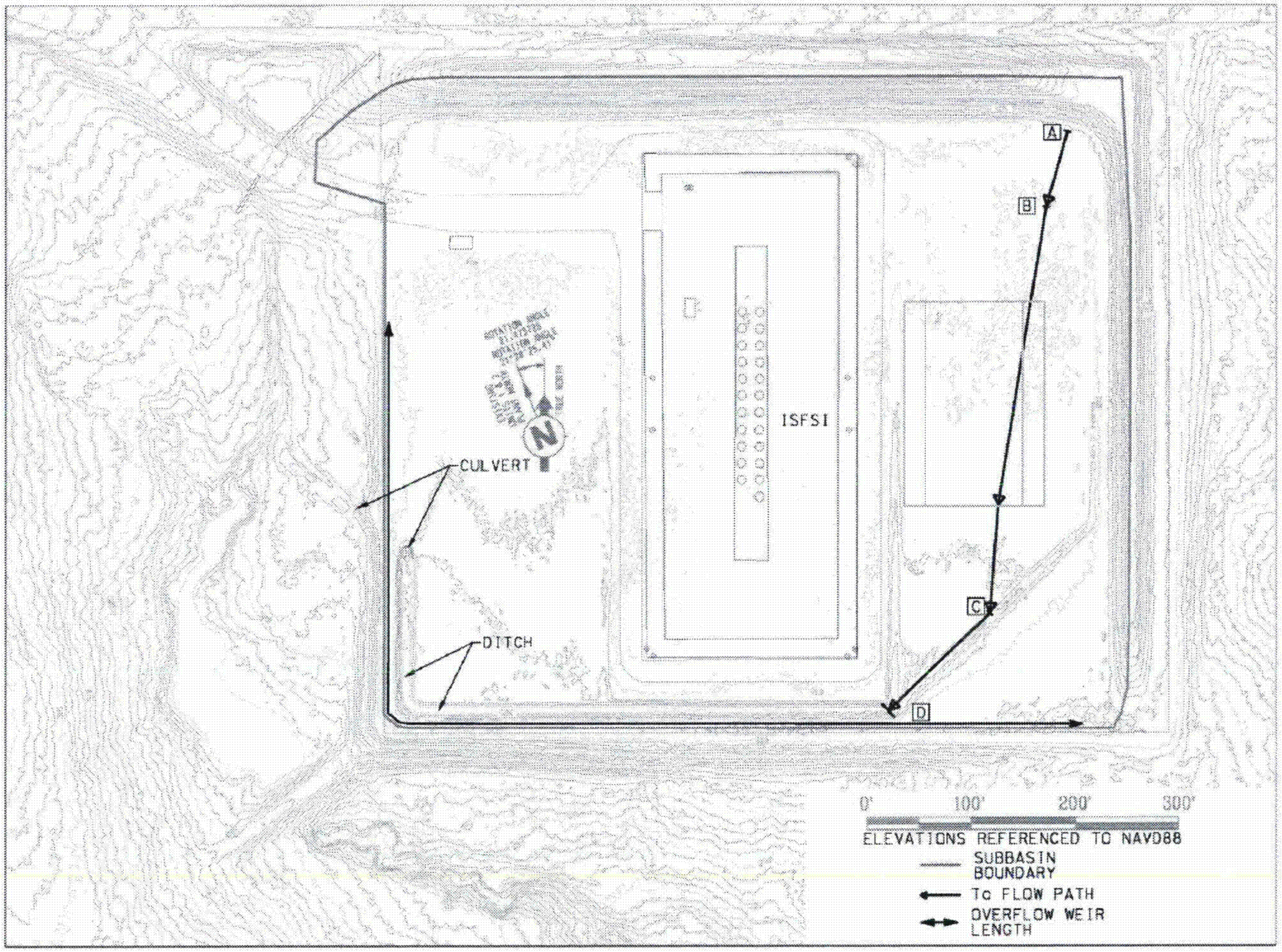

Figure 2.1-3. NAPS Units 1 & 2 ISFSI Sub-basin Drainage Boundary

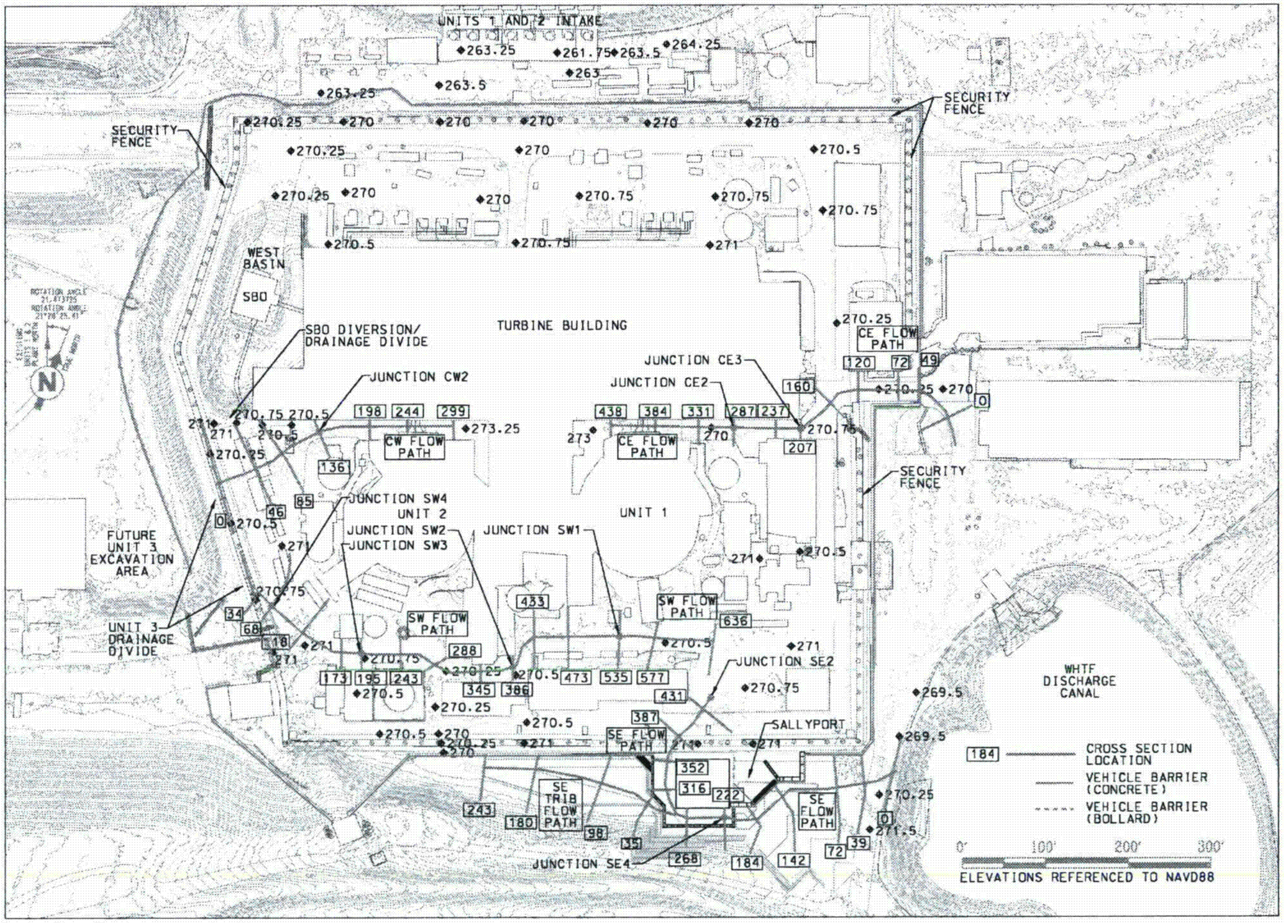

Figure 2.1-4. NAPS Units 1 & 2 Protected Area Flow Path Cross Section Locations

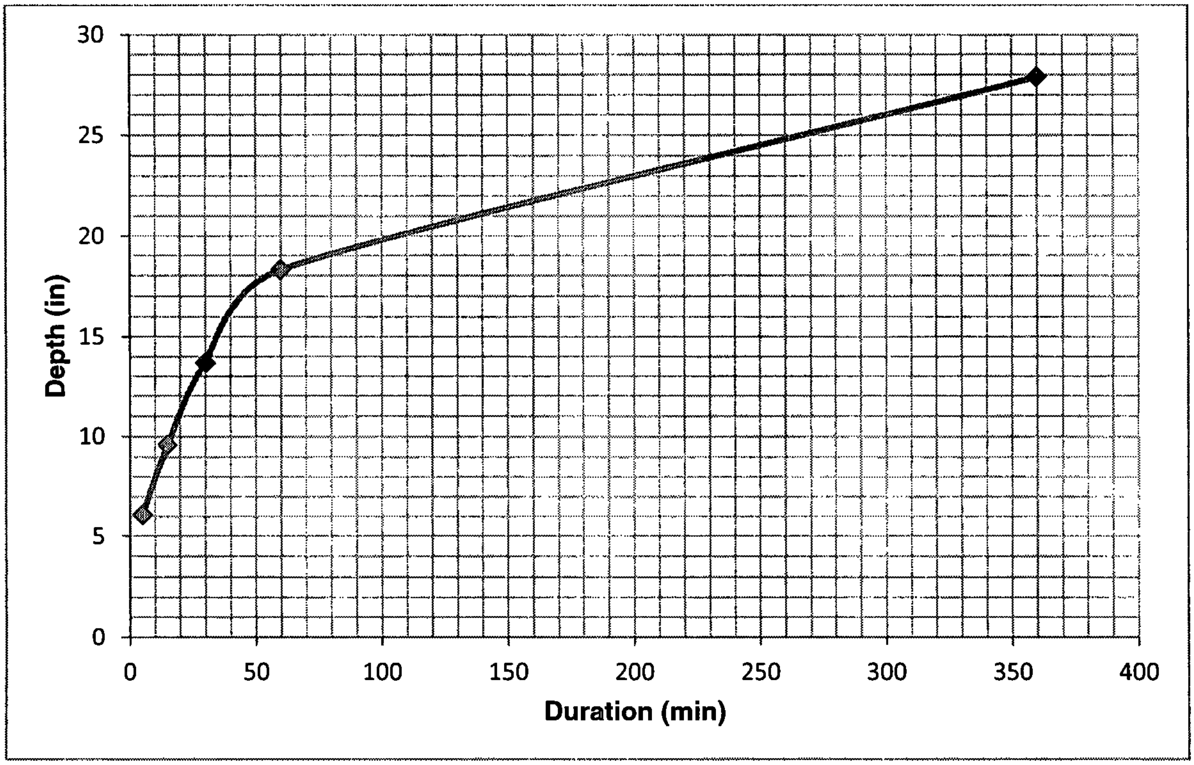

Figure 2.1-5. North Anna Units 1 & 2 Probable Maximum Precipitation Depths vs Duration

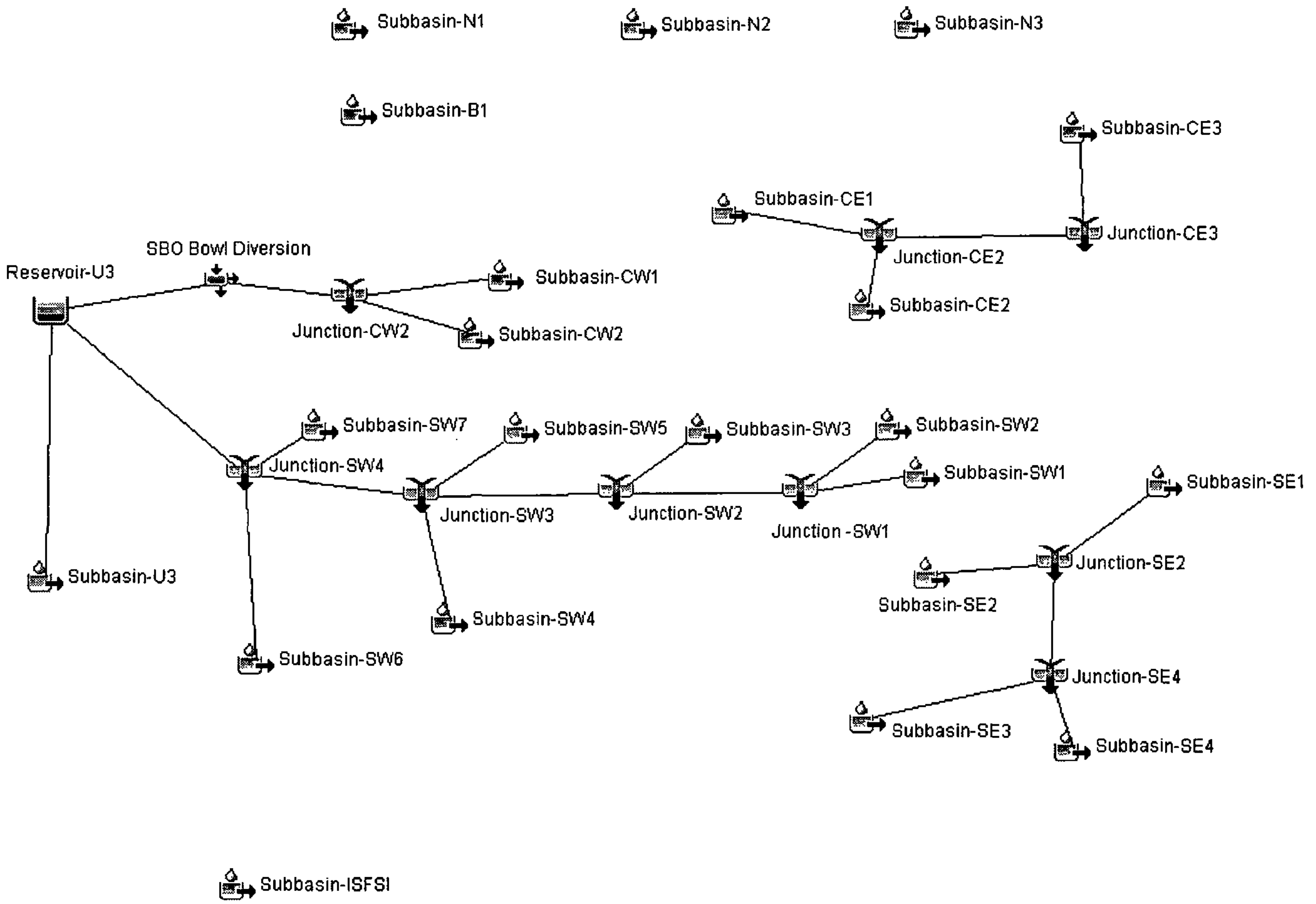

Figure 2.1-6. HEC-HMS Runoff Model Schematic Diagram

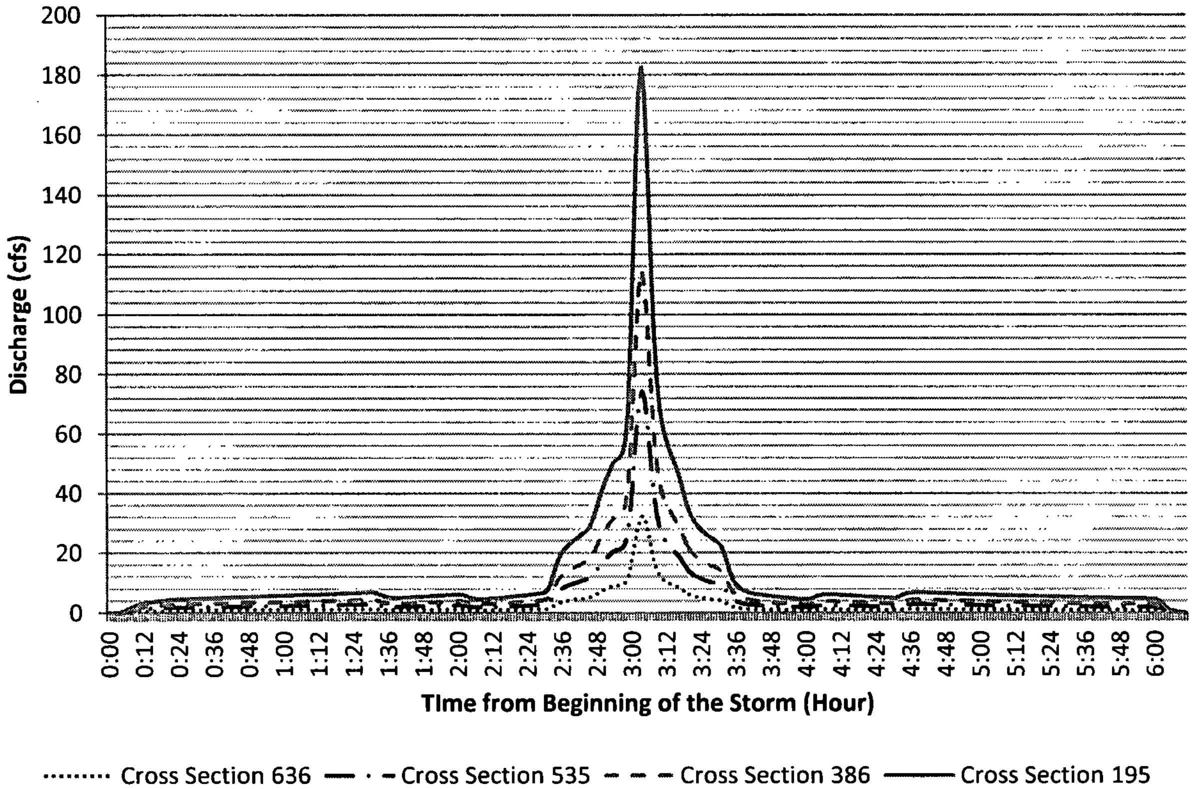

Figure 2.1-7. Flow Path SW PMP Discharge Hydrographs for Selected Cross Sections

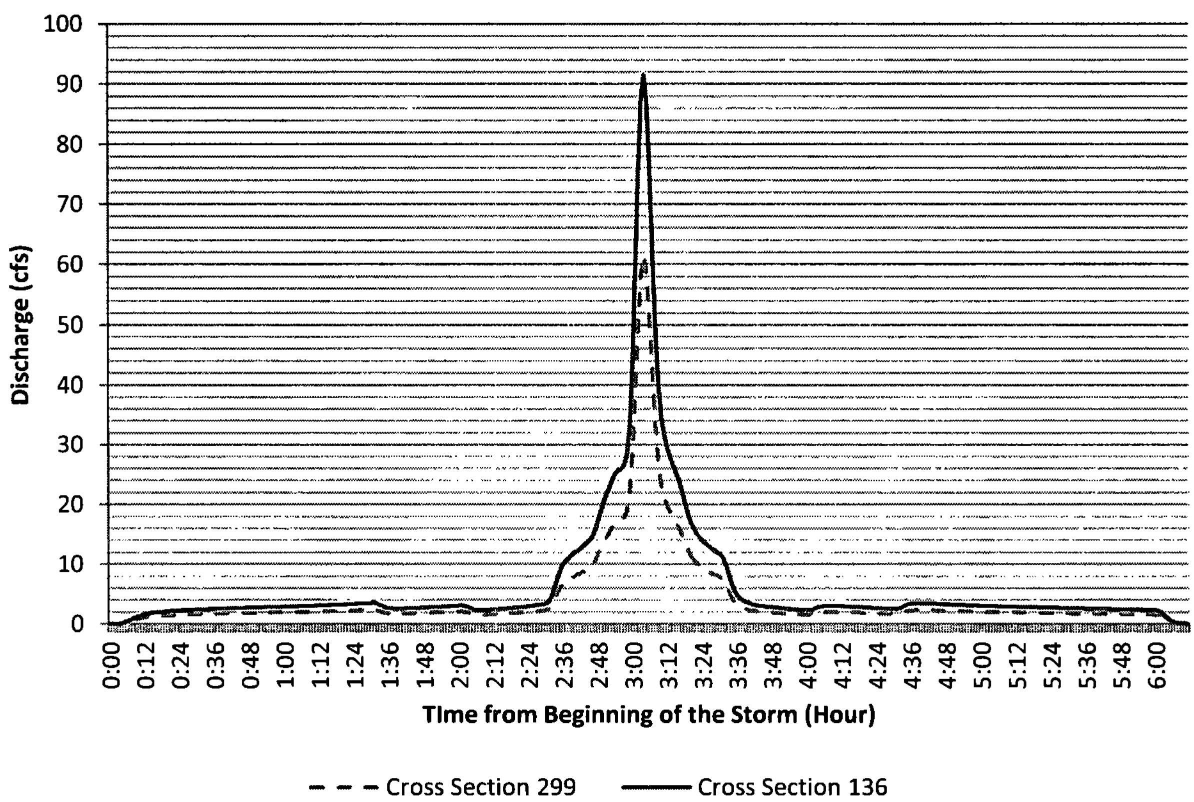

Figure 2.1-8. Flow Path CW PMP Discharge Hydrographs for Selected Cross Sections

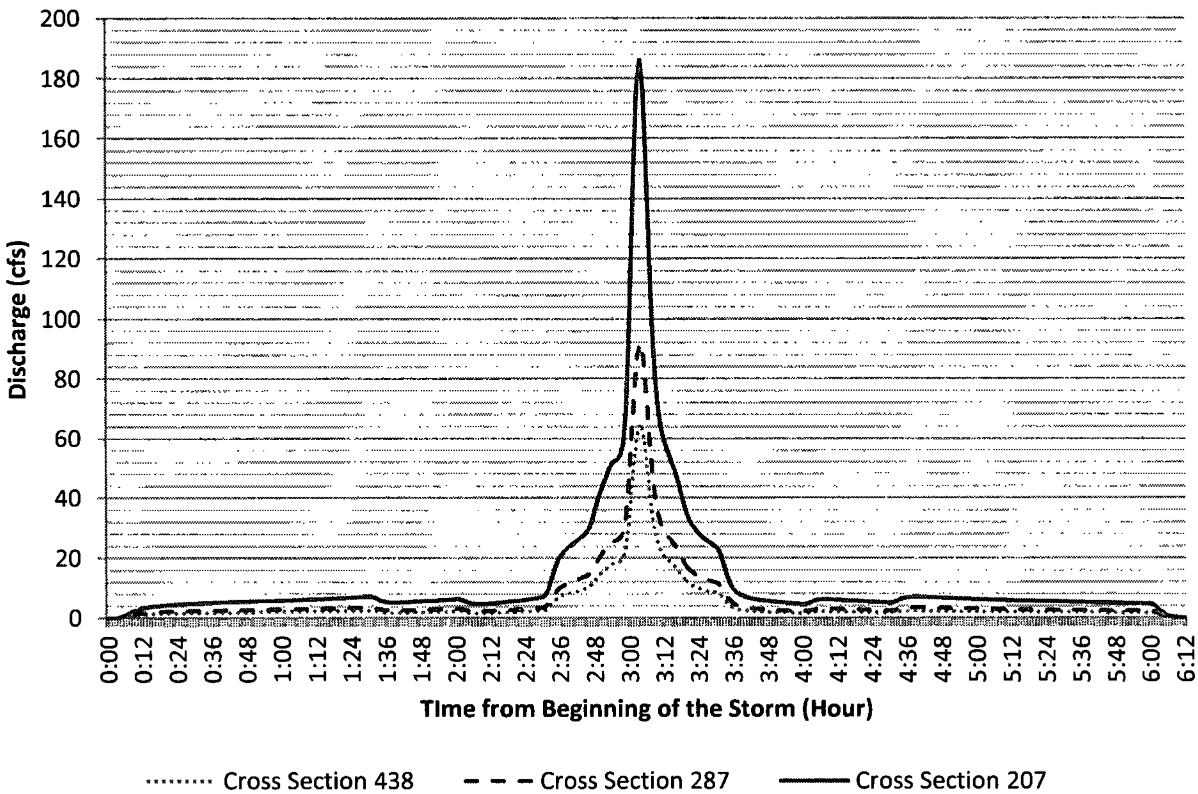

Figure 2.1-9. Flow Path CE PMP Discharge Hydrographs for Selected Cross Sections

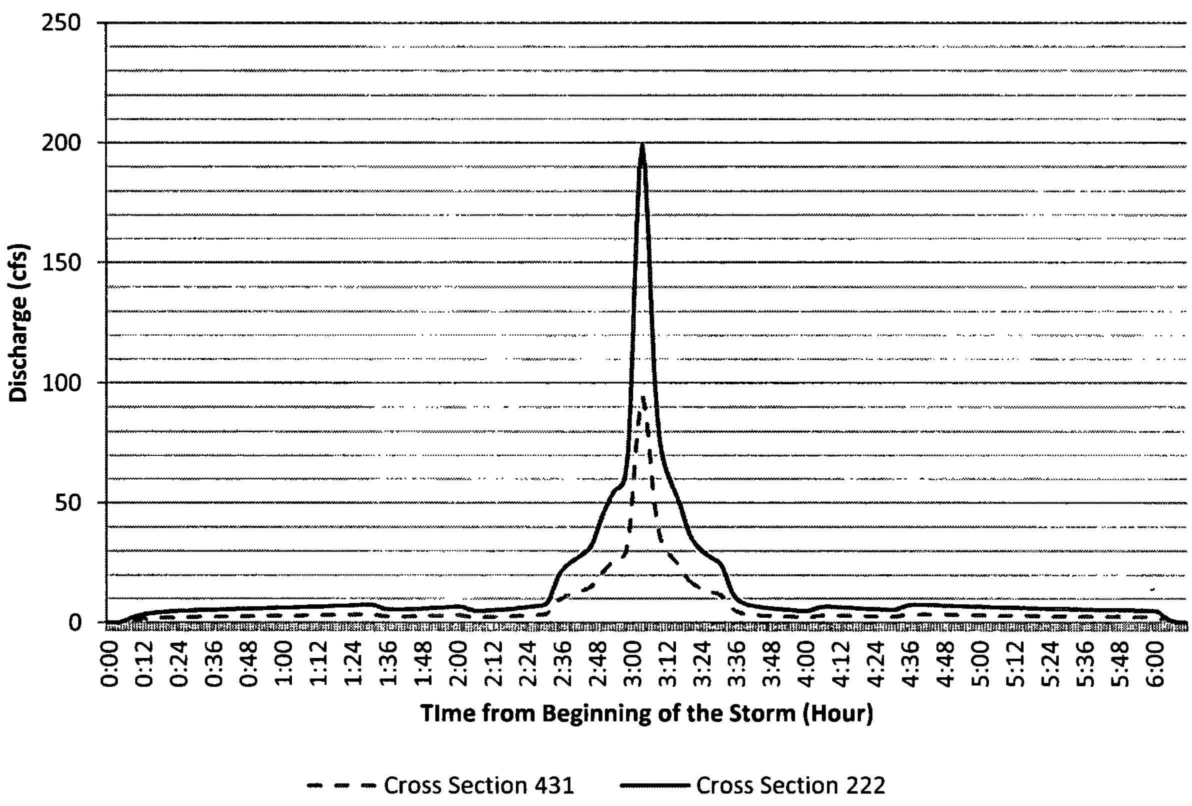

Figure 2.1-10. Flow Path SE PMP Discharge Hydrographs for Selected Cross Sections

2.2 Flooding in Streams and Rivers

Four previous Probable Maximum Flood (PMF) analyses for Lake Anna were performed. The first analysis was performed for the original Units 1 & 2 NAPS Final Safety Analysis Report. The second analysis was performed in 1976 to update the runoff model unit hydrograph based on water level observations since the construction of North Anna Dam. The 1976 analysis is described in the NAPS UFSAR (Section 2.4.3 of Reference 2.2-1). The Probable Maximum Precipitation (PMP) values for a 48-hour storm duration used in the 1976 analysis were based on information contained in the National Weather Service's (NWS's) Hydro-Meteorological Report (HMR) No. 33 (Reference 2.2-2). The estimated maximum PMF water level at NAPS from the 1976 study is 267.3 ft NGVD 29 (Sections 2.4.3.6 and 2A.1 of Reference 2.2-1). Since 1976, the NWS, now under the National Oceanic and Atmospheric Administration (NOAA), has updated PMP estimates and published HMR Nos. 51, 52 and 53 to reflect the updated estimates (References 2.2-3, 2.2-4 and 2.2-5). In general, the PMP estimates in the later HMRs are greater and of longer duration than those presented in HMR No. 33. Thus, the PMF analysis of the Units 1 & 2 flooding reevaluation adopts the updated PMP values from HMR Nos. 51, 52, and 53.

The third Lake Anna PMF analysis (Reference 2.2-6) was performed as part of the Early Site Permit (ESP) application for a future plant adjacent to Units 1 & 2, respectively. The ESP analysis developed the PMP estimates for a 72-hour storm duration from the current HMRs, i.e., Nos. 51, 52 and 53. The runoff unit hydrograph and precipitation losses used in the 1976 study were compared with observed results from storms that have occurred since 1976 and adjusted as necessary. The flood inflow hydrograph and still water elevations in Lake Anna were computed using the U.S. Army Corps of Engineer's (USACE's) Computer Program HEC-1 (Reference 2.2-8). The backwater effects along with appropriate wind-generated setup and wave run-up in accordance with ANS/ANSI-2.8-1992 (Reference 2.2-9) were added to the still water elevation to determine the PMF elevation at the proposed plant site. The analysis resulted in a PMF elevation of 267.39 ft NGVD 29 at the ESP site. Since the proposed plant is adjacent to Units 1 & 2, the procedures and conclusions of the ESP analysis are applicable for the existing station.

The Lake Anna PMF analysis presented in the ESP application was based on a normal pool elevation of 250.00 ft NGVD 29 in Lake Anna (Section 2.4.3.1 of Reference 2.2-1). In the Unit 3 Combined License (COL) application, the fourth PMF analysis (Reference 2.2-7) was performed to include a scenario of a 3 inch increase in the normal pool elevation of Lake Anna, as a result of environmental permit conditions which will require the normal pool elevation to be raised to elevation 250.25 ft NGVD 29 when Unit 3 is operating (Section 2.4.1.1 of Reference 2.2-7). This PMF reevaluation analysis for Units 1 & 2 adopts the 250.25 ft NGVD 29 initial water level as it generates a more conservative flood level. The modeling approach, calibration, and all primary input data from the Lake Anna PMF model presented in the ESP application remained the same for the COL analysis. The input data for the starting water level and the stage-discharge relationship have been revised to reflect the new normal pool elevation. Additionally, the U.S. Army Corps of Engineers (USACE) computer program HEC-HMS (Reference 2.2-10) was used to compute inflow and outflow hydrographs as well as Lake Anna water levels instead of the USACE Computer Program HEC-1, which was used in the ESP PMF analysis. HEC-HMS performs the same function as HEC-1 and is an upgraded program that makes use of modern computer operating systems. All of the methodologies utilized in the ESP HEC-1 analysis with the same input data were utilized in the COL HEC-HMS analysis. For the HEC-HMS model, adjustments to two variables (the Coefficient Ratio and the Recession Ratio) were necessary due to revisions to input parameters for HEC-HMS. Those instances where alterations were required are described in the following subsections. Otherwise, the input parameters described in the ESP application were still valid for the COL HEC-HMS analysis. Initially, the ESP HEC-1 input parameters, without modification to the normal pool elevation, were input into the HEC-HMS model. As mentioned previously, minor adjustments were made to two input variables due to revisions to the input parameters. With these adjustments, the HEC-HMS analysis produced results essentially identical to the results produced in the ESP HEC-1 analysis. Then, the normal pool elevation (starting water level) and the stage-discharge relationship were revised to reflect the raised normal pool for Lake Anna in the HEC-HMS analysis. The results of the HEC-HMS analysis indicated that with a 3 inch increase in the starting water level, the maximum Lake Anna PMF still water level at Lake Anna Dam does not increase and remains at elevation 264.07 ft NGVD 29 (Section 2.4.3 of Reference 2.2-7). Because the still water level at the dam does not increase above that level, the backwater and wind wave activity analysis have not been revised.

In the Units 1 & 2 flooding reevaluation, the unit hydrograph is modified to include the peaking recommendations provided in Sections 3.3.2 and Appendix I.2 of NUREG/CR-7046 (Reference 2.2-11) to account for the non-linearity effect in the basin rainfall-runoff response during extreme storms. Results of this analysis, as described in the following subsections, show that the maximum PMF water level at Units 1 & 2 is 267.4 ft NGVD 29, including wind-wave effects. This elevation is more than 3 ft below the Units 1 & 2 plant grade (elevation of 271.0 ft NGVD 29) (Section 2.4.3 of Reference 2.2-1). In comparison, the NAPS UFSAR documents the maximum PMF flood level at 267.3 ft NGVD 29 at the station (Section 2.4.3.6 of Reference 2.2-1).

Separately, all important areas surrounding the Turbine Building are flood-protected to elevation 257 ft NGVD 29. The Technical Requirements Manual (TRM) requires the station to be taken out of service, the circulating water pumps be secured, and the condenser isolation valves be closed when the lake level exceeds elevation 256 ft NGVD 29 (Section 2.4.3 of Reference 2.2-1).

2.2.1 Probable Maximum Precipitation

The 72-hour PMP was developed according to procedures outlined in HMR Nos. 51, 52, and 53 (References 2.2-3, 2.2-4 and 2.2-5). The values are presented in Table 2.2-1. They have been estimated based on the size and shape of the combined North Anna Reservoir and WHTF watershed drainage area in accordance with the procedures outlined in HMR No. 52 (Reference 2.2-4). The 343 square mile watershed drainage area is shown on Figure 2.2-1. The PMP isohyetal pattern was oriented over the watershed such that the maximum precipitation volume over the entire drainage area has been obtained. The 72-hour PMP storm was temporally distributed according to guidelines in HMR No. 52 and ANS/ANSI-2.8-1992 (References 2.2-4 and 2.2-9) and is shown in Table 2.2-2.

For the runoff analysis, an antecedent storm condition was assumed as indicated in ANS/ANSI-2.8-1992 (Reference 2.2-9). A rainstorm equivalent to 40 percent of the PMP was initially modeled, followed by three days with no precipitation, and then the full 72-hour PMP storm was applied. Based on the historical snowfall information for the NAPS region, snowmelt does not make a significant contribution to flooding situations (Reference 2.2-12). Therefore, antecedent snow-pack conditions were not considered in the PMF analysis.

2.2.2 Precipitation Losses

Precipitation losses for the 1976 study were determined by comparing the rainfall-runoff relationships for various storms. Precipitation losses were determined using historical storms and the HEC-1 loss rate parameter optimization (Section 2.4.3.2 of Reference 2.2-1).

In addition to the historical storms investigated for the 1976 study, three additional storms were investigated in the ESP analysis to determine precipitation losses, including the influence of recent data. The storms occurred in February 1979, March 1994, and June 1995, and were selected because they produced high water levels in the North Anna Reservoir. Hourly precipitation data for these storms were collected from various precipitation gauging stations near the watershed from the National Climatic Data Center (Reference 2.2-13). The Theissen polygon method was used to determine a watershed basin average precipitation for each storm (Reference 2.2-14). The precipitation weighting and basin average precipitation for each storm are shown in Table 2.2-3 through Table 2.2-5. For these three storms, the HEC-1 loss rate parameters were also optimized by comparing the North Anna Dam outflow HEC-1 results with North Anna Dam discharges calculated from observed Lake Anna water levels and gate openings. The precipitation loss rates from the additional storms were factored with the loss rates for the storms analyzed in the 1976 study, and loss rates were determined for the PMF runoff analysis. The loss rates for each of the actual storms and the loss rates for the 1976 and additional PMF storms are shown in Table 2.2-6.

The ESP HEC-1 precipitation loss coefficients listed in Table 2.2-6, DKLTR, ERAIN, RTIOL and STPKR, are replaced in the HEC-HMS model by Initial Range, Exponent, Coefficient Ratio and Initial Coefficient, respectively. With the exception of Coefficient Ratio, all HEC-HMS precipitation loss coefficients have the same definition as their HEC-1 counterparts and adopt the same values. The Coefficient Ratio in HEC-HMS has a definition slightly different from RTIOL of HEC-1, and is assigned a value of 11.055 through a trial and error process until the simulated runoff of the Lake Anna watershed from the HEC-HMS model matches that from the HEC-1 model. The same Coefficient Ratio of the COL HEC-HMS model is used in the present Units 1 & 2 reevaluation model.

2.2.3 Runoff Model

The revised 1976 analysis used the unit hydrograph method to determine the PMF levels in Lake Anna. The unit hydrograph was developed using historical rainfall records from nearby precipitation stations and historical stage-discharge data for the dam. The procedure, as presented in the NAPS UFSAR (Section 2.4.3.3 of Reference 2.2-1), is outlined below:

1. An isohyetal map of total storm rainfall for each storm was plotted and a Thiessen's polygon was drawn on the isohyetal map to determine the distribution of basin rainfall.

2. Mass curves of rainfall were drawn to define the time distribution of rainfall.
3. The base flow was subtracted from the measured stream flow hydrograph to obtain the runoff hydrograph for each storm.
4. The basin infiltration was adjusted to balance rainfall excess with flood runoff.
5. Using the runoff hydrograph and the time distribution of rainfall excess for guidance, the unit hydrograph for each flood was determined.

From the individual unit hydrographs, a composite unit hydrograph for the combined WHTF and North Anna Reservoir watershed was developed. The composite unit hydrograph used in the 1976 HEC-1 runoff model for the combined watershed drainage area (322.7 square miles), excluding the reservoir and WHTF surface areas, is shown on Figure 2.2-2. A separate runoff hydrograph was developed for the drainage area comprising the reservoir and WHTF surface areas (20.3 square miles). This second hydrograph directly reflected the storm precipitation pattern. No infiltration losses were used for the runoff over the combined reservoir and WHTF surface areas.

In the Lake Anna PMF analysis presented in the ESP application, the precipitation data for each of the three additional storms were applied to the 1976 watershed and lake unit hydrographs. The resulting runoff hydrographs were then combined and routed through Lake Anna using the computer program HEC-1 (Reference 2.2-8). The HEC-1 computed discharges from Lake Anna for each storm were then compared with Lake Anna discharges calculated based on gate opening data and water levels measured at the dam during the storms. Adjustments were made to both the base flow and the precipitation loss (infiltration) coefficients. Comparisons of the HEC-1 computed Lake Anna discharges with the discharges based on measured water levels are shown on Figure 2.2-3 through Figure 2.2-5. The results indicated that the 1976 unit hydrograph produced inflow hydrographs that accurately represent the observed lake discharge hydrographs for the additional storms. Thus, the same 1976 unit hydrographs were used in the PMF runoff analysis for the ESP application.

For the Units 1 & 2 flooding re-evaluation study, the unit hydrograph is modified to include the peaking effect to account for non-linear behavior of the flood hydrographs under extreme flood conditions such as those generated by PMFs. Consistent with the guidance in NUREG/CR-7046 the unit hydrograph peak discharge is to be increased by one-fifth and the time to peak is to be reduced by one-third.

The unit hydrograph used in the NAPS UFSAR and the ESP and COL PMF analyses for the future plant has a duration of 30 hours and the discharge peaks at 12 hours after the storm starts. With one third reduction in the time to peak as recommended in NUREG/CR-7046, the revised time to peak is eight hours. To preserve the 3-hour discretization interval used in the ESP and COL unit hydrograph, the time to peak is conservatively reduced by 50%, i.e., by 6 hours. The revised unit hydrograph with the reduced time to peak and the 20% increase of the peak discharge is shown on Figure 2.2-6. For comparison, the original unit hydrograph from the previous Units 1 & 2 analysis is also shown on Figure 2.2-6. Table 2.2-7 and Table 2.2-8 provide information for the original unit hydrograph and the peaked unit hydrograph, respectively.

Routing of flood flows through Lake Anna was accomplished using the level pool reservoir routing procedure in HEC-HMS. For modeling purposes, the reservoir and the WHTF were treated as a single storage facility, Lake Anna. Four dividing dikes, one of which allows limited flow exchange, separate the two facilities. The top crest elevation of the dikes is 260 ft NGVD 29 (Section 2.4.8 of Reference 2.2-1). However, there is a 350-foot long saddle in Dike 3 at elevation 253.5 ft NGVD 29, which functions as a spillway for the WHTF. Thus, once the water level in either storage facility rises above 253.5 ft NGVD 29, equalization of the water level between the two facilities occurs. In view of the fact that flow between the two facilities is restricted for elevations below 253.5 ft NGVD 29, the reservoir modeling used in HEC-HMS conservatively assumed that all rainfall and runoff was routed only through the North Anna Reservoir until the water level reached elevation 253.5 ft NGVD 29. This is equivalent to assuming that the WHTF was full to elevation 253.5 ft NGVD 29 at the beginning of the PMF. The Lake Anna stage-storage data provided to the HEC-HMS model reflected the conservative modeling approach for the WHTF. For elevations below 253.5 ft NGVD 29, only the North Anna Reservoir's storage volume was input into the model and made available for runoff and rainfall storage. For elevations above 253.5 ft NGVD 29, the storage from both facilities was input into the model and made available. The stage-storage curve for the combined WHTF and North Anna Reservoir, reflecting the conservative approach described, is shown on Figure 2.2-7. The present Units 1 & 2 PMF reevaluation model uses this same stage-storage curve for Lake Anna as in the PMF models documented in the ESP and COL applications.

Two adjustable skimmer gates and three spillway radial gates provide control of the discharge from the North Anna Dam, as described in the NAPS UFSAR (Section 2.4.1.2 of Reference 2.2-1). The stage-discharge relationship used in the HEC-HMS runoff model was based on the adopted spillway rule curve. The input data for the starting water level and the stage-discharge relationship have been revised to reflect the future normal pool elevation of 250.25 ft NGVD 29 when Unit 3 will be operating. Because outflow from the dam is controlled by the positions of the skimmer gates and the radial gates, only the portion of the discharge rating data near the normal pool elevation is revised. During flooding events with higher water levels, the same operating procedures and gate openings are used. The physical geometry of the dam has not changed as a result of the raised normal pool elevation. The skimmer gate and spillway discharge capacities remain the same as in the NAPS UFSAR (Section 2.4.1.2 of Reference 2.2-1) and are shown on Figure 2.2-8 and Figure 2.2-9.

The present PMF runoff analysis was performed by applying the PMP values in Section 2.2.1 to the watershed and lake surface area unit hydrographs, combining the two hydrographs, and routing the resultant inflow hydrograph through Lake Anna.

The HEC-1 base flow variable defined as the Recession Ratio (RR) and used in the ESP PMF analysis is defined as the Recession Constant (RC) in HEC-HMS and has a different definition. HEC-HMS provides a formula to convert the RR to an RC as shown below (Reference 2.2-15):

$$RC = \frac{1}{RR^{24}}$$

The RR value used in the ESP HEC-1 model is 1.0135; thus the RC value used in the present HEC-HMS model is 0.72482.

2.2.4 Probable Maximum Flood Flow

The computed PMF inflow hydrograph to the combined WHTF and North Anna Reservoir is shown in Figure 2.2-10. The peak PMF inflow discharge is about 339,840 cfs, and the peak discharge over the dam is about 141,400 cfs. The controlling PMF hydrograph shows a result of the runoff from a 72-hour storm with precipitation values equal to 40 percent of the PMP, followed by three days with no precipitation and then the 72-hour PMP storm.

There are no other dams in existence on the North Anna River, either upstream or downstream of the NAPS Units 1 & 2 site. The only impoundments in the Lake Anna drainage area are small farm ponds and two small recreational lakes, Lake Louisa and Lake Orange, whose failures would not produce any measurable effect on the Lake Anna water levels. Thus, these effects were not included in the PMF flow.

2.2.5 Water Level Determination

The PMF inflow hydrograph was routed through the combined reservoir using HEC-HMS (Reference 2.2-15) to determine the maximum still water level associated with the PMF. This routing resulted in a peak outflow of 141,414 cfs with a maximum still water level of 264.10 ft NGVD 29 at the dam. In comparison, the 1976 analysis, documented in the NAPS UFSAR (Section 2.4.1.2 of Reference 2.2-1), resulted in a peak outflow discharge of 142,000 cfs and a peak still water level of 264.2 ft NGVD 29.

For the 1976 analysis, included in the NAPS UFSAR, a backwater profile curve was developed for the peak discharge of 142,000 cfs, indicating the lake level at the NAPS site to be about 0.2 ft higher than the water level at the dam (Section 2.4.3.5 of Reference 2.2-1). Since the peak outflow discharge for the present analysis is slightly less than the previous discharge, the results of the previous backwater analysis have been conservatively applied to the elevation computed for this PMF analysis. By adding the backwater effect of 0.2 ft to the PMF still water elevation of 264.10 ft NGVD 29 at the dam, the PMF still water elevation at the site is 264.30 ft NGVD 29.

2.2.6 Coincident Wind Wave Activity

In the ESP analysis the wave setup and run-up generated by a 2-year return period wind speed were added to the PMF still water elevation to determine the maximum PMF water level at the ESP site (Section 2.4.3.6 of Reference 2.2-6). The 2-year overland wind speed for the site was determined by investigating data presented in ANS/ANSI 2.8-1992 (Reference 2.2-9) and NUREG/CR-2639 (Reference 2.2-16). From these two references a fastest-mile 2-year wind speed of 50 mph, measured 30 ft above the ground over land, was selected. This translates to a fastest-mile 2-year wind speed over water of 56.0 mph (Reference 2.2-17). The fetch diagram used to determine an effective fetch length of 4700 ft with a maximum fetch of 10,600 ft is shown on Figure 2.2-11.

Using these values and procedures outlined in the U.S. Bureau of Reclamation publication, *Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams,* (Reference 2.2-17) and the USACE–*Shore Protection Manual* (Reference 2.2-18), consistent with the ESP and COL analyses, a significant wave height of 2.15 ft and a maximum wave height of 3.60 ft were calculated. From these values a maximum wind set-up value of 0.09 ft and a wave run-up value of 3.03 ft were calculated.

Since the existing station (Units 1 & 2) is adjacent to the proposed Unit 3 site, the wind-wave analysis for the new plant are applicable for Units 1 & 2 because they share the same set of important physical parameters that are relevant to the predictions of wind-wave effects. In addition, the simulated PMF still water level change at the dam is negligible, about 0.03 ft, between the Unit 3 analysis and the present analysis. As a result, the backwater and wind-wave activities for Unit 3 PMF analysis are adopted directly for Units 1 & 2 PMF reevaluation. Adding the wind setup and wave run-up values to the PMF still water elevation resulted in a maximum PMF elevation at Units 1 & 2 of 267.42 ft NGVD 29 (=264.10 ft NGVD 29 still PMF level at the dam + 0.2 ft to the station + 0.09 ft wind setup + 3.03 ft wave runup). In comparison, the maximum PMF elevation reported in NAPS UFSAR (Section 2.4.3.6 of Reference 2.2-1) is 0.12 ft lower at 267.3 ft NGVD 29.

2.2.7 Conclusion

The reevaluated still water flood elevation at NAPS Units 1 & 2 as a result of flooding in streams and rivers is 264.3 ft NGVD 29, or 267.4 ft NGVD 29 with coincidental wave run-up. This flood elevation is 3.6 ft below the grade elevation of safety-related SSCs at 271.0 ft NGVD 29 and therefore there is no impact to safety-related operations.

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2.2-15 U.S. Army Corps of Engineers, HEC-HMS, Hydrologic Modeling System, Version 3.5.0, Hydrologic Engineering Center, August 2010.

2.2-16 NUREG/CR-2639, Historical Extreme Winds for the United States – Atlantic and Gulf of Mexico Coastlines, U.S. Nuclear Regulatory Commission, May 1982.

2.2-17 ACER Technical Memorandum No. 2, Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams, Assistant Commissioner – Engineering Research, U.S. Bureau of Reclamation, December 1981.

2.2-18 Shore Protection Manual, Volumes 1 and 2, Fourth Edition, Coastal Engineering Research Center, Waterways Experiment Station, U.S. Army Corps of Engineers, 1984.

Table 2.2-1. Probable Maximum Precipitation Depths

6-hour Incremental Depths		Total PMP Depths	
6-hour Increment	Incremental PMP Depth (in)	Storm Duration (hr)	Total PMP Depth (in)
1	17.71	6	17.71
2	3.67	12	21.38
3	2.24	24	24.89
4	1.27	48	29.09
5	1.27	72	30.65
6	1.07		
7	0.98		
8	0.88		
9	0.59		
10	0.39		
11	0.29		
12	0.29		

Table 2.2-2. Probable Maximum Precipitation Temporal Distribution

Time		Incremental PMP
Time (hours)	Increment	Depth (inches)
0 to 6	12	0.29
6 to 12	11	0.29
12 to 18	10	0.39
18 to 24	9	0.59
24 to 30	4	1.27
30 to 36	2	3.67
36 to 42	1	17.71
42 to 48	3	2.24
48 to 54	5	1.27
54 to 60	6	1.07
60 to 66	7	0.98
66 to 72	8	0.88

Table 2.2-3. February 1979 Rainfall Data (Inches)

Date	Time	Columbia #44192900 (3 mi^2)	Piedmont #44671200 (307 mi^2)	Elkwood #44272900 (34 mi^2)	Basin Weighted Average
24 Feb	6am – 9am	0.0	0.0	0.0	0.00
	9am – 12pm	0.6	0.4	0.3	0.39
	12pm – 3pm	0.3	0.5	0.6	0.51
	3pm – 6pm	0.5	0.3	0.3	0.30
	6pm – 9pm	0.8	0.5	0.3	0.48
	9pm – 12am	0.1	0.1	0.1	0.10
25 Feb	12am – 3am	0.0	0.0	0.1	0.01
	3am – 6am	0.0	0.0	0.0	0.00
	6am – 9am	0.1	0.0	0.0	0.00
	9am – 12pm	0.0	0.0	0.0	0.00
	12pm – 3pm	0.3	0.1	0.0	0.09
	3pm – 6pm	0.6	0.6	0.7	0.61
	6pm – 9pm	0.2	0.3	0.4	0.31
	9pm – 12am	0.1	0.1	0.4	0.13
	Total (in)	3.6	2.9	3.2	2.94

Table 2.2-4. March 1994 Rainfall Data (Inches)

Date	Time	Bremo Bluff #440399302 (0 mi^2)	Culpeper #44215904 (343 mi^2)	Richmond #44720102 (0 mi^2)	Basin Weighted Average
27 Mar	12am – 3am	0.0	0.0	0.00	0.00
	3am – 6am	0.2	0.4	0.01	0.40
	6am – 9am	0.7	0.7	0.43	0.70
	9am – 12pm	0.0	0.1	0.46	0.10
	12pm – 3pm	0.0	0.0	0.00	0.00
	3pm – 6pm	0.0	0.0	0.00	0.00
	6pm – 9pm	0.3	0.2	0.00	0.20
	9pm – 12am	0.2	0.5	0.44	0.50
28 Mar	12am – 3am	0.5	0.7	0.01	0.70
	3am – 6am	0.4	0.3	0.00	0.30
	6am – 9am	0.3	0.1	0.12	0.10
	9am – 12pm	0.2	0.0	0.32	0.00
	12pm – 3pm	0.0	0.0	0.04	0.00
	3pm – 6pm	0.1	0.1	0.10	0.10
	6pm – 9pm	0.1	0.3	0.05	0.30
	9pm – 12am	0.2	0.2	0.15	0.20
29 Mar	12am – 3am	0.3	0.20	0.20	0.20
	3am – 6am	0.2	0.19*	0.17	0.19
	6am – 9am	0.1	0.06*	0.02	0.06
	9am – 12pm	0.0	0.04*	0.07	0.04
	Total (in)	3.8	4.1	2.59	4.09

* Due to missing data at the station, the value is estimated from data at Richmond and Bremo Bluff.

Table 2.2-5. June 1995 Rainfall Data (Inches)

		Station Measured Precipitation (in)		
Date	**Time**	**Piedmont #44671204 (343 mi^2)**	**Richmond #44720102 (0 mi^2)**	**Basin Weighted Average**
27 Jun	12am – 3am	0.7	0.00	0.70
	3am – 6am	1.7	0.00	1.70
	6am – 9am	3.4	0.00	3.40
	9am – 12pm	0.1	0.00	0.10
	12pm – 3pm	0.0	0.00	0.00
	3pm – 6pm	0.0	0.02	0.00
	6pm – 9pm	0.0	0.00	0.00
	9pm – 12am	0.0	0.00	0.00
	Total (in)	5.9	0.02	5.90

Table 2.2-6. Lake Anna Watershed HEC-1 Precipitation Loss Rates

	HEC-1 Precipitation Loss Coefficients			
Storm	**DKLTR**	**ERAIN**	**RTIOL**	**STRKR**
June 1972*	4.02	0.55	3.86	0.440
April 1973*	1.93	0.34	22.07	0.140
March 1975*	0.00	0.52	10.39	0.120
1976 PMF Storm based on HMR 33	2.00	0.47	12.11	0.233
February 1979	0.00	0.60	12.11	0.010
March 1994	0.80	0.55	12.11	0.100
June 1995	5.20	0.50	12.11	0.150
ESP/COL PMF Storm based on HMRs 51, 52	1.37	0.54	12.11	0.100

* Storms investigated in 1976 PMF analysis.

Notes: ERAIN – Exponent of precipitation for loss rate function; RITOL – Loss coefficient recession constant; STRKR – Initial value of loss coefficient (in/hr); DKLTRR – Initial accumulated rain loss during which the loss coefficient is increased (in).

Table 2.2-7. Original Lake Anna Watershed Unit Hydrograph (Excluding Reservoir Surface Area)

Time (hrs)	0	3	6	9	12	15	18	21	24	27	30
Flow (cfs)	0	2176	7471	12082	14890	14166	10867	5073	1743	599	0

Peak Discharge: 14,890 cfs

Time to Peak 12 hrs

Runoff Volume: 207,201 ft^3

Drainage Area: 322.7 mi^2

Runoff Depth: 1.0 in

Table 2.2-8. Peaked Lake Anna Watershed Unit Hydrograph (Excluding Reservoir Surface Area)

Time (hrs)	0	3	6	9	12	15	18	21	24	27	30
Flow (cfs)	0	10200	17868	16000	11157	6000	3500	2000	1743	599	0

Peak Discharge: 17,868 cfs

Time to Peak: 6 hrs

Runoff Volume: 207,201 ft^3

Drainage Area: 322.7 mi^2

Runoff Depth: 1.0 in

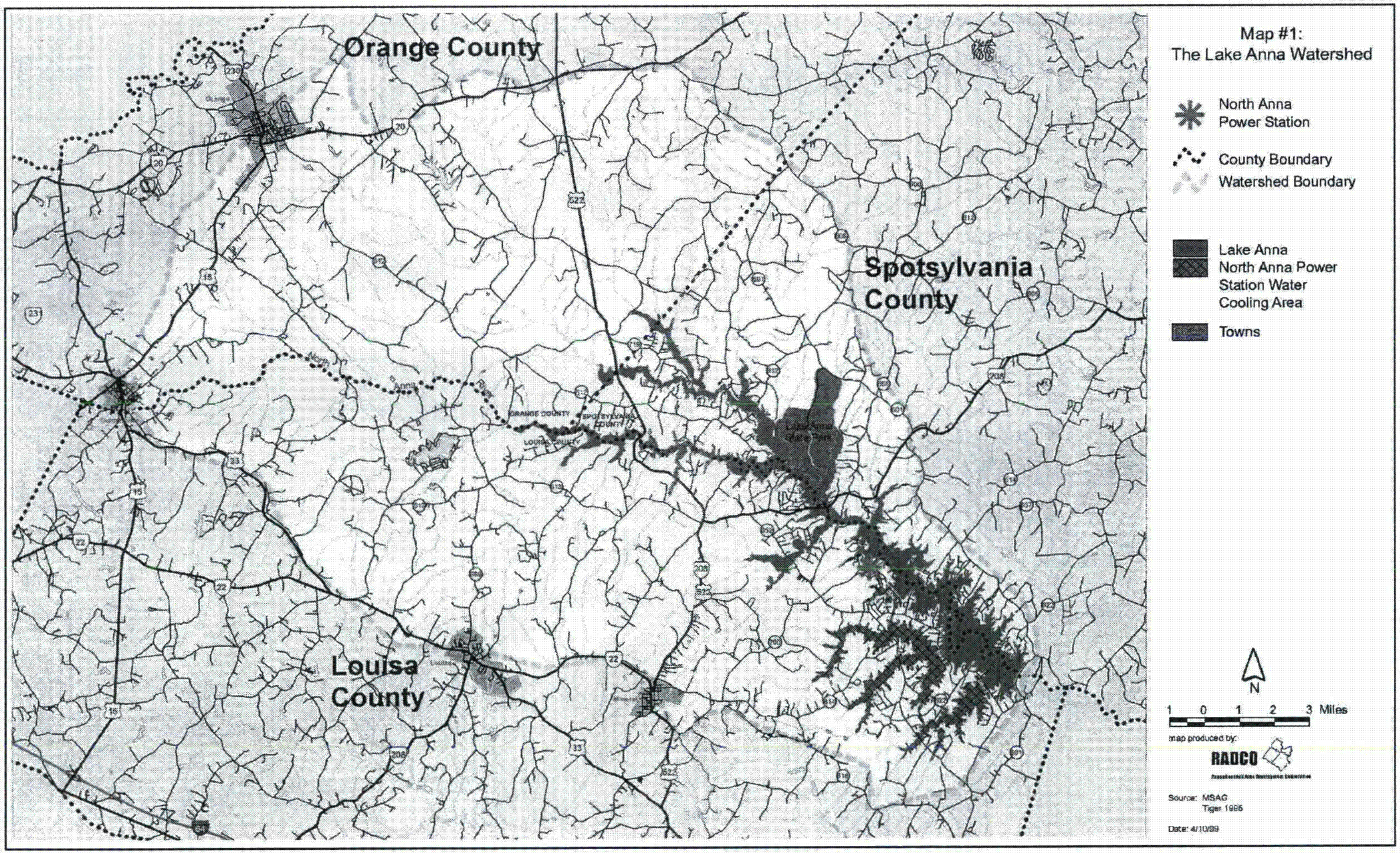


Figure 2.2-1. Combined Lake Anna and WHTF Drainage Area

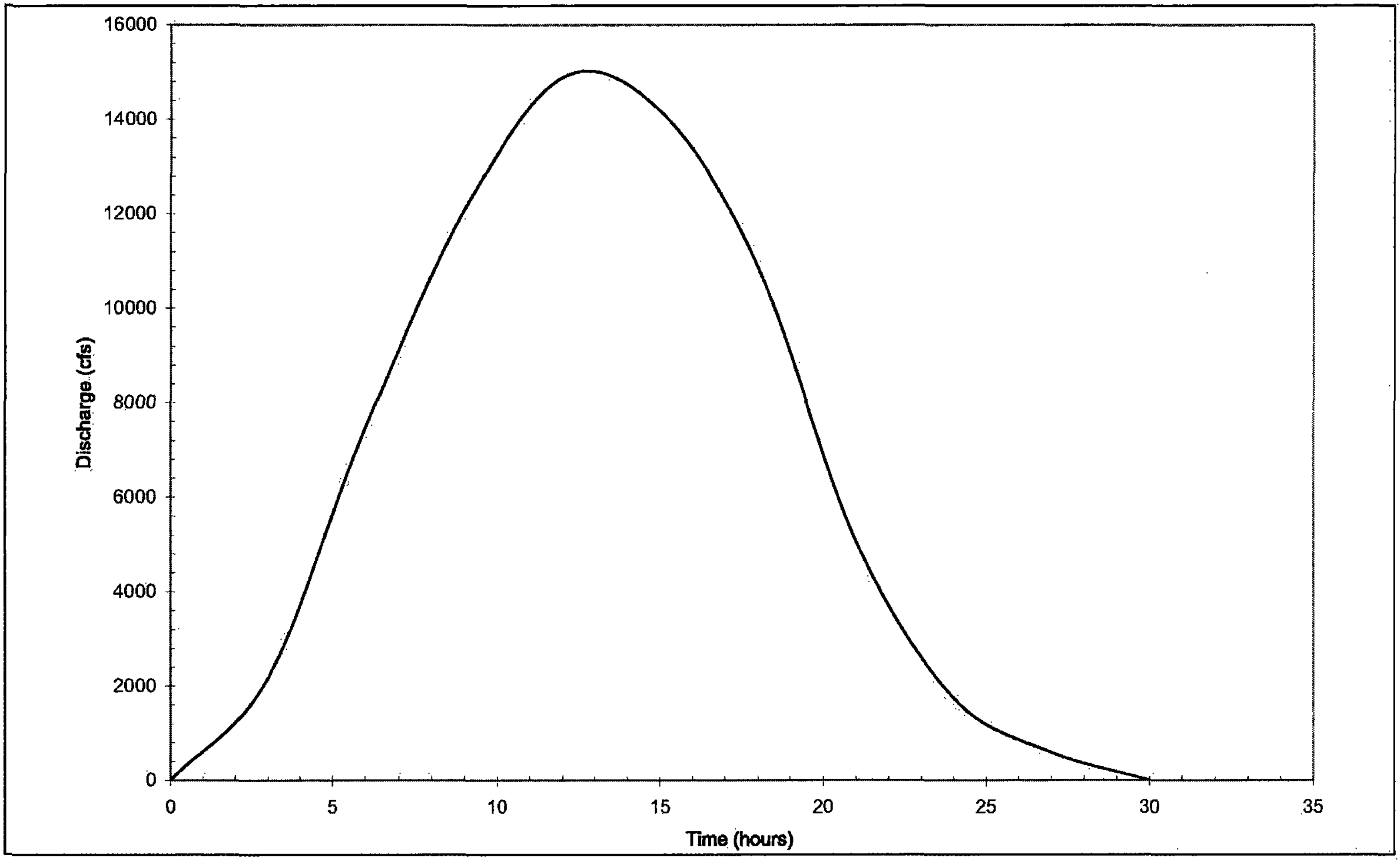


Figure 2.2-2. Combined North Anna Reservoir and WHTF Watershed: Unit Hydrograph

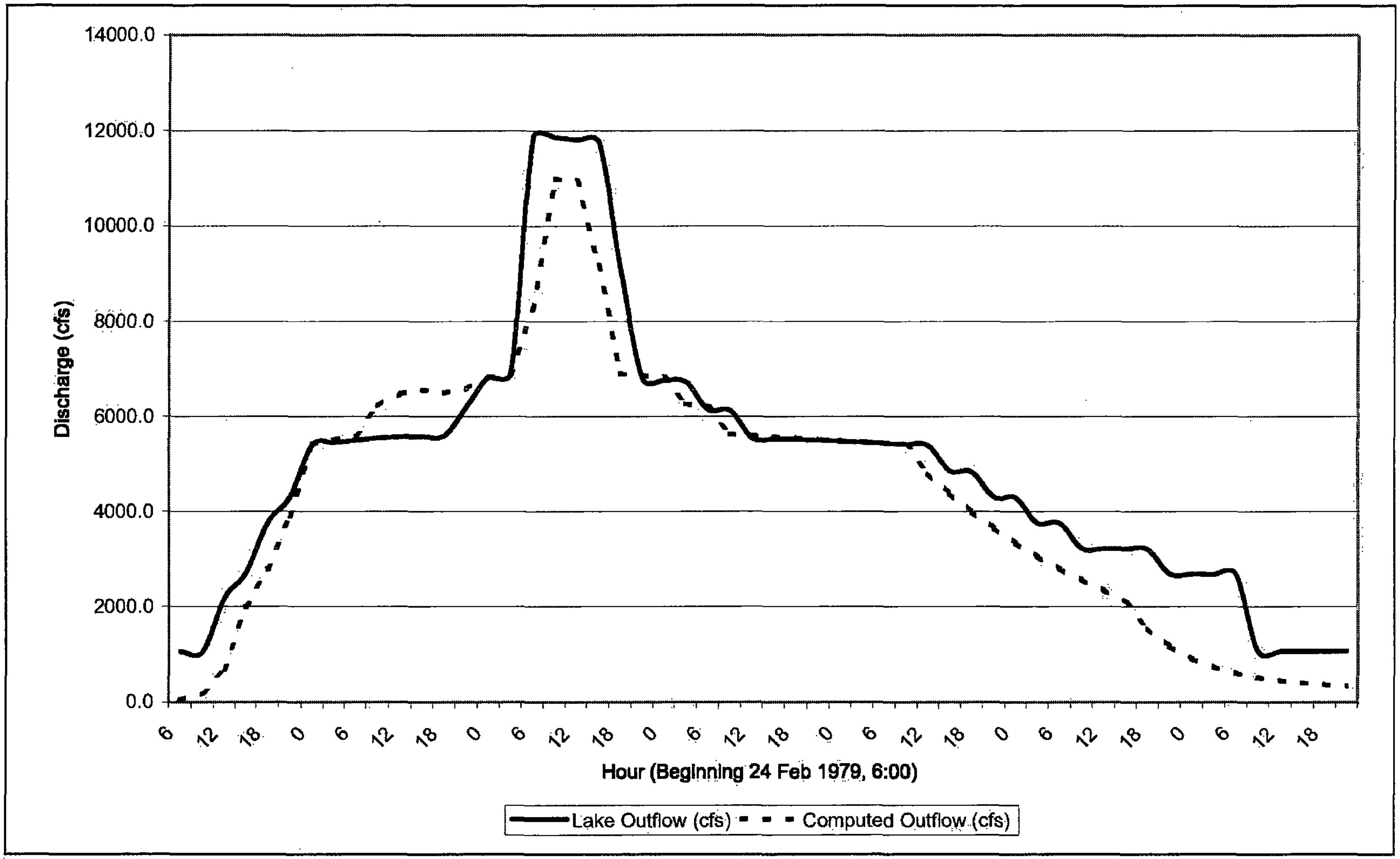


Figure 2.2-3. 1979 Storm Outflow Hydrograph Comparison

Figure 2.2-4. 1994 Storm Outflow Hydrograph Comparison

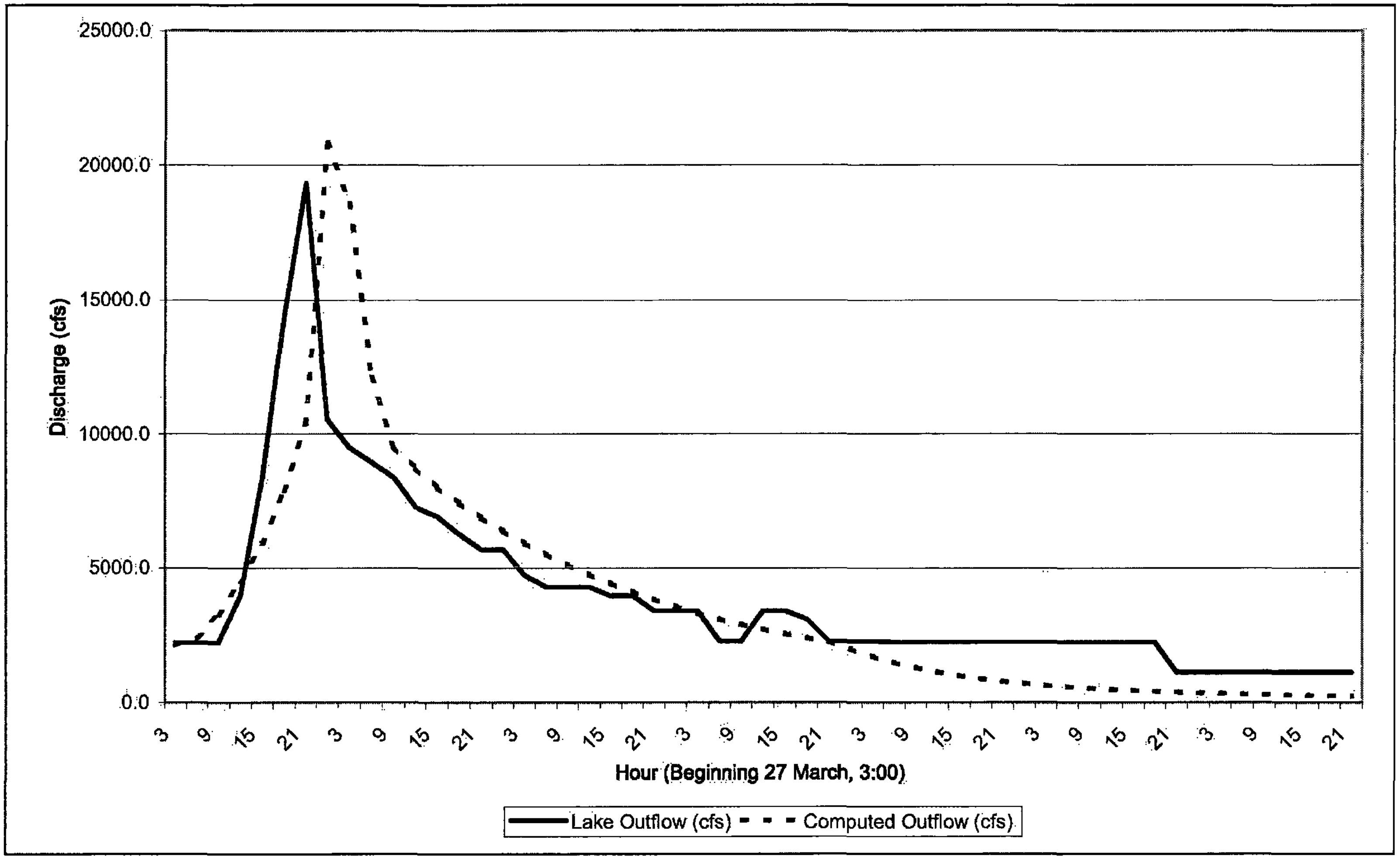


Figure 2.2-5. 1995 Storm Outflow Hydrograph Comparison

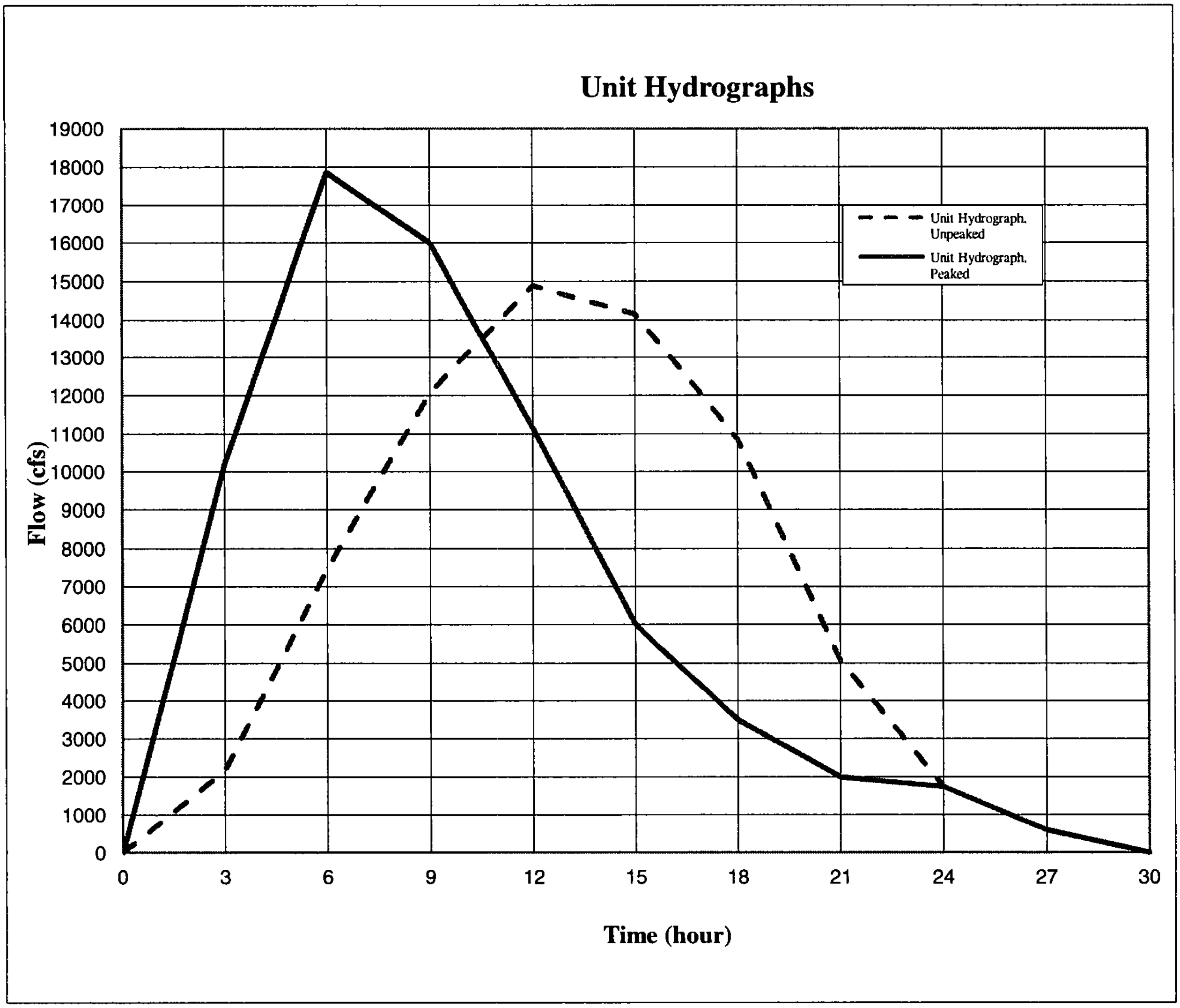


Figure 2.2-6. North Anna Reservoir Watershed Unit Hydrographs (Excluding Reservoir Surface Area)

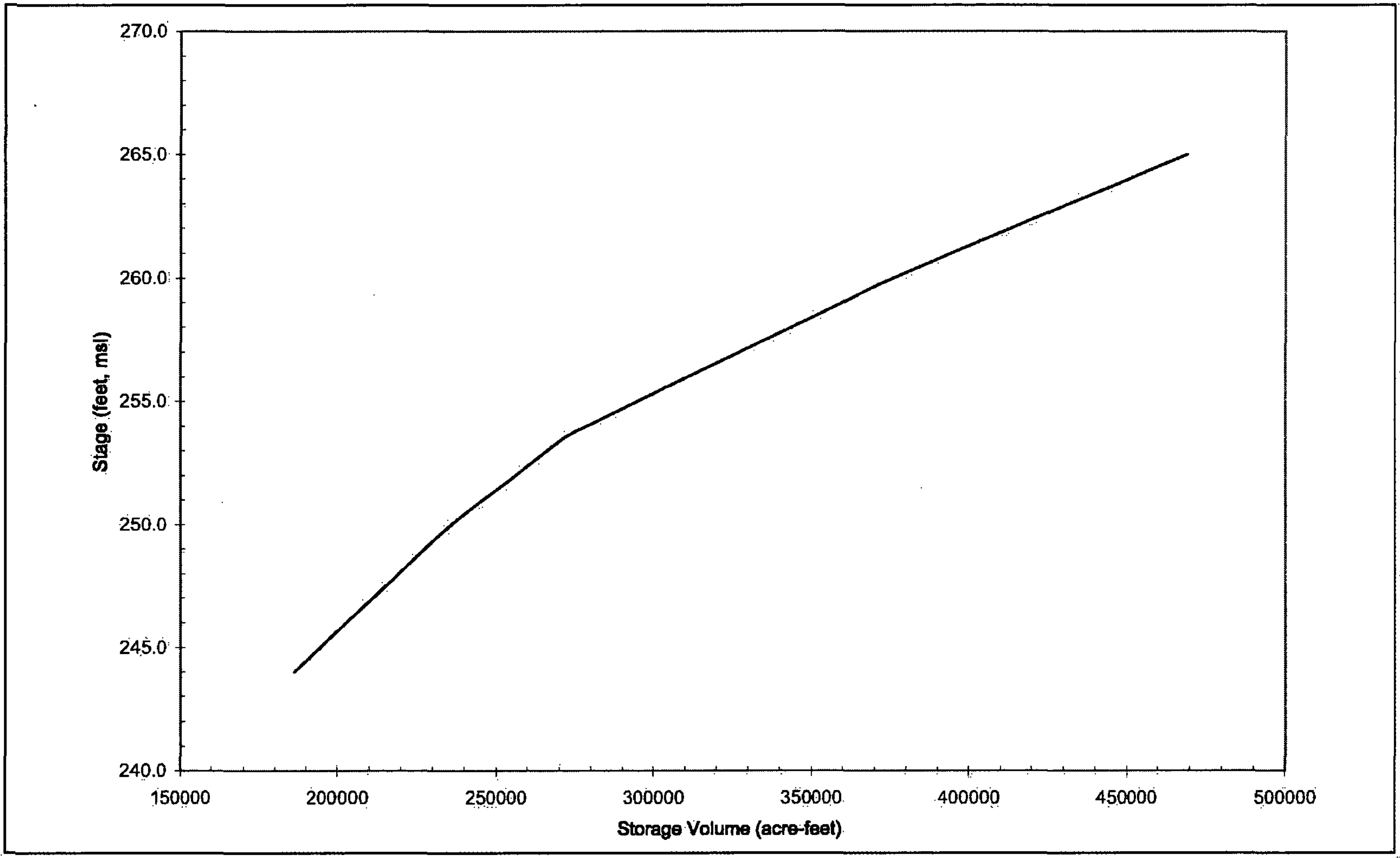


Figure 2.2-7. North Anna Reservoir and WHTF Combined Stage-Storage

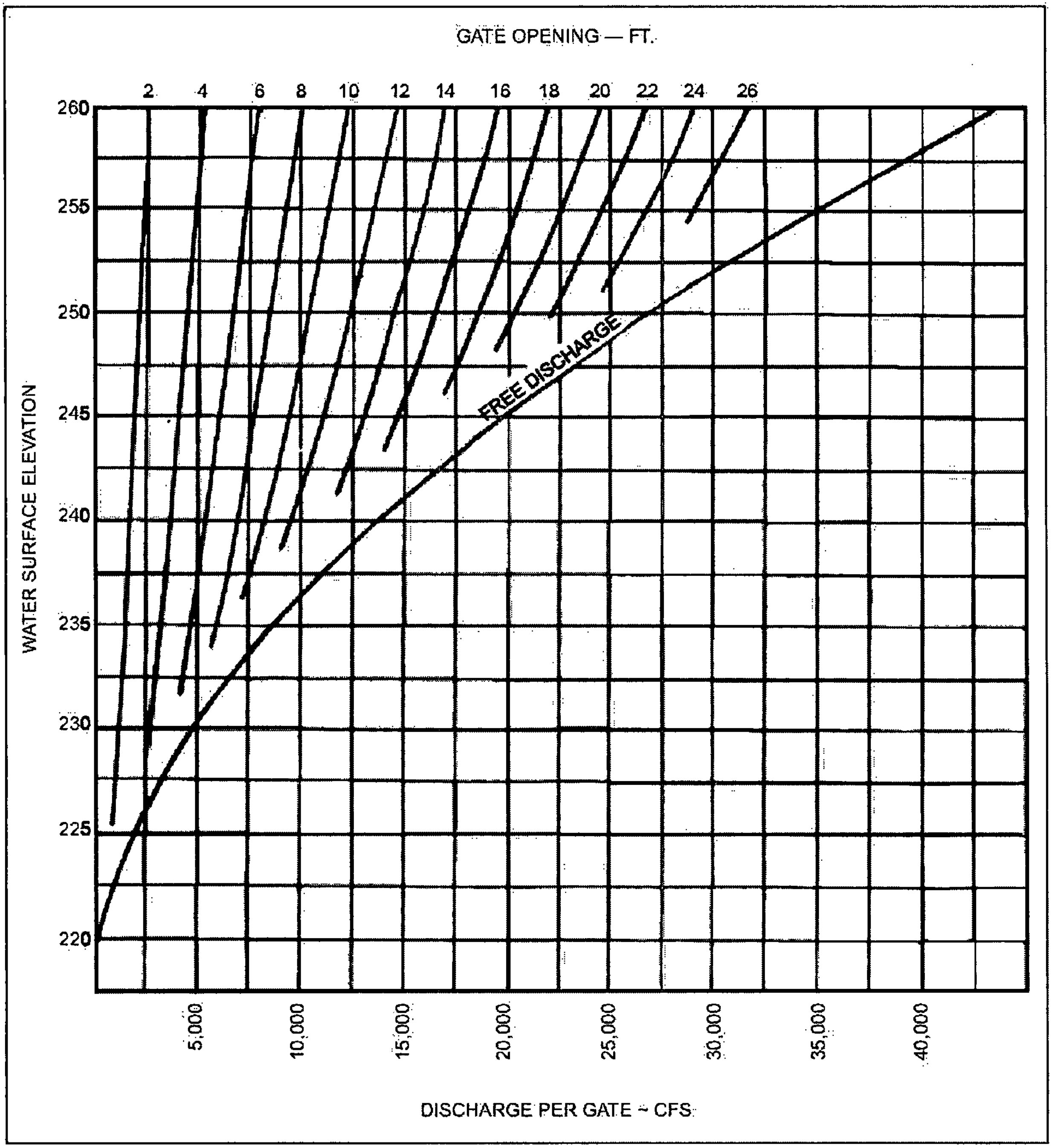
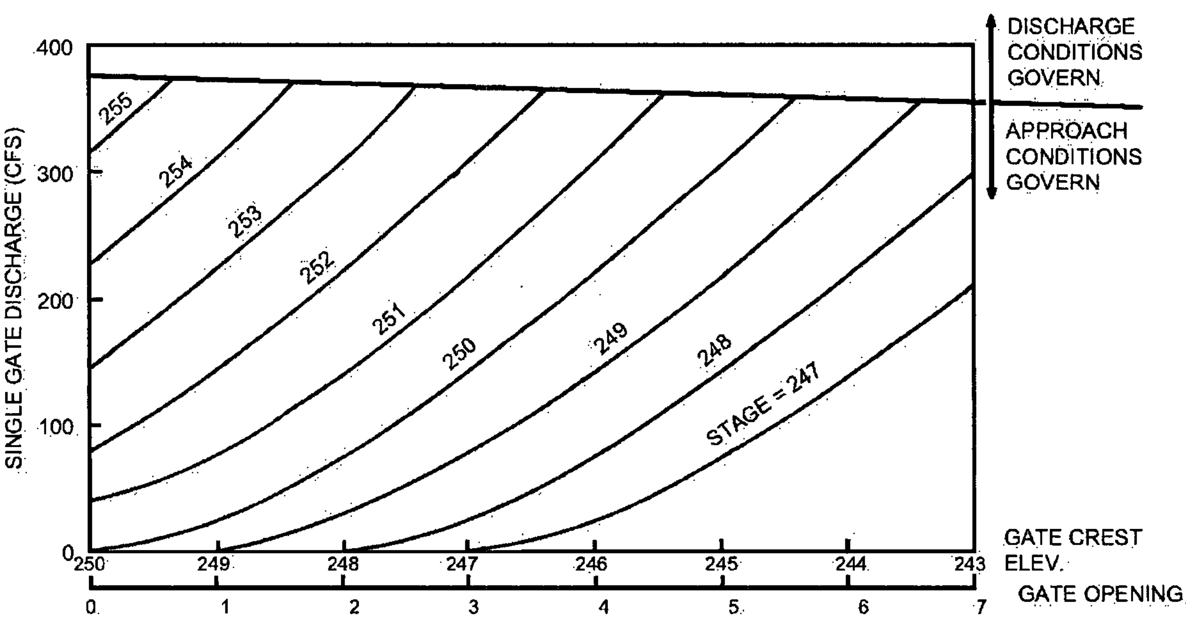


Figure 2.2-8. Spillway and Discharge Capacity (One Gate of Three) North Anna Dam



NOTE:
7 FT. IS MAXIMUM GATE OPENING

Figure 2.2-9. Skimmer Gate Discharge Capacity North Anna Dam

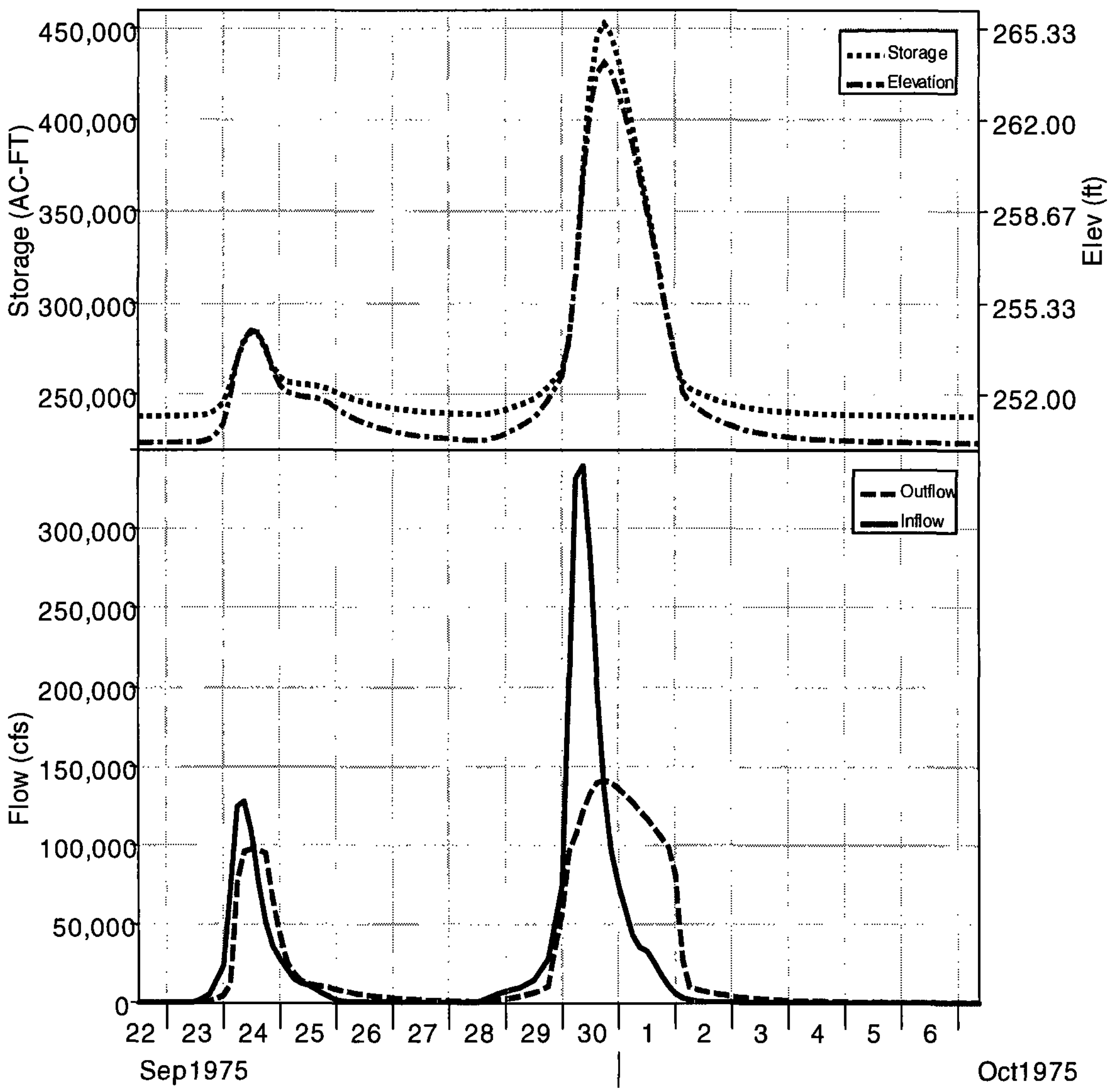


Figure 2.2-10. Lake Anna PMF HEC-HMS Results with Peaked Unit Hydrograph

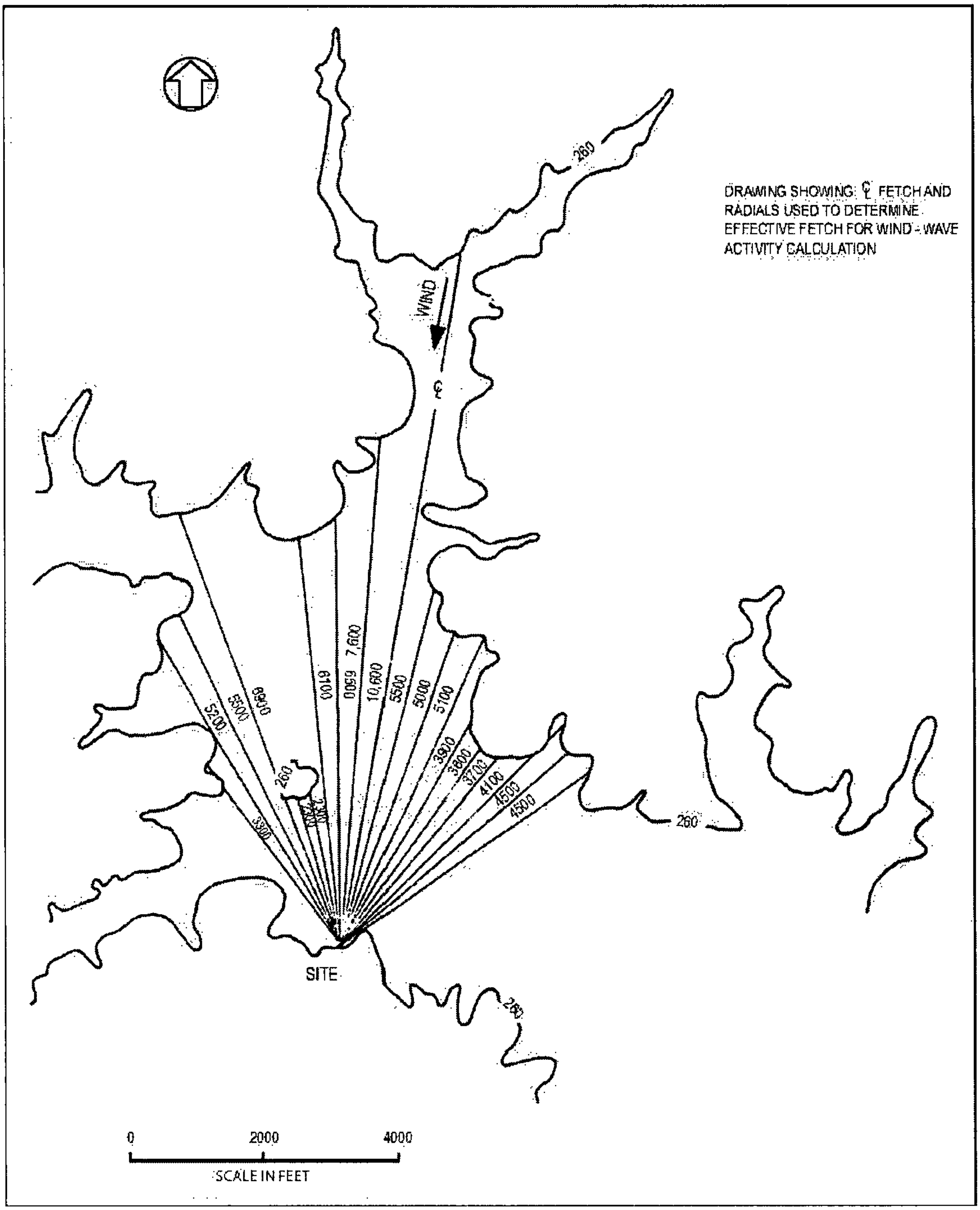


Figure 2.2-11. North Anna Site – Fetch Diagram

2.3 Dam Breaches and Failures

2.3.1 Introduction

This section describes dams near North Anna Units 1 & 2 and the impacts to the site due to dam breaches and failures, which has been evaluated previously in the Early Site Permit (ESP) application for a future plant adjacent to Units 1 & 2 (Reference 2.3-1).

2.3.2 Upstream Dam Failures

Two impounded bodies of water of significant volume were identified upstream of the site, Lake Louisa and Lake Orange. Lake Louisa lies on Hickory Creek, a tributary to the North Anna River, about 3.4 miles upstream of Lake Anna, and has an impounded volume 4713.0 acre-ft (Reference 2.3-2, RAI 2.4.4-1, page 48). Lake Orange is located on Clear Creek, also a tributary to the North Anna River, 8.8 miles upstream of Lake Anna and contains an impounded volume of 2958.0 acre-ft (Reference 2.3-2, RAI 2.4.4-1, page 48). A failure of these dams would be most critical during the probable maximum flood (PMF) event. As described in Section 2.2, the still-water elevation of Lake Anna during the PMF event is 264.3 ft NGVD 29 at the site. The surface area of the lake (including the Waste Heat Treatment Facility (WHTF)) is approximately 19,400 acres at this water level (Reference 2.3-3, Figure 2A-2). Adding the impounded water in Lake Orange and Lake Louisa, a total of 7671 acre-ft, to Lake Anna under these conditions would cause a small increase of approximately 0.4 ft above the PMF level due the large surface area of Lake Anna (7671 acre-ft / 19400 acres = 0.4 ft), resulting in a still-water level of 264.7ft NGVD 29 at the site. This postulation conservatively assumes that no water flows over the North Anna Dam during this event and that the flood waves arrive simultaneously and unmitigated by the intervening terrain. The volume of the upstream impoundments, therefore, is insufficient to cause any dramatic rise in the water levels of Lake Anna. Because the plant grade elevation is 271 ft NGVD 29, there is no credible risk of flooding or other impacts to the safety-related functions of the plant due to the failures of these dams.

2.3.3 Downstream Dam Failures

The North Anna Dam, located downstream of the Units 1 & 2 site, impounds Lake Anna which provides a source of circulating cooling water for Units 1 & 2. However, a downstream failure of the North Anna Dam would not compromise the safety-related water supply for Units 1 & 2, as Lake Anna is not used as a primary source of safety-related cooling water. The primary source of safety cooling water is the Service Water Reservoir, which is designed to contain at least 30 days of inventory without makeup from Lake Anna (Reference 2.3-3, Section 9.2.1.2.2). Therefore, a failure of the downstream dam would not compromise the safety-related functions of the plant.

2.3.4 Service Water Reservoir

The Service Water Reservoir is a safety-related facility located approximately 500 ft south of the station site area and partially enclosed by an earthen impounding dike (Reference 2.3-3). The dike is a Seismic Class I structure, and has been evaluated to preclude overtopping, piping, slide and other sources of failures (Reference 2.3-3, Section 3.8.4). For added conservatism, the slopes and crest of the dike are protected by an outer rock shell that offers erosion protection in the unlikely event of overtopping during a major storm (Reference 2.3-3, Section 3.8.4.7.2). Further conservatism for flood protection of the station is provided by an emergency dike and intercepting channel on the south side of the station. A hypothetical dike failure event at the Service Water Reservoir would have to occur in the vicinity of the pump house to have any potential impact on the safety facilities of Units 1 & 2. During this highly unlikely event, the emergency dike and intercepting trench to the north would divert the flood wave from the breach and would conduct the flow east toward the Waste Heat Treatment Facility (WHTF) (Reference 2.3-3, Section 3.8.4.7.5). This dike is about 10 ft above the bottom of the trench. The flood flow path would cross an access road near the end of the emergency dike and continue on towards the discharge canal and the WHTF, which would not have any adverse impact on the safety facilities of the station. Therefore, a failure of the Service Water Reservoir, though unlikely, will not impact the safe functioning of Units 1 & 2.

2.3.5 Conclusion

Due to the position and size of impounded bodies of water upstream of Units 1 & 2, there is no risk of flooding or other impacts to safety-related functionality of the plant due to a breach or failure of these dams. Similarly, there is no credible safety related impact to the site due to a failure of the North Anna Dam or the Service Water Reservoir.

References

2.3-1 North Anna Early Site Permit Application, Site Safety Analysis Report, Revision 9, Dominion, September 2006.

2.3-2 North Anna Early Site Permit Application Response to Request for Additional Information Letter No. 4, Serial No. 04-318, Docket No. 52-008, Dominion.

2.3-3 Updated Final Safety Analysis Report, Rev. 46.06, North Anna Power Station, Virginia Power, Updated Online 05/12/2011.

2.4 Storm Surge

The NAPS site is not located on an estuary or open coast. Therefore, surge flooding would not produce maximum water levels at the site. The maximum surge flooding is to be considered using an antecedent water level corresponding to the 100-year maximum water level in the lake (Reference 2.4-1). The published Flood Insurance Study for Louisa County, Virginia, indicates only an approximate flood hazard area designation for Lake Anna (Reference 2.4-2). From the flood hazard shading, an approximate flood elevation of 255 ft NGVD 29 was estimated. This elevation is 9.10 ft below the maximum still-water elevation of 264.10 ft NGVD 29, as presented in Section 2.2.

Section 2.2 describes the analysis of wind setup (surge) and wave runup completed as part of the PMF evaluation. This analysis indicates that the maximum fetch length at the site is 10,600 ft, and the effective fetch length is 4700 ft. Given these relatively short lengths, the surges and waves produced from winds generated in a probable maximum hurricane or from the oscillatory waves generated by lake reflection or harbor resonance would not be sufficient to produce water levels greater than the still water level resulting from the PMP over the watershed.

References

2.4-1 Regulatory Guide 1.70, Standard form and Content of Safety Analysis Reports for Nuclear Power Plants, LWR Edition, Revision 3, office of Standards Development, U.S. Nuclear Regulatory Commission, November 1978.

2.4-2 Flood Insurance Rate Map, Louisa County, VA and Incorporated Areas, Federal Emergency Management Agency, U.S. Department of Interior, November 1997.

2.5 Seiche

A seiche is a standing wave oscillation in closed or semi-enclosed bodies of water, which can be triggered by various forcing mechanisms, primarily wind and moving pressure systems, tides, seismic activities, and in some cases, landslides. The seiche motion and associated flooding impact can amplify significantly as the result of a resonance effect if the excitation period of the external forcing event is close to the natural period of the water body. However, seiche-induced flooding is not expected to affect the safety-related structures, systems, and components of North Anna Power Station (NAPS) Units 1 & 2 because of the following:

- No occurrence of any seiches on Lake Anna has been reported by plant personnel (Reference 2.5-1).
- Literature review, including the paper by Lockridge et al. (Reference 2.5-2), does not identify any seiche activity, either seismically or otherwise induced, in the Commonwealth of Virginia, including Lake Anna (Reference 2.5-1). The paper was published in the *Science of Tsunami Hazards*, the International Journal of the Tsunami Society, which lists all known reports of tsunamis or tsunami-like waves (including seiches) that have occurred in the eastern United States since 1600.
- Some forcing mechanisms, such as tides and landslides, can be precluded with respect to NAPS Units 1 & 2 due to the physical, hydrological, geological, and topographical conditions of Lake Anna and nearby region as described in Subsection 2.5.1.
- Seiche forcing mechanisms relevant to Lake Anna, such as atmospheric and seismic events, have characteristic periods that are significantly different from the natural period of the lake, thus eliminating the potential for in-phase amplification of the seiche motion as demonstrated in Subsection 2.5.2.

2.5.1 Tides and Landslide Potentials

Tidal oscillations in a small body of water such as Lake Anna are negligible and are therefore not considered to have the potential to generate any perceivable flooding impact, seiche induced or otherwise, to NAPS Units 1 & 2.

Landslide hazards for the NAPS area have been investigated, as described in the Response to Question 2.4.2-2 of the Request for Addition Information Letter No. 4 (Reference 2.5-1) on the North Anna Early Site Permit Application for a future plant, to assess the potential for landslide-induced flooding. The methodology for the assessment includes: field reconnaissance, air-photo interpretation, literature search for available information on landslides, review of existing literature, and discussions with researchers from the U.S. Geological Survey and universities who are familiar with the site region.

As indicated in Reference 2.5-1, large, deep-seated landslides are not present in the site area or along the shores of Lake Anna. The gently rolling topography prevalent in the Piedmont region of Lake Anna generally is not susceptible to deep-seated landslides or to extensive debris flows. Metamorphic bedrock in the site area is deeply weathered to a saprolitic soil. The saprolite erodes primarily by sheetwash and downslope colluvial transport and locally by stream and gully incision. This type of erosion leads to the development of gently rolling topography. There are also no published maps of landslides in the Lake Anna area, similar to other parts of Virginia, primarily because landslides are not prevalent in the region. Based on field reconnaissance and air-photo interpretation, there are no observed landslides in the Lake Anna region other than sparse minor debris flows, soil slips, and rock falls. Evaluation of pre-Lake Anna photography also shows that there are no large over-steepened slopes submerged beneath the lake. Given the absence of observed landslides and the gently rolling topography, it is determined that there is no potential for large, deep-seated landslides or debris flows to produce a seiche within Lake Anna (Reference 2.5-1).

2.5.2 Natural Resonant Period of Lake Anna

The primary natural period of the lake at the NAPS site is estimated to be on the order of 9.5 to 11.5 minutes, by approximating the lake as a rectangular basin with a constant depth. The formulation for the natural period, T, in seconds, of the first mode of oscillations for a simple rectangular basin is:

$$T = \frac{2L}{\sqrt{gd}}$$

where g is the gravitational acceleration, in ft/s^2, and L and d are the length and depth of the basin, in ft, respectively (Reference 2.5-3).

The length, L, of the basin is represented by the longest fetch to the North Anna site, estimated to be about 10,600 ft as shown in Figure 2.5-1 (Reference 2.5-4, Figure 2.4-9). The longest fetch is selected for the evaluation as it would result in the largest wind setup and in turn lead to the most dominant seiching motions. At the peak still lake level of 264.2 ft NGVD 29 (or mean sea level) from a probable maximum flood (PMF) event as documented in the UFSAR, the average water depth is 43.2 ft over the fetch length (Reference 2.5-4, Section 2A.2.7), resulting in a natural period of about 9.5 minutes for the lake. Section 2.2 of this report states that the reevaluated PMF still lake level is slightly higher at 264.42 ft NGVD 29, i.e., a small difference of about a 0.2-ft increase over the corresponding UFSAR level. The new PMF still lake level may affect the fetch length and the average depth slightly, but these potential changes will not have an appreciable impact on the lake's natural period estimate.

Similarly, using the same fetch of 10,600 ft as the basin length and lowering the average water depth of the basin to approximately 29 ft to represent the lake at its normal operating level of 250.0 ft NGVD 29, the natural period of the first seiche mode at the NAPS site would be about 11.5 minutes. When the future Unit 3 begins operation, the normal lake level will increase 3 inches to 250.25 ft NGVD 29. This small increase has a minor effect on the natural period estimate of the lake.

In order for the seiche to resonate with the first mode of the lake's natural period, external forcing mechanisms would need to have a period on the same order of 9.5 to 11.5 minutes. The higher-mode seiches are less critical as they dissipate faster than the first mode as the energy dissipation rate increases with the decreasing natural period of the water body.

2.5.3 Atmospheric and Seismic Forcing Mechanisms

Atmospheric forcing events such as cyclical wind and moving pressure systems typically have a time scale on the order of hours, which is significantly higher than the lake's natural period estimates of 9.5 to 11.5 minutes. Also, wind-induced waves have periods on the order of a few seconds, which is much smaller than the lake's natural period. Therefore, it is highly unlikely for any atmospheric forcing to be capable of amplifying seiche motions in Lake Anna to cause flooding concerns at NAPS Units 1 & 2 safety-related facilities.

As described in Section 2.5.1, seismic-induced seiches have not been reported at Lake Anna, either from plant personnel or from available literature. In addition, typical seismic wave periods are on the order of seconds, which make it unlikely for any seiche to be amplified from seismic motions because of the significant difference between the forcing period and the lake's natural period.

2.5.4 Conclusion

Based on the absence of evidence for seiche motions in Lake Anna from historical observations and literature reviews, the screening out of some of the forcing mechanisms based on the evaluation and interpretation of Lake Anna's physical, hydrological, geological, and topographical environment as well as the comparison of Lake Anna's natural period with the periods of the relevant forcing events, it is concluded that resonant seiche motion in Lake Anna is highly unlikely. Thus, there is no risk of seiche-related flooding to the safety-related functions of NAPS Units 1 & 2.

References

2.5-1 Response to Request for Additional Information Letter No. 4, North Anna Early Site Permit Application, Dominion Nuclear North Anna, LLC, Serial No. 04-318, Docket No. 52-008, August 2, 2004.

2.5-2 Lockridge, P.A., Whiteside, L.S., Lander, J.F., Tsunamis and Tsunami-Like Waves of the Eastern United States, Science of Tsunami Hazards, Vol. 20, No. 3, p. 120, 2002.

2.5-3 Dean, R.G., and Dalrymple, R. A., Water Wave Mechanics for Engineers and Scientists, World Scientific Publishing Co. Pte. Ltd., 353 pp., 1991.

2.5-4 Updated Final Safety Analysis Report, Rev. 46.06, North Anna Power Station, Virginia Power, Updated online May 12, 2011.

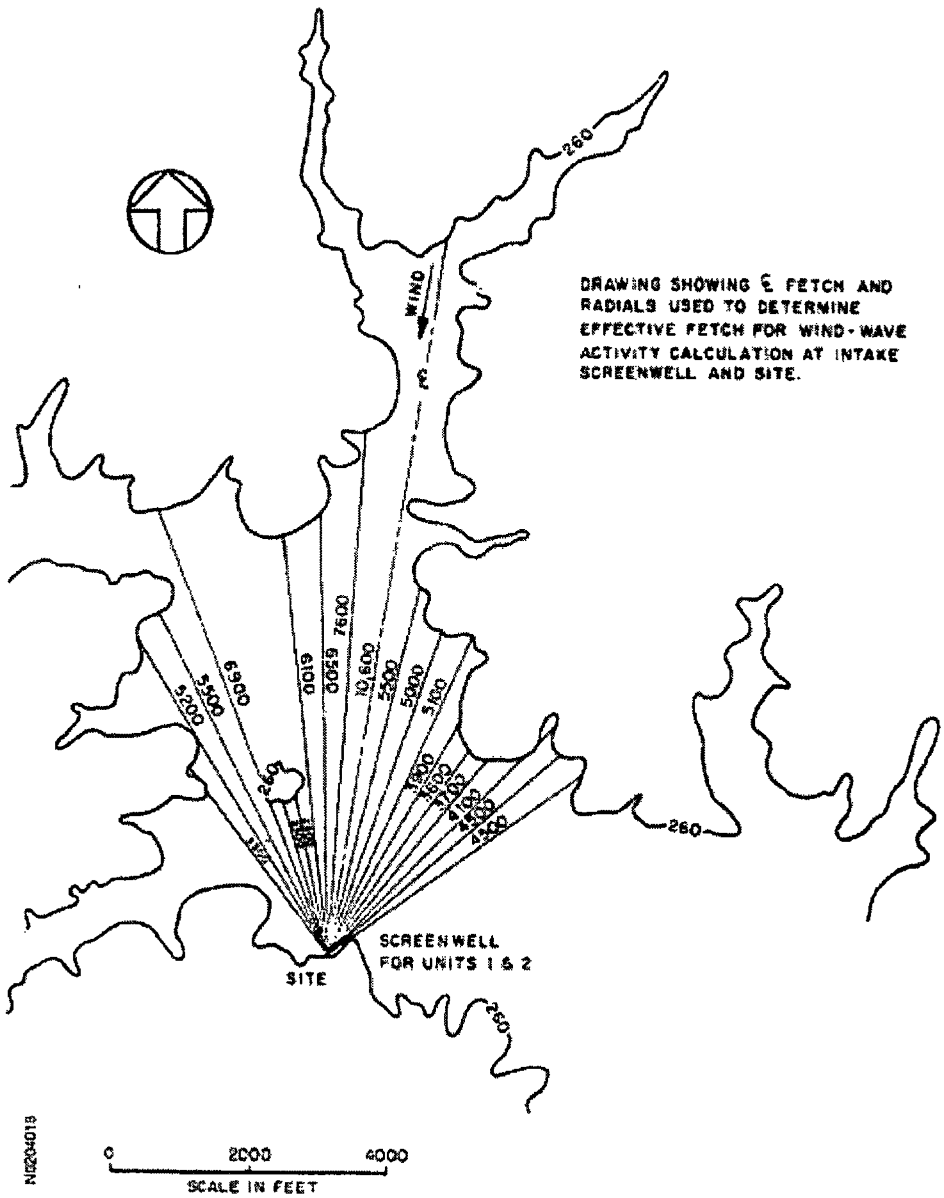


Figure 2.5-1. Fetch at NAPS Units 1 & 2 Site (Reference 2.5-4, Figure 2.4-9)

2.6 Tsunami

The NAPS site is at an inland location and not located on an estuary or open coast. Therefore, tsunami flooding is not a credible flooding source.

2.7 Ice Induced Flooding

2.7.1 Introduction

Ice can cause flooding at a site by the formation of ice jams in the vicinity of the site. Ice jams upstream of the site can impound water and subsequently release it, causing a flood wave similar to that of a dam failure. Formation of ice jams downstream of a site can lead to flooding as a result of backwater effects. Ice formation can also cause blockage of drains and compromise the drainage systems of the site. The potential flooding impact on the safety facilities of Units 1 & 2 due to blockage of the drainages from ice and other factors such as debris during extreme storm events is evaluated in Section 2.1, Local Intense Precipitation.

2.7.2 Potential for Ice Jam Formation

2.7.2.1 Air Temperature

Prolonged periods of sub-freezing temperatures promote ice formation in waterways that can lead to ice jam formation. As such, regional air temperature data from 1939 to 2012 were examined to determine if extended periods of severely cold temperatures occurred for this area.

Records from Richmond International Airport (Reference 2.7-1) showed multiple periods of severely cold temperatures. For example, in 1940 the month of January had 24 days with mean temperatures (estimated as the average of the maximum and minimum daily temperatures) below freezing, and 10 days with mean temperatures below 20° F, six of them consecutive. This period of cold weather preceded two recorded ice jam events in Virginia in early February, 1940 (Reference 2.7-2). Therefore, there is evidence of cold weather periods conducive to ice jam formation in the region.

2.7.2.2 Historical Ice Jam Records

The Ice Jam Database maintained by the Army Corps of Engineers (Reference 2.7-2) was queried for historical records of ice jam incidents in the vicinity of the site. The query focused on the area defined by USGS Hydrological Unit Code (HUC) 0208, which contains catchments in Virginia including the North Anna, James, and Rappahannock Rivers, as well as the site and all tributaries to Lake Anna.

The database contains 19 records of ice jam events in HUC 0208, summarized in Table 2.7-1 (Reference 2.7-2). Six of these records described increased gauge height due to backwater, and 11 described low flow conditions due to freezing. Two records reported more notable effects. The ice jam recorded on Feburary 11, 1936, near Richmond, VA on the James River produced high water levels and breakups that released ice and water that damaged and destroyed vessels on the river (Reference 2.7-2). The second notable ice jam was recorded on December 30, 1998 on the Rappahannock River near Fredericksburg, VA. The recorded ice jam, described as 4-6 ft thick, produced high water levels but minimal flooding (Reference 2.7-2).

While none of the recorded ice jams were in the immediate vicinity of the site, one ice jam was recorded on the North Anna River near Doswell, VA, in 1934 (Reference 2.7-2). This suggests that there is a potential for ice jam formation in the area of Units 1 & 2.

2.7.3 Potential for Ice Induced Flooding at the Units 1 & 2 Site

The potential for flooding due to upstream ice jams was evaluated in the Response to DSER Open Item 2.4-4 as part of the NAPS Unit 3 Early Site Permit (ESP) application (References 2.7-3 and 2.7-5). This analysis hypothesized an ice jam on the North Anna River upstream of the site measuring 10 ft in height with an impounded volume of water estimated to be 1500 acre-ft. The height of the dam was based on the recorded height of the 1989 Rappahannock River ice jam, 4-6 ft (Reference 2.7-2), which is the tallest on record for the area. The volume was calculated by estimating the surface area of the impoundment created by the ice dam based on the topography, 150 acres, and conservatively assuming a water depth of 10 ft throughout (Reference 2.7-5). The effects on Lake Anna's water level as a result of the failure of such an ice dam are bounded by the analysis presented in Section 2.3, which considered the failure of two upstream impoundments with a combined volume of 7671 acre-ft (Reference 2.7-4). It was estimated that the simultaneous failure of these dams would produced a 0.4 ft rise in Lake Anna, which would not threaten the site. The formation of an ice dam with a volume greater than the combined volume of the two upstream impoundments is not credible, given the conservatism of the volume estimate described above and that there is no record in the area of an ice jam taller than 10 ft. Therefore, there is no risk of flooding at Units 1 & 2 due to the breaching of an ice jam formed upstream of the site.

Surface water from the NAPS site area drains to Lake Anna where it is discharged over the dam to the North Anna River downstream. The tailwater elevation in the North Anna River is on the order of 180 ft NGVD 29 (Reference 2.7-7), which is substantially lower than the dam crest elevation of 265 ft NGVD 29 (Reference 2.7-6, Section 2.4.1.2) and the normal operating level of 250 ft NGVD 29 in the lake (Reference 2.7-6, Section 2.4.1.2). As such, there is no risk of flooding at Units 1 & 2, where the grade elevation is at 271 ft NGVD 29, due to backwater effects from downstream ice jams in the North Anna River. In summary, the hydrologic setting at NAPS demonstrates that there is no discernible ice induced flood risk at the Units 1 & 2 site.

2.7.4 Conclusion

A review of historical temperature data shows that periods of low temperatures conducive to ice formation occurred in the region of NAPS. Similarly, ice jams on waterways in the area, though infrequent, were reported in the historical database. The hydrologic conditions of the site, however, preclude any threat of ice induced flooding of the plant's safety-related facilities due to the proximity of Lake Anna, which provides for the storage and discharge of upstream flood waves, and the North Anna Dam which isolates the site from downstream backwater effects.

References

2.7-1 NOAA National Climatic Data Center (NCDC), Available at http://www.ncdc.noaa.gov/cdo-web/, accessed October 11, 2012. Station ID: GHCND:USW00013740; data period: March 1, 1939 to October 3, 2012.

2.7-2 "Ice Jam Database," U.S. Army Corps of Engineers, Cold Region Research and Engineering Laboratory (CRREL), Available at https://rsgisias.crrel.usace.army.mil/apex/f?p=273:2:85695751952301, accessed October 11, 2012.

2.7-3 North Anna Early Site Permit Application, Site Safety Analysis Report, Revision 9, Dominion, September 2006.

2.7-4 North Anna Early Site Permit Application Response to Request for Additional Information No. 4, Serial No. 04-318, Docket No. 52-008, Dominion.

2.7-5 North Anna Early Site Permit Application, Responses to Draft Safety Evaluation Report Open Items. Serial No. 05-785B, Docket No. 52-008, Dominion, March 2, 2005.

2.7-6 Updated Final Safety Analysis Report, Rev. 46.06, North Anna Power Station, Virginia Power, Updated Online 05/12/2011.

2.7-7 Lake Anna East Quadrangle, 7.5-Minute Series [map], 1:24,000, Reston, VA: U.S. Department of the Interior, U.S. Geological Survey, 2010.

2.8 Channel Migration or Diversion

The possibility of an upstream diversion of the North Anna River is considered extremely remote. Historical information indicates that the river has not had a major change of course in recent history (Reference 2.8-1) (Reference 2.8-2). Inspection of US Geological Survey 7.5-minute topographic maps and pre-Lake Anna aerial photography shows that the North Anna River lies in a valley that is at least 250 ft lower than the surrounding drainage divide. There is no apparent man-made or natural event (e.g., earthquake, subsidence, landslide, or ice blockage) that could divert the North Anna River from its current drainage basin. Thus, the flow of water into Lake Anna from the North Anna River and tributaries is secure from unexpected upstream diversions.

References

2.8-1 Updated Final Safety Analysis Report, Rev. 46.06, North Anna Power Station, Virginia Power, Updated Online 05/12/2011.

2.8-2 North Anna Power Station Units 1 and 2, Appendix E – Applicant's Environmental Report, Operating License Renewal Stage, Dominion, May 2001.

2.9 Combined Effect Flood

Combined effects of different flood causing mechanisms are discussed within the individual sections from Section 2.1 through 2.7, where applicable. The combined effects flooding criteria for this reevaluation are based on the guidelines presented in NUREG/CR-7046 (Reference 2.9-1) and ANSI/ANS-2.8-1992 (Reference 2.9-2).

References

2.9-1 Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7046, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, November 2011.

2.9-2 ANSI/ANS-2.8-1992, Determining Design Basis Flooding at Power Reactor Sites, American National Standard, American Nuclear Society, July 1992.

3.0 Comparison of Current and Reevaluated Flood Causing Mechanisms

Table 3.0-1 summarizes the comparison of current and reevaluated flood causing mechanisms, which included wind effect for Flooding in Streams and Rivers.

Local Intense Precipitation

The current NAPS Units 1 & 2 flood elevation from local intense precipitation is the ground level with no accumulation or ponding in the protected area. The ground level in the protected area is generally at elevation 271.0 ft NGVD 29. However, some areas in the corridors between the reactor containment buildings and the service building have ground elevations at 274.3 ft NGVD 29.

The current flood elevation inside the Turbine Building as a result of flooding in the West Basin area is 256.1 ft NGVD 29, which is 0.9 ft below the top of the flood protection wall inside the Turbine Building at elevation 257.0 ft NGVD 29.

A current flood level as a result of the local PMP has not been determined for the NAPS Units 1 & 2 ISFSI area. However, the ISFSI site is at a higher elevation than the surrounding area. The ISFSI site is drained by gradual fine grading away from the storage pads, and a combination of earthen, and concrete channels to the exterior of the site. The drainage system was designed using a 10-year storm rainfall intensity. Storm drainage channels were checked and provided with additional depth to accommodate the 100-year storm. These channels were checked to confirm that adequate hydraulic capacity existed in the storm drain system, and that the hydraulic grade line elevations were less than the top elevations of the storage pads. Since the storage pads are located at the highest part of the ISFSI, severe flooding or hydrodynamic loading due to floodwater is not credible based on the 100-year storm.

The reevaluated flood elevation in the NAPS Units 1 & 2 protected area from local intense precipitation ranges from 271.3 ft to 274.5 ft NGVD 29, with the increase in flood elevation above grade ranging from 0.3 to 2.9 ft. The increases in flood levels are due to consideration that all passive storm drains are clogged in combination with the increased local PMP depths, which were used in the reevaluation analysis as described in Section 2.1.

The reevaluated flood elevation inside the Turbine Building as a result of flooding in the West Basin area is greater than elevation 257.0 ft NGVD 29 and is above the elevation of the Turbine Building flood protection wall. The increased flood level inside the Turbine Building is due to the consideration that the Turbine Building roof drains are clogged and non-operational during the local PMP event.

The reevaluated flood elevation in the ISFSI area is elevation 311.2 ft NGVD 29. The concrete ISFSI pads are at elevation 311.5 feet NGVD 29, which is 0.3 ft above reevaluated flood elevation.

Flooding in Streams and Rivers

The current NAPS Units 1 & 2 flood elevation from probable maximum flooding in streams and rivers is 264.4 ft NGVD 29, or 267.3 ft NGVD 29 with coincident wave run-up. The reevaluated flood elevation is 264.3 ft NGVD 29, or 267.4 ft NGVD 29 with coincidental wave run-up. The flood elevation is 3.6 ft below the grade elevation of safety-related SSCs at 271.0 ft NGVD 29.

Dam Failures

No current flood evaluation for NAPS Units 1 & 2 has been made for dam failures as described in Section 1.2.

The reevaluated flood elevation is 264.7 ft NGVD 29, which is 6.3 ft below the grade elevation of safety-related SSCs at 271.0 ft NGVD 29.

Storm Surge

The NAPS Units 1 & 2 site is not located on an estuary or open coast. Therefore, storm surge is not expected to affect the site. See Section 2.4.

Seiche

The absence of evidence for seiche motions in Lake Anna from historical observations and literature reviews and the screening out of the seiche forcing mechanisms as described in Section 2.5 indicate that seiches are not expected to affect the NAPS Units 1 & 2 site.

Tsunami

The NAPS Units 1 & 2 site is not located on an estuary or open coast. Therefore, tsunami induced flooding is not expected to affect the site. See Section 2.6.

Ice Induced Flooding

Historical temperature data and the hydrologic conditions of the NAPS Units 1 & 2 site presented in Section 2.7 indicate that ice induced flooding is not expected to affect the site.

Channel Diversion

Historical and topographical information presented in Section 2.8 indicate that channel diversions are not expected to affect the site.

Associated Flooding Hazards

In addition to flood inundation levels, additional hazards are presented as a result of flooding, which include hydrodynamic/hydrostatic forces, water-borne missiles, debris and sediment impact. The only source of flooding which impacts the NAPS Units 1 & 2 site is local intense precipitation. Flooding inundation from this source does not produce high velocity flows which carry debris or strong hydrodynamic forces. Thus, these are not considered hazards applicable to this site. Additionally, there is no run-on flooding from offsite areas and the entire drainage area is impervious. Thus, sediment impact would be minor and also not considered a hazard applicable to this site. The increase in hydrodynamic and hydrostatic forces due to the local PMP flooding level will be evaluated in the development of interim and permanent flood protection measures or mitigation actions provided to protect the plant's safety functions during the local PMP event.

Table 3.0-1. Current and Reevaluation Flood Elevations

Flood Causing Mechanism	Current Design Basis Flood Elevation	Reevaluation Flood Elevation	Difference[A]
Local Intense Precipitation (Protected Area)	Ground level with no accumulation	Ranges from 271.3 ft to 274.5 ft NGVD 29	Ranges from 0.3 ft to 2.9 ft
Local Intense Precipitation (ISFSI Area)	No impact identified	311.2 ft NGVD 29	N/A
Local Intense Precipitation (West Basin Area)	256.1 ft NGVD 29	>257.0 ft NGVD 29[B]	> 0.9 ft[B]
Flooding in Streams and Rivers	267.3 ft NGVD 29	267.4 ft NGVD 29	0.1 ft
Dam Failures	No impact identified	264.7 ft NGVD 29	N/A
Storm Surge	No impact identified	No impact identified	N/A
Seiche	No impact identified	No impact identified	N/A
Tsunami	No impact identified	No impact identified	N/A
Ice Induced Flooding	No impact identified	No impact identified	N/A
Channel Diversion	No impact identified	No impact identified	N/A

Notes: [A] Difference = Reevaluated flood elevation – Current design basis flood elevation
[B] Storage volume in Turbine Building above elevation 257.0 ft NGVD has not been determined and thus, flood elevations are not computed