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DUKE ENERGY®

APR 0 1 2015

10 CFR 50.54(f)

Serial: HNP-15-027

U.S. Nuclear Regulatory Commission
Attn: Document Control Desk
11555 Rockville Pike
Rockville, MD 20852

Shearon Harris Nuclear Power Plant, Unit 1
Docket No. 50-400
Renewed License No. NPF-63

Subject: **Flood Hazard Reevaluation Report, Revision 1**

Reference:

1. NRC Letter to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, *Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident*, dated March 12, 2012, ADAMS Accession No. ML12053A340
2. NRC Letter to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, *Prioritization of Response Due Dates for Request For Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Flooding Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident,* dated May 11, 2012, ADAMS Accession No. ML12097A509
3. Letter from Sean O'Connor to U.S. Nuclear Regulatory Commission (NRC), *Flooding Hazard Reevaluation Report*, dated March 12, 2013, ADAMS Accession No. ML13079A253

Ladies and Gentlemen:

By letter dated March 12, 2012, (Reference 1) the Nuclear Regulatory Commission (NRC) issued a Request For Information (RFI) pursuant to Title 10 of the Code of Federal Regulations 50.54(f). Enclosure 2 of the RFI addresses Near-Term Task Force (NTTF) Recommendation 2.1, Flooding. Required Response 2 requires licensees to prepare and submit a Hazard Reevaluation Report in accordance with the priorities specified in Reference 2. Reference 2 placed the Shearon Harris Nuclear Power Plant, Unit 1, in Category 1, requiring a response by March 12, 2013. On March 12, 2013, the Flooding Hazard Reevaluation Report (FHRR) for Shearon Harris Nuclear Power Plant, Unit 1 (HNP), was submitted (Reference 3). It was determined later that the previous submittal of the FHRR contained a number of administrative errors which required clarification. Specifically, the phrases "licensing basis" and "Current Licensing Basis" were used interchangeably with the phrase "design basis."

Enclosure 1 to this letter contains a table which outlines the changes made in Revision 1 of the FHRR. Enclosure 2 to this letter contains Revision 1 of the FHRR.

If you have any questions or require additional information, please contact Dave Corlett, Manager, Regulatory Affairs, at 919-362-3137.

I declare under the penalty of perjury that the foregoing is true and correct. Executed on
APR 01 2015.

Sincerely,

Benjamin C. Waldrep

Enclosure 1: Table of Changes to Flood Hazard Reevaluation Report

Enclosure 2: Flood Hazard Reevaluation Report, Revision 1

cc: Mr. J. D. Austin, NRC Sr. Resident Inspector, HNP
Ms. M. Barillas, NRC Project Manager, HNP
Mr. V. M. McCree, NRC Regional Administrator, Region II

ENCLOSURE 1

TABLE OF CHANGES TO FLOOD HAZARD REEVALUATION REPORT

SHEARON HARRIS NUCLEAR POWER PLANT, UNIT NO. 1

DOCKET NO. 50-400

Table of Changes to Flood Hazard Reevaluation Report

Section	Change	Paragraph	Line
1.0	Current Licensing Basis (CLB) to design basis (DB)	2	2
1.0	CLB to DB	2	9
2.1	Current Design Basis to Design Basis	Heading	n/a
2.1.4	CLB to DB	1	1
2.1.5	CLB to DB	1	1
2.1.6	CLB to DB	1	1
2.1.7	CLB to DB	1	1
2.2	Current Design Basis to Design Basis	Heading	n/a
2.3	Licensing Basis to Design Basis	Heading	n/a
2.3	licensing basis to design basis	2	2
3.1.3	CLB to DB	2	1
3.1.3	CLB to DB	2	2
3.1.3	CLB to DB	2	3
3.1.3	CLB to DB	2	5
3.1.3	CLB to DB	2	10
4.0	Current Design Basis to Design Basis	Heading	n/a
4.0	CLB to DB	1	1
4.1	CLB to DB	1	1
4.1	CLB to DB	1	4
4.2	CLB to DB	1	1
4.2	CLB to DB	1	4

Section	Change	Paragraph	Line
4.3	CLB to DB	1	1
4.3	CLB to DB	1	2
4.3	CLB to DB	1	5
4.8	CLB to DB	1	1
4.8	CLB to DB	1	4
4.9	CLB to DB	1	1
5.0	CLB to DB	1	2
5.1	CLB to DB	1	1
5.1	CLB to DB (2 instances)	2	1
5.2	CLB to DB	1	1
5.2	CLB to DB	2	1
5.2	CLB to DB	3	1
5.3	CLB to DB	1	1
5.4	CLB to DB	1	1
5.4	CLB to DB	2	1
Table 3	CLB to DB (2 instances)	Table heading	n/a
Table 3	CLB to DB (2 instances)	notes	n/a
Table 6	CLB to DB	title	n/a
Table 6	CLB to DB	Table heading	n/a

ENCLOSURE 2

FLOOD HAZARD REEVALUATION REPORT, REVISION 1

SHEARON HARRIS NUCLEAR POWER PLANT, UNIT NO. 1

DOCKET NO. 50-400

Table of Contents

1.0 Executive Summary

This report summarizes the results of the flood hazard reevaluations performed at Shearon Harris Nuclear Power Plant, Unit No. 1 (HNP) in response to the March 12, 2012 NRC 50.54(f) Request for Information, Item 2.1. The flood hazard reevaluation was completed using current regulatory guidance and methodologies used for early site permits and combined license applications. For each flood hazard, the reevaluated flood elevations were compared to the design basis flood hazard level to determine whether it was bounded.

There were several instances of higher elevations in the reevaluated flood hazards versus the design basis (DB). Therefore, an Integrated Assessment will be completed and report submitted to the NRC on or before March 12, 2015. The integrated assessment will be performed to address higher reevaluated flood hazards for the Diesel Fuel Oil Storage Building and Waste Processing Building (WPB) during a local intense precipitation event; the Main Dam, Auxiliary Dam, and plant site, during a probable maximum flooding event; the Auxiliary Dam during a Probable Maximum Hurricane (PMH) induced storm surge event; and the Main Dam and Auxiliary Dam during a combined effect event. The Main Dam, Auxiliary Dam, and plant site are protected under the reevaluated flood hazards; however, consistent with the DB, areas in the Waste Processing Building were determined to be susceptible to flooding during a local intense precipitation event with no impact on safety-related equipment. Therefore, no interim actions are planned.

2.0 Site Information

HNP lies within the floodplain of Buckhorn Creek in Wake and Chatham Counties of North Carolina. The Main Dam constructed on Buckhorn Creek, approximately 2.5 miles north of its confluence with the Cape Fear River, created the 4,000 acre Main Reservoir. The Auxiliary Dam created the smaller 317 acre Auxiliary Reservoir. Each dam is equipped with an uncontrolled spillway. The plant island is bounded by the Main Reservoir on the east, south, and southwest sides and by the Auxiliary Reservoir on the west and northwest sides. The Auxiliary Reservoir is the preferred source of Emergency Service Water (ESW). The Main Reservoir is the cooling tower makeup water source and secondary source of ESW.

Values for the flood level in the original design specifications were slightly different than those presented in the HNP Final Safety Analysis Report (FSAR) due to the fact that the original design specifications were based on the assumption that the water level of the Main Reservoir would be raised to meet the needs of four units. However, the additional units were never built and the water level of the Main Reservoir was not raised to this assumed height.

2.1 Design Basis Flood Hazard

2.1.1 Local Intense Precipitation

The site is subject to local intense precipitation. The local Probable Maximum Precipitation (PMP) was calculated using U.S. Weather Bureau Hydro-meteorological Report Nos. 51 and 52 (HMR-51 and HMR-52). The maximum incremental PMP depth at the site is 18.8 inches (in). Since losses in unpaved areas were not considered during the PMP event, the HNP site active drainage capacity to drain four inches of runoff from the site was credited in this calculation. As a result, the accumulated flood depth during the considered PMP event is approximately 14.8 in. Dynamic effect of wind on flood water accumulated on the plant grade was assumed to be insignificant and not considered. If all drainage inlets and pipes were to become blocked during a PMP event, the maximum elevation to which flood water will pond on site is 261.27 feet (ft) Mean Sea Level (MSL).

2.1.2 Flooding in Streams and Rivers

The Probable Maximum Flood (PMF) on streams and rivers and a designed wind wave activity in the reservoirs was considered as a flood hazard. The PMF was estimated using hypothetical flood characteristics considered to be the most severe reasonably possible at a particular location based on comprehensive hydrometeorological analysis of critical runoff-producing precipitation and hydrologic factors favorable for maximum flood runoff. The PMP was applied to the unit hydrograph with the appropriate infiltration losses to develop the flood hydrograph for each sub-basin, as well as for the Buckhorn Creek drainage basin. An antecedent precipitation with an intensity of one-half of the PMP was also applied to the unit hydrograph with appropriate infiltration losses to develop the flood hydrograph for each sub-basin in order to have a more conservative estimate of the PMF still water level in the Main and Auxiliary Reservoirs.

The total inflow to the Main Reservoir was the summation of the outflow from all sub-basins located above the Main Dam. After obtaining the inflow hydrograph, the PMF was then routed through the reservoirs to estimate the PMF still water level in the reservoirs. The PMP depths were calculated using Hydrometeorological Report No. 33 of the National Oceanic and Atmospheric Administration (NOAA). The PMF hydrographs were computed based on the one-hour incremental PMP Hyetograph derived from a six-hour rainfall distribution curve applicable to the North Carolina region. A storm duration of 36 hours was used for the PMP. The coincident wind wave activities for the PMF were determined in accordance with the procedures and methods presented in the U.S. Army Corps of Engineers Engineering Technical Letter (ETL) 1110-2-221 and in the Shore Protection Manual.

With the known PMF flow hydrographs over the spillways of both reservoirs, the PMF stillwater levels in the reservoirs were determined by the corresponding spillway rating curves. For the most severe cases analyzed, the PMF flow rate over the Auxiliary Reservoir Spillway is 5,030 cubic feet per second (cfs), and the corresponding water level elevation in the Auxiliary Reservoir is 256 ft MSL, the PMF flow rate over the Main Reservoir Spillway is 14,190 cfs, and the water elevation in the Main Reservoir is 238.9 ft MSL.

2.1.3 Storm Surge

Storm surge was considered as a combined effect flood in Section 2.1.8 below.

2.1.4 Seiche

Seiche was not considered applicable in the DB.

2.1.5 Tsunami

Tsunami was not considered applicable in the DB.

2.1.6 Ice-Induced Flooding

Ice-induced flooding was not considered applicable in the DB.

2.1.7 Channel Migration or Diversion

Flooding due to channel migration or diversion was not considered applicable in the DB.

2.1.8 Combined Effect Flood

The two-year wind speed generated maximum wave runup at the Main Dam is 4.1 ft This value in combination with the wind setup of 0.1 ft, and the PMF stillwater elevation of 238.9 ft MSL produces a probable maximum water level at the Main Dam of approximately 243.1 ft MSL. This maximum water level is 16.9 ft below the top of the Main Dam, 260 ft MSL. The two-year wind speed generated maximum wave runup on the upstream face of the Auxiliary Dam is 1.9 ft, which in conjunction with the wind setup of 0.1 ft, and the PMF stillwater level of 256.0 ft MSL, yields a probable maximum water level at the Auxiliary Dam of 258.0 ft MSL. This maximum water level is 2.0 ft below the top of the Auxiliary Dam, 260 ft. MSL.

The plant is generally protected from wind-generated waves by high ground from all quadrants. On the plant island, the southerly fill portion of the Emergency Service Water Intake Channel and the embankment faces of the plant island are protected by sacrificial fill. The two-year wind speed generated maximum wave runup and wind setup level on the side of the plant island is 1.7 ft. For a maximum PMF stillwater level of 256 ft MSL in the Auxiliary Reservoir, the maximum water level is estimated to be 257.7 ft MSL, which is 2.3 ft below the plant grade of 260 ft MSL.

The PMH wind wave activity when the water levels in the reservoirs are at normal operation level was considered as a flood hazard. The wind setup and wave runup values were calculated using the NOAA HUR 7-97 report that describes a hypothetical hurricane having a combination of characteristics which will make it the most severe that can probably occur in the particular region. The maximum gradient overland wind speed at the site was calculated to be 123 miles per hour (mph). Because the top of the Main Dam is 40 ft above the normal water level in the Main Reservoir, wave action was not considered on the Main Dam. The wind setup, wave height, and wave period for the critical locations at the Auxiliary Dam and around the plant island were calculated based on known values of fetch, water depth, and wind speed. With a maximum runup of 3.8 ft, a wind setup of 0.4 ft, and the normal reservoir water level, the maximum water level elevation for the Auxiliary Reservoir was calculated to be 256.2 ft MSL, which is 3.8 ft below the top of the Auxiliary Dam. The maximum runup at the plant island was calculated to be 2.7 ft, which when combined with a wind setup of 0.2 ft and a normal operation water level in the Auxiliary Reservoir, resulted in a maximum water elevation of 254.9 ft MSL, which is 5.1 ft below the grade elevation of the plant island. The results of the PMH are bounded by the results of the PMF.

2.1.9 Dam Breaches and Failures

Failure of the Auxiliary Dam, Auxiliary Reservoir Separating Dike, or Main Dam would not result in any rise of water level above elevation 258.0 ft MSL at the Auxiliary Dam and, therefore, the HNP site at elevation 260 ft MSL will not be flooded by downstream dam failures.

2.2 Design Basis Flood Protection and Mitigation Features

The Main Dam and Spillway, Auxiliary Dam and Spillway, Auxiliary Reservoir Separating Dike, Auxiliary Reservoir Channel, ESW Intake Channel, ESW Discharge Channel, ESW Screening Structure, ESW Discharge Structure, and ESW and Cooling Tower Make-Up Water Intake Structure are designed to withstand the effects of the design basis flood hazard level or flood condition. The manholes for electrical cables of the Auxiliary and Emergency Power System, cables of the Auxiliary and Emergency Power System, Diesel Fuel Oil Storage Tank Building (DFOSTB), Diesel Generator Building (DGB), Tank Building, WPB, Fuel Handling Unloading Area Building, Fuel Handling Building (FHB), Reactor Auxiliary Building (RAB), Containment Building (CB), and part of the Turbine Building (TB) are positioned to preclude effects of the design basis flood level or flood condition.

The top of the Main Dam is 16.9 ft above its maximum water level and will therefore not be overtopped. The top of the Auxiliary Dam, at 260 ft MSL, is 2.0 ft higher than its maximum water level. The Auxiliary Separating Dike with its crest at 255 ft MSL will be subjected to overtopping due to waves generated by the PMH wind action on the normal reservoir level or the PMF with associated winds. The upstream and downstream slopes of the dike are protected by riprap. The upstream faces of both Main and Auxiliary Dams will not be subjected to any dynamic forces due to flooding, other than local wave action which is dissipated by the use of riprap. The dams and associated spillways have been designed for hydrostatic forces corresponding to the PMF levels in the two reservoirs. The downstream face of the Main Dam is protected by a layer of oversized rock.

The embankment of the plant island along the Main Reservoir is protected by sacrificial spoil fill. The berm of the spoil fill, at elevation 245 ft MSL, is above the maximum Main Reservoir water level and has a width of 300 ft on the south and southeast exposures. The extent of erosion due to the two worst fetches is estimated to be 150 ft resulting from a PMH duration of 48 hours. Therefore, the 300 ft wide sacrificial spoil fill provides a conservative design. After the event, the eroded portion is inspected, restored, and stabilized where required.

The maximum elevation to which water will pond on the HNP site during a PMP event, assuming the entire drainage system became blocked, would be 261.27 ft. The plant island is capable of draining the PMP-generated runoff by overland flow on the open roads and ground surface directly to the Main Reservoir and to the Auxiliary Reservoir via ESW Intake and Discharge Channels. Ponding to this elevation will not impact the ability to safely shutdown. All safety-related structures which have entrances at elevation 261 ft are protected against ponding through either artificial barriers such as watertight or airtight doors, or low structural barriers, such as curbs (minimum curb elevation is 262.0 ft). The only exceptions to that criteria are entrances to the WPB which are not protected above 261.06 ft but do not provide access to areas that house any safety-related equipment.

The PMP storm water collected in the area between the Retaining Wall and the FHB is pumped out to the storm drainage system using sumps and pumps. In addition to the direct rainfall and groundwater infiltration through the retaining wall, this area collects storm water as overflow from the WPB and the FHB if the drains are assumed to be plugged during the PMP occurrence. If the failure of pumps occurs, the water will accumulate to a level below elevation 236 ft in this area. All openings in the FHB and the WPB below elevation 236 ft have been closed and other penetrations sealed to preclude access of storm water to safety-related areas inside the buildings.

The storm water from the cancelled Unit No. 2 RAB and CB drains in to the centrally located sump and is pumped into the plant drainage system. The sump and pump are sized for the design basis rain fall intensity. However, the wall heights are adequate to accommodate the PMP considering pump failure. All openings below 243.0 ft have been closed and waterproofed to minimize water seepage from this area into Unit No. 1 structures.

All safety-related buildings other than the ESW Intake Structure, Screen Structure and Discharge Structure have structural features surrounding their roofs that would impound rainwater on the roofs assuming that the roof drains are plugged. In general, the ponding is caused by curbing whose height varies depending on the roof but is a maximum of one foot above the high point of the surrounded roof. In addition to curbing around roof edges, the portions of the RAB roofs which wrap around the west side of the CB is partially surrounded by taller structures. Also, the Tank Building has two areas without roofs where walls enclose the tanks. If the regular roof drains are plugged during a local intense PMP event, the storm water will pond on the roof and overflow the curbs. For the local intense PMP event, the water level on all roofs will exceed the top of the surrounding curb by less than three inches except for some areas of the RAB roof which are surrounded by higher walls. In these areas the accumulated water depth will exceed the top elevation of the curb by a maximum of 1.5 ft. The open areas of the Tank Building which are surrounded by 25 ft walls will not overflow; however, rainwater will accumulate to a depth of 23.36 ft. The floor of the unroofed areas of the Tank Building and the roofs of all safety-related buildings where water accumulates are strong enough to withstand the ponding load in addition to other dead and live loads that can reasonably be expected to occur coincident with the PMP event. The varying depths of water on a given roof due to the slope of the roof were accounted for in determining the structural adequacy.

The design basis groundwater level for the HNP site was established to be 251 ft MSL and the subsurface portions of Seismic Category I structures on the plant island are designed for hydrostatic loading with groundwater at elevation 251 ft MSL. HNP structures contain openings and penetrations below grade in exterior walls of structures housing safety-related equipment. The CB, FHB, WPB, RAB, Tank Building, TB, and Fuel Handling Unloading Area are separated by seismic gaps, which are cut off from groundwater by horizontal waterstops between the base mats and vertical waterstops. The exterior walls of the FHB, WPB, RAB, Tank Building, TB, and Fuel Unloading Area are in direct contact with soil and exposed to groundwater. The DGB, DFOSTB, ESW Screening Structure, and the ESW and Cooling Tower Make-up Water Intake Structure also have penetrations below grade. Penetrations for pipes and electrical conduits have been sealed with waterstops and boots in structures housing safety-related equipment. Exterior walls of the buildings which are exposed to groundwater have been provided with impervious bithuthene waterproofing membrane up to elevation 259 ft MSL and all of the vertical and horizontal construction joints in the walls below grade and in the mats, except for the construction joints in the northwest corner walls of the WPB, have been provided with waterstops. Any inleakage through the waterproofing membrane, construction joints or cracks in the reinforced concrete walls or base mats will be handled by floor drains routed to associated sumps and pumps. Any water in the seismic gaps will be drained into the lowest building through weepholes at the lowest level of the gap and will be drained by the Floor Drain System. Any groundwater seeping through the vertical joints in the retaining wall or coming out of the retaining wall drainage system will be collected into drainage sumps and pumped out to the storm drainage system.

Electrical manholes and duct runs for Auxiliary and Emergency Power System cables are capable of normal function while completely or partially flooded. The duct runs are sloped towards the electrical manholes and groundwater in the PVC conduit will be drained to the electrical manholes. The electrical manholes have been provided with collection sumps for water coming through PVC conduits or cracks in the reinforced concrete walls or slabs of the manholes. When necessary, the water in the sumps will be removed by portable pumps.

The CB is a steel-lined reinforced concrete structure. To preclude external water pressure on the steel liner, a continuous impervious PVC waterproofing membrane has been placed between the containment foundation mat and the foundation rock. The waterproofing membrane is continuous under the mat and terminates in the waterstops at the joint with adjacent structures. Leakage through the waterproofing membrane will be drained through porous concrete drains placed between the membrane and the mat. The porous concrete drains lead to two sumps in the RAB mat. Each sump contains two full capacity pumps for redundancy. The porous concrete drains are interconnected so that water at any place has two paths for egress. The pumps discharge water to the Heating, Ventilation, and Air Conditioning (HVAC) Condensate Drainage System. In case of failure of the sump pumps, water will overflow the pump casing pipe at elevation 194 ft MSL and will be drained by the Floor Drain System. Since the top of the casing pipe is at elevation 194 ft MSL and the steel liner at the reactor cavity is at 210 ft MSL, no water pressure will be exerted on the liner.

The Refueling Water Storage Tank (RWST) level transmitters are located in the RWST pit area approximately 1.5 ft above grade. The RWST level transmitters are protected from flooding conditions by installed submersible transmitters and are fully capable of providing their design basis operation during and after maximum PMP flooding event.

Safety-related equipment will not be jeopardized as a result of the maximum still water level due to the PMF or wave runup associated with the PMH or storm water accumulated at the HNP site due to a PMP event; therefore, it will not be necessary to bring the reactor to cold shutdown for flood conditions. The flood protection and mitigation features are not associated with a unique mode of operation of the plant.

2.3 Flood Related Changes to the Design Basis since Licensing Issuance

The local watershed condition on the HNP site has changed slightly due to uneven settlement of surfaces in accordance with recent Digital Elevation Model (DEM) data for the site and photos from a site walkdown. The site surface is not as smooth as the original design and can lead to irregular shapes of flow paths for surface runoffs. Temporary/portable buildings, permanent buildings, paved parking lots, and vehicle (Jersey) barriers have been added to the HNP site since the original license was issued.

There are no significant changes to the watersheds of the Main and Auxiliary Reservoirs from the original design basis.

3.0 Summary of Flood Hazard Reevaluation

3.1 Local Intense Precipitation

The effects of local intense precipitation were evaluated. For the assessment of flood hazards at safety-related Structures, Systems or Components (SSCs), the Hierarchical Hazard Assessment (HHA) process, as described in NUREG/CR-7046 was followed. It is conservatively assumed there are no precipitation losses during the entire PMP event and runoff process, and all underground storm drains, driveway pipes, and culverts are clogged and not functioning during the local PMP storm event. The HNP site is drained by overland flow on open roads and ground surface away from the safety-related structures and directly to the Main Reservoir and the Auxiliary Reservoir.

3.1.1 Hydrologic Analysis

The Rational Method was used for hydrologic analysis. The analysis was performed using the local PMP data for the site. HMR-51 and HMR-52 are the basis for the PMP depth. For the PMP drainage area, the entire HNP site was considered. The HNP site is located in the Harris Lake drainage watershed. The Digital Elevation Model data for HNP site was used as the topography for the site. The Shearon Harris Nuclear Power Plant Storm Water Outfall Study drawing by McKim & Creed was used as a reference for the drainage basin boundary. The data also included roads and building footprints for the HNP site. The data was used to determine the proper watershed boundary and PMP flow paths.

The drainage sub-basin boundary in the McKim & Creed drawing was used as the first draft of the watershed delineation. A representative flow path was then determined for each sub-basin by using the information from contours, roads and building blocks. A field walkdown was performed by the site team to assess current site conditions. Figure 1 in Attachment 2 presents the watershed boundary and flow paths and Table 1 in Attachment 1 shows the area of each site basin.

The rainfall data for PMP was obtained from the HMR-51 and HMR-52 following the procedure in HMR-52 for the calculation of one-hour, one square mile PMP as shown in Table 2 in Attachment 1. From Figure 24 of HMR-52, the one-hour, one-square-mile PMP is 18.9 in. The 30 minute, 15 minute and five minute multipliers are obtained from Figures 38, 37 and 36 in HMR-52, respectively. The local intense precipitation for 30 minutes, 15 minutes and five minutes is 13.9 in, 9.7 in and 6.2 in, respectively as shown in Table 2. In HMR-52, the one-hour storm-area averaged PMP values were derived from 29 storms, which include two tropical storms and 27 non-tropical type storms. All storms occurred between the end of May and early October, which is representative of the local climate pattern at the HNP site. For tropical cyclone type events, the National Hurricane Center normally issues early warnings prior to the storm. For storms which are associated with non-tropical low-pressure centers, the National Weather Service (NWS) usually issues a flood warning if the heavy rainfall is likely to occur in the region.

The time of concentration for each sub-basin is calculated using the Kerby-Kirpich Method as described in the Texas Department of Transportation Hydraulic Design Manual. This approach estimates the time of concentration in two parts, the overland flow time and the channel flow time.

$$T_c = T_{ov} + T_{ch}$$

where

T_{ov} = overland flow time and

T_{ch} = channel flow time.

T_{ov} is estimated by the Kerby Method. This method is applicable to small watersheds where overland flow is an important component of overall travel time. The Kerby Method is given by the following equation:

$$T_{ov} = K(LN)^{0.467} S^{-0.235}$$

where

T_{ov} = overland flow time of concentration, in minutes.

K = unit conversion coefficient, K = 0.828 for U.S. units

L = overland flow length, in feet or meters as indicated by K. The maximum length for overland flow is limited to 1200 ft in Kerby Method.

N = a dimensionless retardance coefficient, N = 0.02 for pavement area.

S = slope of terrain conveying overland flow. The maximum slope is limited to 1% in Kerby Method.

For channel flow component of runoff, the Kirpich equation is given by:

$$T_{ch} = K_{ch} (L_{ch})^{0.770} (S_{ch})^{-0.385}$$

where

T_{ch} = the time of concentration, in minutes,

K_{ch} = a unit conversion coefficient = 0.0078 for U.S. units,

L_{ch} = the channel flow length, in feet or meters as indicated by the units, and

S_{ch} = the dimensionless main channel slope.

The value for time of concentration was also used to justify the rainfall intensity for the Rational Method in each of the large sub-basins. If the time of concentration was less or equal to five minutes, then the 5-minute duration precipitation intensity was applied. Otherwise, the duration was adjusted in accordance with the time of concentration.

The rainfall intensity duration frequency (IDF) curve for the site can be represented by a power function, as shown in Figure 4 in Attachment 2, which was derived from the curve fitting procedure using the data in Table 2. The relationship between the rainfall duration and the rainfall intensity can be expressed in the following equation:

$$I = 181.4663T^{-0.55847}$$

where

I = rainfall intensity in inches/hour and

T = rainfall duration in minutes = time of concentration of a basin (in minutes).

In the calculation procedure for rainfall intensity, the length of time of concentration for each sub-basin was used to calculate the rainfall intensity from the IDF curve. A large value of time of concentration will result in a low rainfall intensity. Conservatively, the slope of each sub-basin is limited by a lower bound value of 0.001 in order to avoid a long time of concentration. The Rational Method applies the rainfall intensity for each sub-basin determined from the IDF curve equation above.

The runoff hydrograph for each sub-basin was calculated using the Rational Method as expressed by the following equation.

$$Q = CIA$$

where

Q = maximum rate of runoff in cfs,

C = runoff coefficient = 1 for conservatism,

I = rainfall intensity for a duration equal to the time of concentration of the watershed, in inches/hour, and

A = drainage area in acres.

The Rational Method is applicable for watersheds under 200 acres in urban area. Each sub-basin in HNP site is smaller than 200 acres and most are covered by paved or gravel surface. Therefore, the Rational Method is suitable for the current site.

Table 1 in Attachment 1 shows the list of sub-basins for this analysis. These sub-basins are further sub-divided into small cross-section divides for the HEC-RAS model. The Rational Method was then applied to each of these small cross-section divides to determine the peak flow rate. Small cross-section divides from the further delineation used the rainfall intensity from the corresponding sub-basin. The peak flows were aggregated toward the downstream direction along each reach. These aggregated peak flows were used as the input to cross-sections in the HEC-RAS model.

In order to verify the assumption that rainfall onto the power block area discharges into retaining areas without contributing to the site drainage volume, the storage capacities of Basins 5, 6, 8, and 16 were compared to the runoff volume. Each area was determined to have adequate storage capacity to retain the associated runoff volume as follows.

In Basin 5, runoff is directed to a sump area that has dimensions of 70 ft by 200 ft (Fuel Handling Building-Retaining Wall Cavity Area). The invert of this sump is at elevation 216 ft and the sump extends to elevation 261 ft before overflowing onto the site. The sump provides 14,000 cubic feet (ft^3) for each foot of storage. The runoff volume of 45,111 ft^3 requires 3.22 ft of storage depth, filling the sump to an elevation of 219.22 ft. All of the runoff volume from the PMP event would be retained within the sump.

In Basin 6, runoff is directed to a sump area that also has dimensions of 70 ft by 200 ft (Fuel Handling Building-Retaining Wall Cavity Area). The invert of this sump is at elevation 216 ft and the sump extends to elevation 261 ft before overflowing onto the site. The sump provides 14,000 ft^3 for each foot of storage. The runoff volume of 136,087 ft^3 requires 9.72 ft of storage depth, filling the sump to an elevation of 225.72 ft. All of the runoff volume from the PMP event would be retained within the sump.

In Basin 8, runoff from the area building roofs ends up within the walls that enclose the Tank Building. This containment consists of two separate spaces; both of which have an invert at elevation 261.0 ft and a top of wall elevation of 286.0 ft. The larger space is 50 ft by 55 ft and contains a 45 ft diameter tank. The space available for water storage is the area outside of the tank. This area provides storage of 1,160 ft^3 for each foot of storage. The smaller space is 35 ft by 35 ft and contains a 27 ft diameter tank. The space available for water storage is the area outside of the tank. This area provides storage of 653 ft^3 for each foot of storage. The total storage volume is 1,813 ft^3 for each foot of storage. The runoff volume of 27,301 ft^3 requires 15.1 ft of storage depth, filling the space in the Tank Building to an elevation of 276.1 ft. All of the runoff volume from the PMP event would be retained within the Tank Building.

In Basin 16, runoff is directed to a sump area of variable dimensions. The lowest portion of that sump has dimensions of 175 ft by 200 ft. The invert of this sump is at elevation 236 ft and the sump extends to elevation 242 ft before extending into an additional sump area. The sump will not overflow onto the site until elevation 261.5 ft is exceeded. The sump provides 35,000 ft^3 for each foot of storage. The runoff volume of 131,865 ft^3 requires 3.76 ft of storage depth, filling the lower sump to an elevation of 239.76 ft. All of the runoff volume from the PMP event would be retained within the sump.

3.1.2 Hydraulic Analysis

Following the calculation of runoff peak flows, the HEC-RAS Model was applied to calculate the water depth and velocity at the power block area which includes safety-related SSCs on the HNP site.

The HEC-RAS cross-sections are generated manually by cutting across the entire sub-basin width following the flow path in each sub-basin. Distance between each cross-section ranges between 150 ft and 450 ft. Distance was adjusted, if necessary, such as moving a cross-section to safety-related SSCs, adding cross-sections to important areas or moving around the bend of streamline. Figure 3 in Attachment 2 presents the distribution of cross-sections and locations of safety-related SSCs. The Manning's roughness coefficient for gravel and concrete/asphalt are 0.025 and 0.013, respectively. The area for gravel surface is approximately 50% of the site and the remaining is covered by asphalt, concrete and roofs. By taking the averaged value over the entire site, the value of 0.02 was used in the model for Manning's roughness coefficient.

The peak flow rates from the Rational Method were used as inputs to the cross-sections in the HEC-RAS steady-state model. For the steady-state run, there was no need of an initial condition in the HEC-RAS model. The downstream boundary condition for each reach was assumed to be at critical depth except for Basins 11 and 12. This was derived from the site condition that the surface runoffs will flow directly into the reservoir in the form of free outfall. For Basins 11 and 12, the normal depths are assumed with a profile slope equal to the ground slope since these two outlets are not close to the embankment of the reservoir. For the upstream boundary condition, normal depths were assumed with averaged basin slope. In the HEC-RAS model, the mixed flow regime option for the steady state was applied for calculation. Water level from the HEC-RAS results at each safety-related structure or component was used to evaluate the flood hazard.

3.1.3 Water Level Determinations

The flood levels at safety-related structure locations under the local PMP condition are summarized in Table 3 in Attachment 1. The total duration of the PMP event is one hour. Peak flows are expected to occur at timing associated to the time of concentration on the order of seven to 52 minutes. Total duration of peak flood elevation is not expected to exceed 30 minutes based on the one-hour rainfall distribution for the local PMP.

As shown in Table 3, the flood level is slightly higher than the DB flood elevation at the WPB. For the other safety-related SSCs, flood levels are lower than the DB flood level or the protected elevation. The original DB flood is 261.27 ft National Geodetic Vertical Datum of 1929 (NGVD29) for local PMP as reported in Section 3.4.1.1 of the HNP FSAR. However, according to the DB, all SSCs are protected from flooding up to elevation 262 ft NGVD29 at the TB, DGB, and DFOSTB by curbing or raised entrances. Only two doors of the Waste Processing Building do not have flood protection for flood levels higher than 261.06 ft NGVD29 according to the HNP FSAR. In addition, the water elevation in the Fuel Handling Building-Retaining Wall Cavity Area has been shown to not rise above the DB elevation of 236 ft MSL.

The potential of erosion due to high velocity flow is low at the site. According to the results presented in Table 3, the maximum velocity at safety-related SSCs is 1.27 feet per second (fps). For flow velocity less than 3 fps, the earth bed will not be eroded according to Chow (Open-Channel Hydraulics).

3.2 Flooding in Streams and Rivers

As representative of the HNP site, portions of the evaluation for the PMF were adopted from the Harris Advanced Reactor Units 2 and 3 (HAR) Combined License Application (COLA) FSAR as detailed below.

The PMF has been defined as an estimate of the hypothetical flood (peak discharge, volume, and hydrograph shape) that is considered to be the most severe reasonably possible at a particular location based on comprehensive hydrometeorological application of PMP and other hydrologic factors favorable for maximum flood runoff. The PMF represents an estimated upper bound on the maximum runoff potential for a given watershed. Thus, the objective of this study was to obtain a PMF hydrograph and estimation of the reservoir flood level to ensure the plant's safety.

The PMF for HNP was developed using the following steps:

- Delineation of the sub-basins of the Buckhorn Creek drainage basin above the Main Dam.
- Development of runoff hydrographs for each sub-basin using the unit hydrograph theory, except the Auxiliary Reservoir and Main Reservoir pool surfaces where the direct rainfall was assumed to be equal to the runoff without any loss and lag.
- Development of the PMP storm hyetograph for the Buckhorn Creek drainage basin using guidance given in HMR-51 and HMR-52 and application of the hyetograph to the unit hydrographs with the appropriate infiltration losses in order to develop the estimated flood hydrographs for each sub-basin and the entire drainage basin.
- Determination of the combined inflow to the Main Dam based on summation of inflow hydrographs from various sub-basins upstream using the HEC-HMS model without conducting reach routing.
- Routing of the PMF hydrograph through the reservoir, spillway, and outlet works using the level pool reservoir routing method to estimate the maximum PMF stillwater level in the reservoirs.

3.2.1 Probable Maximum Precipitation

As representative of the HNP site, evaluations for PMP were adopted from the HAR COLA FSAR as detailed below.

The PMP is theoretically the greatest depth of precipitation for a given duration that is physically possible over a given-size storm area at a particular geographical location at a certain time of the year. Alternatively stated, the PMP is the estimated depth of precipitation for which there is virtually no risk of exceedance. The PMP depths used in this study were calculated using the criteria and step-by-step instructions given in HMR-51 and HMR-52.

The drainage area for the Buckhorn Creek watershed above the Main Dam is 182.1 square kilometers (km^2) (70.3 square miles (mi^2)) and the location of the centroid of the basin is approximately 35°38'00" N, 78°57'22'' W. Using HMR-52 as a guide, the PMP for the Buckhorn Creek drainage basin was developed using the following steps:

- Determination of six-hour Incremental PMP

- Determination of six-hour Incremental PMP Isohyetal Pattern
- Maximization of Precipitation Volume
- Distribution of Storm-Area Averaged PMP over the Drainage Basin
- Development of Design Storm for Basin above the Main Dam
- Development of Design Storm for Drainage Basin above the Auxiliary Dam

3.2.1.1 Determination of Six-hour Incremental PMP

The generalized estimates of all-season PMP depths available from Figures 18 through 47 of HMR-51 were obtained for various-sized areas, both larger and smaller than the drainage area under study for the Buckhorn Creek watershed. The six-hour incremental depth-area-duration data taken from Figures 18 through 47 of HMR-51 was used to produce smooth depth-area-duration curves for the Buckhorn Creek drainage basin above the Main Dam.

From the smooth curves, the depth-area-duration values for a set of standard isohyet area sizes, both larger and smaller than the size of the drainage area under study, were read. The selected standard isohyet area sizes for the study are 10, 25, 50, 100, 175, 300, and 450 mi^2. The depth-area-duration data for the selected standard areas were plotted on a linear paper, and smooth curves were fitted. The PMP values corresponding to an 18-hour duration were read and incremental differences for the first three six-hour periods were obtained by successive subtraction of the values. Each set of six-hour values was plotted against the corresponding area values, and smooth lines were fitted through these points. Using the smooth curves, the data were tabulated for the six-hour incremental PMP differences.

3.2.1.2 Determination of Six-Hour Incremental PMP Isohyetal Pattern

There is a preferred orientation for storms at a particular geographic location. That orientation is related to the general movement of storm systems and the direction of moisture-bearing winds. Based on contours of preferred orientation shown on Figure 8 of HMR-52, the preferred orientation for storms at the location having its latitude 35°38’00” N and longitude 78°57’22” W was 200 degrees. The orientation of the storm pattern to produce maximum precipitation volume in the watershed was found to be approximately 215 degrees. The angular difference in the orientations is 15 degrees, which is less than 40 degrees. This indicates that no adjustment is required in the incremental storm pattern.

3.2.1.3 Maximization of Precipitation Volume

The maximum precipitation volume for the three largest six-hour incremental periods resulting from placement of the storm pattern over the Buckhorn Creek drainage basin above the Main Dam was determined. To do this, it was necessary to obtain the value to be assigned to each isohyet in the pattern that occurs over the drainage basin during each period. Computations used were based on the

HMR-52 procedure. The pattern area size that maximizes the volume of precipitation for the three largest six-hour incremental periods was found to be 259 km^2 (100 mi^2).

3.2.1.4 Distribution of Storm-Area Averaged PMP over the Drainage Basin

It was concluded that the maximum volume occurs for a PMP pattern near 259 km^2 (100 mi^2) when placed over the Buckhorn Creek watershed. With this information, the values for each isohyet for all 12 six-hour increments were then determined. To obtain incremental average depths for this drainage, it was necessary to compute the incremental volumes and then divide each incremental volume by the drainage area.

After obtaining the drainage-averaged PMP storm depths, they were distributed according to ANSI/ANS-2.8-1992 guidelines.

For the HNP evaluation, the guidelines used were from NUREG/CR-7046 which is based on ANSI/ANS-2.8-1992 as used for HAR COLA FSAR.

3.2.1.5 Development of Design Storm for Basin above the Main Dam

Using the PMP rainfall distribution, a design storm was developed. The design storm was developed by accounting for the antecedent rainfall that precedes the PMP storm based on ANSI/ANS-2.8-1992 (NUREG/CR-7046 for the evaluation of the HNP site) guidelines. This design storm, which was used as the rainfall input in the hydrologic modeling, has the following components:

- An antecedent 72-hour storm that comprises 40 percent of the PMP volume
- A 72-hour dry period following the antecedent 72-hour storm
- The full 72-hour PMP following the 72-hour no-rain period

3.2.1.6 Development of Design Storm for Drainage Basin above the Auxiliary Dam

The total drainage area of the Auxiliary Reservoir watershed is 7.8 km^2 (3 mi^2). The smallest area considered in HMR-52 is 26.0 km^2 (10 mi^2), with a 72-hour PMP of 119.6 centimeters (cm) (47.10 in). Extrapolating depth-area-duration curves for a drainage area of 7.8 km^2 (3 mi^2), the 72-hour PMP for the drainage basin above the Auxiliary Dam was found to be 126.62 cm (49.85 in). Using the temporal distribution of the design storm above the Main Dam, the design storm for the drainage basin above the Auxiliary Dam was determined.

3.2.2 Precipitation Losses

As representative of the HNP site, evaluations for Precipitation Losses were adopted from the HAR COLA FSAR as detailed below.

The amount of rainfall loss (the portion that does not contribute to runoff) is a function of the type of soil, the ground cover (vegetated, bare, or paved), and the soil moisture prior to the storm. The amount of rainfall loss can be characterized by various methods; the loss methods and their parameters are selected in accordance with recognizable characteristics of the drainage basin under study. The HEC-HMS model offers several methods for estimating precipitation losses.

The traditional initial and constant loss rate method for PMF computations was selected from the HEC-HMS model precipitation loss methods based on Federal Energy Regulatory Commission (FERC) recommendations. The following assumptions were made:

- Saturated antecedent conditions existed in the entire watershed prior to the start of the PMP
- The initial loss for the sub-basins was zero inches (conservative assumption)
- Infiltration occurs at the minimum rate (for consistency with saturated soil conditions).

To determine the minimum infiltration rate, the average soil type for each sub-basin was determined. The land use in the study basin is primarily forested game lands throughout the watershed, with some transitional and urban areas well beyond the major watershed. The Buckhorn Creek watershed contains primarily three soil types: Creedmoor, Mayodan, and White Store. The U.S. Department of Agriculture (USDA) soil texture can be described approximately as sandy clay loam that falls into hydraulic soil group “C.” The HNP site is classified as heavy industrial, and the remaining area of the Buckhorn Creek watershed can be classified as approximately 85 percent forest and 15 percent transitional lands.

The USDA soil texture at the HNP site is described as approximately sandy clay loam that falls into hydraulic soil group “C.” Based on TR-55 the range of infiltration rates for the hydrologic soil group “C” is 0.05 to 0.15 in/hr. To ensure that the PMF estimate is both conservative and representative of the site, the PMF analysis was performed by taking credit for an initial infiltration loss of 0.15 in/hr, which then decreases linearly to zero at the end of 72 hours of the antecedent storm. During the hours of 72 through 144 when there is no rainfall, but soil is still saturated and depressions are full, the infiltration loss rate was assumed to be zero. During the full PMP event, which includes the hours of 72 and after, the infiltration loss rate was assumed to be zero.

For the HNP evaluation the maximum potential loss rate due to infiltration was estimated using the following formula:

$$Infiltration\,loss\,rate\,(in/hr) = 0.15\left(1-\frac{t}{72}\right)\ for\ 0 \le t \le 72$$

$$Infiltration\,loss\,rate\,(in/hr) = 0.00\ \ for\ 72 \le t \le 360,\ where\ t = time\ (hour).$$

3.2.3 Runoff and Stream Course Model

As representative of the HNP site, evaluations for Runoff and Stream Course Model were adopted from the HAR COLA FSAR as detailed below.

A runoff model was used to transform excess precipitation into surface runoff. For the purpose of this analysis, runoff was modeled using two different methods: one for rain falling on land surfaces, and a second for rain falling directly on reservoir pool surfaces. The runoff modeling approach is generally described as follows:

- Land Surface Areas - Unit hydrographs were applied to transform excess rainfall over land surface areas into runoff

- Reservoir Pool Surface Areas - Precipitation falling directly over reservoir pool areas was converted into runoff without considering any infiltration loss or lag time

- No reach routing was used; traveling time of runoff from land areas into the reservoir was neglected

- Level pool routing was used to determine the PMF elevations in both the Main and Auxiliary Reservoirs

3.2.3.1 Runoff Model

An overland runoff model is generally represented in the form of a unit hydrograph. A unit hydrograph is defined as the direct runoff hydrograph produced by one unit (inch) of effective rain uniformly distributed over a sub-basin. Unit hydrographs are combined with precipitation data to determine the direct runoff hydrograph for a given storm event in a particular basin. Thus, separate unit hydrographs were developed for each sub-basin using their specific hydrologic parameters.

Several different methods can be used to develop a unit hydrograph for a given sub-basin. Selection of an appropriate method depends on knowledge of its hydrologic response characteristics. Based on the hydrologic characteristics of the Buckhorn Creek drainage basin, the Snyder hydrograph method was selected as acceptable. The required hydrologic parameters for developing the Snyder's synthetic unit hydrographs were readily available. The HNP FSAR calculated the required generalized values of the shape coefficients that are empirical in nature. The other parameters of the Snyder's method were determined from the geometry of each sub-basin.

The following information summarizes the Snyder's synthetic hydrograph method. The Snyder unit hydrograph relationships define only the unit hydrograph peak discharge (Q_P) and the lag time (t_L) that are defined as:

$$t_L = CC_t(LL_C)^{0.3}$$

$$Q_P = (640C_PA)/t_L$$

where

L = flow path length from outlet to the hydraulically farthest point (basin divide),

L_C = flow path length from outlet to sub-basin centroid,

C_t = Snyder basin lag coefficient, and

C_P = Snyder peaking coefficient

The parameters C_t and C_P are strictly empirical values often recommended as applicable to a specific region. Coefficient C_t accounts for storage and shape of the watershed, and C_P is a function of flood-wave velocity and storage. The generalized values of C_t and C_P as given in the HNP FSAR are 3.91 and 0.75, respectively.

To apply the unit hydrograph approach to the Buckhorn Creek drainage basin, unit hydrographs were developed for three surfaces: (1) Main Reservoir pool surface, (2) Auxiliary Reservoir pool surface, and (3) Residual Land Surface around the Main Reservoir and the seven sub-basins in the Buckhorn Creek drainage basin above the Main Dam. Sub-basins I, II, and III fall below the Main Dam spillway. Therefore, these sub-basins were not considered in the drainage area at the Main Dam. Excluding these sub-basins, the total drainage area at the Main Dam is 182.1 km^2 (70.3 mi^2). This area also includes the drainage area at the Auxiliary Reservoir.

A unit hydrograph has meaning only in connection with a specific duration of runoff. A sub-basin may have many different unit hydrographs, each associated with a different duration of runoff. The catchment lag is a parameter used in unit hydrograph theory to provide a global measure of the response time of a catchment area. Since this global parameter incorporates various basin characteristics, such as hydraulic length, gradient, drainage density, and drainage patterns to determine these characteristics, it was necessary to delineate the sub-basins according to their drainage pattern.

More conservative alternate parameters were used for the residual area. A lag time of 10.6 hours was obtained by substituting the geometric characteristics associated with the land area surrounding the Main Reservoir in the Snyder's unit hydrograph equations. To increase conservatism, the calculated lag time was reduced from 10.6 hours to 1.7 hours by assuming a coefficient of L = 0.4 and of LC = 0.15 in the equation above for lag time. By decreasing the lag time, the peak flow increases from 796 cfs to 4,992 cfs within the residual area.

Using the standard Snyder hydrograph parameters and the more conservative lag time and peak flow parameters for the residual area as input in HEC-HMS model, one-hour unit hydrographs were developed.

3.2.3.2 Hydrograph Peaking

For this reevaluation, in order to compensate for the nonlinearity of extreme precipitation-generated flood runoff, unit hydrographs from most sub-basins were peaked by 25% and the time to peak was decreased by 20%. Unit hydrographs for the reservoir water surfaces were direct transformations of rainfall to inflow with no need for peaking. Unit hydrographs for the Main Reservoir water surface and the Residual Land Surface were adjusted from the HAR COLA FSAR model input by a ratio of the drainage areas.

3.2.3.3 Dam Spillways

Both the Main Dam and the Auxiliary Dam have uncontrolled ogee spillways. The crest of the Main Dam spillway is at elevation 220 ft NGVD29, and the crest of the Auxiliary Dam spillway is at elevation 252 ft NGVD29. The elevation of the top of both dams is 260 ft NGVD29. The spillway crest at the Main Dam has a net length of 50 ft with a pier at its mid-length, while the spillway crest at the Auxiliary Dam has a length of 170 ft. Both spillways are ogee-shaped and designed with a design head and the upstream dam height of 10 ft and 30 ft, respectively, for the Main Dam spillway, while the corresponding values for the Auxiliary Dam spillway are five feet and seven feet, respectively.

3.2.3.4 Discharge Rating Curves

For this reevaluation, spillway discharge rating curves were based on Figures 2.4.3-3 and 2.4.3-4 of the HNP FSAR. The input table for the Auxiliary Dam spillway for the HEC-HMS model was taken directly from a HAR COLA FSAR supporting calculation. An extended rating curve for the Main Dam spillway was calculated based on Figure 2.4.3-4 of the HNP FSAR, by back-calculating the discharge coefficient as 3.5. This was necessary for reservoir elevations above 239.5 ft NGVD29.

The discharge over an ogee crest is given by the following equation:

$$Q = CLH_e^{3/2}$$

where

Q = discharge (cfs),

L = effective length of crest (ft),

H_e = total head on the spillway crest including velocity of approach (ft), and

C = variable discharge coefficient.

The effective length of the spillway was determined by taking contraction effects from piers and abutments into account. The effective length of the spillway (L) was determined using the following relationship:

$$L = L' - 2(NK_P + K_a)H_e$$

where

L' = the net length of the spillway,

N = the number of piers, and

K_P and K_a = the pier and abutment contraction coefficients, respectively. For the Main Dam spillway, K_P = K_a = 0.01 and N = 1. Further, K_P = K_a = 0.01 and N = 0 for the Auxiliary Dam.

The discharge coefficient C varies with the ratio of upstream dam height P to water depth above the spillway crest H_0 and with the ratio of total head H_e to design head H_0. Figures 9.23 and 9.24 in Section 9.12 of Design of Small Dams provide discharge coefficient curves. To determine the discharge coefficients, the following relationships were developed and used in the calculations:

$$C = C_0 [0.86242043 + 0.13731086(H_e / H_0)^{0.5}]^2$$

where, C_0 is the discharge coefficient when $H_e = H_0$. The HAR COLA FSAR gives C_0 = 3.95 for the Main Dam spillway and 3.92 for the Auxiliary Dam spillway. However, from the existing rating curve, HNP FSAR Figure 2.4.3-3, C is clearly less than 3.95 for the Main Dam spillway. Therefore, a back calculation of C was performed to fit Figure 2.4.3-3. Then, the highest value was used to extend the rating curve from elevation 240 ft NGVD29 to elevation 260 ft NGVD29. The back-calculated discharge coefficient C was calculated as 3.5.

3.2.4 Probable Maximum Flood Flow

PMF hydrographs for the various sub-basins and the entire Buckhorn Creek drainage basin were developed using the HEC-HMS model incorporating: (1) application of the one-hour incremental effective PMP values to the unit hydrographs of various sub-basins considering 25% peaking, and (2) values of initial loss and infiltration parameters. The HEC-HMS model is flexible and offers many options to input precipitation, to estimate runoff hydrographs, and to manipulate and route hydrographs. HEC-HMS has been used extensively throughout the U.S. to predict stream flows in both gauged and non-gauged watersheds.

The runoff contributions from the various sub-basins positioned up-gradient of the Main Dam were aggregated without considering their travel times from their most upstream inflow points to the Main Dam, to determine the combined PMF inflow hydrograph. Level pool routing was used along with the stage-storage curve and storage-discharge curves (based on the HNP FSAR) of the dam spillways to estimate the maximum PMF stillwater level in the Auxiliary and Main Reservoirs. For the Main Reservoir the peak inflow is 127,989 cfs and the peak outflow is 19,988 cfs. For the Auxiliary Reservoir the peak inflow is 6962 cfs and the peak outflow is 6261 cfs.

3.2.5 Water Level Determinations

For the Auxiliary Reservoir, the maximum PMF stillwater level is 256.50 ft NGVD29. For the Main Reservoir, the maximum PMF stillwater level is 243.84 ft NGVD29. With a nominal plant grade and top of Main and Auxiliary Dams elevation of 260 ft NGVD29, the reevaluated PMF will not overtop the dams or flood the site.

3.3 Storm Surge

The effect of extreme winds generated by the Probable Maximum Hurricane (PMH) to generate wind setup and wave action was evaluated. Methodologies consistent with the HAR COLA FSAR were used in the analyses and were supplemented with guidance from NUREG/CR-7046 as appropriate.

3.3.1 Probable Maximum Winds and Associated Meteorological Parameters

The maximum wind speed for the PMH used in the reevaluation was adopted from the HAR COLA FSAR as detailed below.

The meteorological characteristics used to calculate the PMH were obtained from NOAA Technical Report NWS 23. According to this report, the PMH is a hypothetical steady-state hurricane having a combination of values of meteorological parameters that will give the highest sustained wind speed that can probably occur at a specific coastal location. From values of the parameters, a wind field is specified, which is termed the “PMH wind field.” The following are the over-water PMH wind field parameters taken from NOAA Technical Report NWS 23 corresponding to the Milepost 2200 and Latitude of 35.6 degrees:

- Coriolis parameter (f) = 14.584×10^{-5} Sin(35.6) = 0.31 hour^{-1},
- Peripheral pressure (P_w) = 30.12 in,
- Central pressure (P_0) = 26.4 in,
- Radius of maximum wind (R) = 9 nautical miles for small storms, and 25 nautical miles for large storms,
- Forward speed (T) = 10 knots (KT) for small storms, and 34 KT for large storms , and
- Density coefficient, K = 68.7 KT-in.

Using the above parameters, the maximum gradient wind speed (V_{gx}) was calculated for a stationary hurricane using the following relationship:

$$V_{gx} = K\left(P_w - P_0\right)^{\frac{1}{2}} - \frac{f\,R}{2}$$

The obtained value of V_{gx} was 128.7 KT (148.2 mph). The value of V_{gx} was adjusted to the maximum 10-meter, 10-minute value (V_{xs}) for a stationary hurricane using the following relationship:

$$V_{xs} = 0.95 V_{gx}$$

The obtained maximum 10-meter, 10-minute wind speed for a stationary hurricane, V_{xs} is 122.2 KT (140.8 mph). In order to determine wind speed for a moving hurricane, the stationary hurricane wind speed needs to be adjusted for asymmetry due to storm forward speed (T). The asymmetry factor (AF) is given as:

$$AF = 1.5 T^{0.63} T_0^{0.37} Cos(\beta)$$

where

AF = the asymmetry factor in knots,

T = the forward speed of the storm in knots,

T_0 = 1 when A and T are in knots, and

β = the angle between track direction and the surface wind direction.

To be conservative, β was assumed to be zero, giving Cos(β) a value of 1. Substituting values of T_0 and Cos(β), the maximum value of asymmetry factor is:

$$AF = 1.5 T^{0.63}$$

The maximum value of T is 34 KT (39.2 mph). Substituting T = 34 KT in the above equation, the maximum value of asymmetry factor (AF) is 13.8 KT (15.9 mph). Adding the stationary hurricane wind speed (V_{xs}) and asymmetry factor (AF) together, the wind speed for a moving hurricane is 136.1 KT (156.8 mph).

When the center of a hurricane crosses the coast, over-water wind speeds are reduced because of filling by a factor that decreases with travel time after landfall. Kaplan and Demaria developed a mathematical model for predicting decay of maximum sustained surface winds after storm landfall using a combination of physical and empirical considerations. In the simplest version of this model, the maximum winds inland are a function of the maximum winds at landfall and of the travel time after landfall. With the assumption of a track perpendicular to the coastline, it was used to estimate the maximum inland penetration of winds of a given speed using the storm's landfall intensity and speed of motion. This decay model is given as:

$$V(t) = V_b + (R_f V_0 - V_b)\exp(-\alpha t)$$

where

V(t) = the inland storm wind speed on traveling overland for time (t) hours after landfall,

V_b = the background wind speed,

R_f = the initial decay factor just after the landfall

V_0 = the storm wind speed just before the landfall, and

α = a coefficient

The values of R_f, V_b, and α are 1.0, 26.7 KT, and 0.095 $hour^{-1}$, respectively. Substituting these values in the equation above, the decay model can be written as:

$$V(t)=26.7+(V_0-26.7)e^{-0.095t}$$

In the above equation, V(t) and V_0 are in KT and t is in hours. The HNP site is located 225.3 kilometers (km) (140 miles (mi)) inland from the coastline. With a forward speed of 34 KT (39.2 mph), the overland travel time is 3.6 hours.

Substituting the value of the overland travel time t = 3.6 hours and V_0 =136.1 KT into the above equation, the maximum 10-meter, 10-minute overland wind speed is 94.9 KT (109.3 mph).

3.3.2 Fetch Determination

In order to determine impact of wind-wave action due to the Probable Maximum Wind, five straight line fetch distances from the HAR COLA FSAR were used in wave runup and setup calculations. These fetches were determined for the future proposed Main Reservoir level of 240 ft NGVD29. Since several headlands interrupt the fetches when the Main Reservoir level is at the present lower level of 220 ft NGVD29, the fetches were conservative for present conditions.

3.3.3 Wind Speed Corrections

Adjustment to a one-hour duration resulted in a design wind speed of 104.1 mph. Adjustment from overland to overwater wind speed, which applies to all the reservoir fetches, yielded a wind speed of 93.7 mph. Further adjustments were made for each fetch, adjusting the wind speed to the duration required to generate the maximum waves for each fetch length.

3.3.4 Wind Setup and Wave Runup

The PMH-induced wave runup and wind setup at the Main and Auxiliary Reservoirs were determined assuming the normal operating levels in the Auxiliary and Main Dams, and by following the procedure given in the U.S. Army Corps of Engineers' Coastal Engineering Manual, Engineer Manual 1110-2-1100.

Wind Setup was calculated by using the Zuider Zee equation:

$$S=U^2X/(1400d)$$

where

S = setup in feet above the stillwater level,

X = fetch length in miles, and

d = average water depth in feet.

Average depth of the reservoirs was calculated by taking the storage volume at the stillwater elevation divided by the surface area. The HAR COLA FSAR used a different approximation of average depth that underestimated the wind setup for the Main Reservoir and thus, that value was not used. For the Auxiliary Reservoir, wind setups in the HAR COLA FSAR were higher than those newly calculated and the higher value from the HAR COLA FSAR was used in subsequent calculations. Wind setup values are shown in Table 4 in Attachment 1.

Wave generation and runup were estimated by methods of the Coastal Engineering Manual. Deep-water significant wave height H_s and mean period T_m were determined as a function of fetch length and wind speed. Wave runup was determined from the following relationships, based on laboratory model studies. Wave runup was determined for rough impermeable slopes as a function of the slope angle and the wave steepness.

Wave steepness = $s_{om} = 2\pi H_s/(gT_m^2)$

Surf-similarity parameter = $\xi_{om} = Tan\ \alpha/(s_{om})^{1/2}$ where α is the ground slope

The ground slope of the runup location is an important variable. The slopes of the upstream and downstream faces of the Auxiliary Dam are 0.4, and the slope of the upstream face of the Main Dam is 0.5. Wave runup in the HNP vicinity was calculated for the steepest ground slope facing either reservoir, 0.2.

For values of $\xi_{om} < 1.5$, wave runup height above stillwater is given by

$$R/H_0 = A\ \xi_{om}$$

For values of $\xi_{om} > 1.5$, wave runup height above stillwater is given by

$$R/H_0 = B\ (\xi_{om})^C$$

Wave runup was calculated for waves of 10%, 2%, 1% and 0.1% frequency, meaning the percent of waves in a given wave train that reach that elevation. For these frequencies, the coefficients A, B and C in the above equations are:

Wave Frequency	A	B	C
0.1%	1.12	1.34	0.55
1%	1.10	1.24	0.48
2%	0.96	1.17	0.46
10%	0.77	0.94	0.42

According to NEI white paper Post-Fukushima Near-Term Task Force Recommendation 2.1 Supplemental Guidance for the Evaluation of Dam Failures, Rev. B, maximum wave runup should be taken as the 1% wave, meaning the highest of 100 waves. For a wave train of a four-second period, only one wave every 6.7 minutes would reach this elevation. Wave runup values are given in Table 4.

3.3.5 Maximum PMH Elevation Due to Coincident Wind-Wave Activity

The maximum PMH elevation due to coincident wind-wave activity was calculated by the summation of the initial stillwater elevation, wind setup, and wave runup. The values for the locations analyzed are presented in Table 4. No wave runup elevations reached the nominal plant grade of 260.0 ft NGVD29 or dam crest elevations of 260.0 ft NGVD29.

3.4 Seiche

As representative of the HNP site, evaluations for resonance were adopted from the HAR COLA FSAR as detailed below.

In order to discuss a possibility of oscillations of waves at natural periodicity such as lake reflection and harbor resonance phenomena and any resulting effects at the site, two sinusoidal waves of the same amplitude and wavelength traveling in opposite directions are considered and their interference can be studied through the equations below:

$$y(x,t) = y_0 \sin(kx-\omega t)$$

$$y(x,t) = y_0 \sin(kx+\omega t)$$

where y_0 is the initial magnitude of a wave, k is the wave number represented as $2\pi/\lambda$, in which λ is a wavelength of oscillations generated by an external perturbation. The symbol ω represents the angular frequency and is represented as $2\pi/T$, in which T is the time period, and x and t represent location and wave travel time. According to the superposition principle, the resultant wave is simply the sum of the two waves, i.e.:

$$y_{Resonance}(x,t)= y_0 \sin(kx-\omega t)+ y_0 \sin(kx+\omega t)$$

Using trigonometry identities, the sum of two sines is given by:

$$y_{Resonance}(x,t)= 2y_0 \sin(kx)\cos(\omega t)$$

The amplitude of the resonance wave is $2y_0\sin(kx)$, which varies with position x. Alternatively, the above equation can also be written as:

$$y_{Resonance}(x,t)= 2y_0 \sin(2\pi x/\lambda)\cos(2\pi t/T)$$

Well defined points exist where the magnitude of the resonance wave is zero. Such as x=0, λ/2, λ, 3λ/2, 2λ, etc; these points are called the nodes. Similarly, points of maximum magnitude oscillations, called the antinodes, are the locations of maximum amplitude oscillations such as at x = λ/4, 3λ/4, 5λ/4, etc. At the locations, the two waves undergo constructive interference and result in resonance with magnified magnitude. This indicates that the length of a water body over which waves are generated should be a multiple of λ/2. This multiple is also known as the mode or harmonic of an oscillating wave. The oscillation resonance mode n can be determined as:

$$n=2L/\lambda$$

The corresponding resonance period T_n is given as"

$$T_n=2L/(n\sqrt{(gh)})$$

where L is the length, h the average depth of the body of water, and g is the acceleration of gravity. Using the physical parameters of the Main and Auxiliary Reservoirs, the numbers of modes n at which resonance may occur were determined and all modes are over 100.

The amplitude and persistence of a wave depends not only on the magnitude of energy source but also on the energy losses within a water body. Such losses include dissipative effects resulting from friction of the sides and bottom of the basin. If a wave is generated by an external impulsive event such as a sudden change in atmospheric pressure gradient, the amplitude is seen to decay by nearly constant fractions with each successive period or mode. In general, the rate of decay is greater for basins that are shallow or have narrow constrictions and complex topography.

Using geometric progression and assuming a constant rate loss of r, the magnitude of resonance wave A_n after n modes is given as:

$$A_n=2y_0(1-r)^{n-1}$$

This equation indicates that only the first few modes are important; as n increases, A tends to decrease. At a large value of n, A_n becomes insignificant. Based on literature, the decay rate r in Lake Geneva and Lake Erie has been estimated to be approximately three percent with each successive wave period. Both the Main and Auxiliary Reservoirs are very shallow and have narrow constrictions with complex topography. Therefore, it is expected that the decay rate r will be large. In the absence of observed data, a conservative decay rate of ten percent was assumed and the magnitude of resonance wave after n modes A_n was determined.

The magnitude of resonance wave for all fetch lengths is zero. Therefore, wave amplification due to resonance will not occur on the Auxiliary or Main Reservoirs at the HNP site, because the wind fetch is approximately 100 times longer than the significant wave length. The resonance due to such a high mode, if it does occur, would not have an appreciable effect due to the fact that only the first few modes of resonance are of concern for wave amplification.

3.5 Tsunami

As representative of the HNP site, evaluations for tsunami were adopted from the HAR COLA FSAR as detailed below.

Coastal areas bordering the Pacific Ocean and U.S. territories in the Caribbean, notably Puerto Rico and the US Virgin Islands, are most susceptible to tsunamis. The HNP site is located approximately 225.3 km (140 mi) inland from the Atlantic coast, where tsunami hazards are relatively low. Therefore, the HNP site is not subjected to the effects of tsunami flooding.

The potential for a slope failure into the Main or Auxiliary Reservoirs causing a tsunami-like wave is negligible based on the extensive site-specific investigations associated with topography, geology, seismicity, and groundwater. In addition, the current land use is not conducive to landslide activity and no observed or recorded land slippage of any kind has occurred along the shore of either the Main or Auxiliary Reservoirs since the reservoirs were filled in late 1980.

3.6 Ice-Induced Flooding

As representative of the HNP site, evaluations for ice effects were adopted from the HAR COLA FSAR as detailed below.

A review of historical temperature records from the NWS Cooperative Observer Station No. 317069 in Raleigh, North Carolina, for the period 1971 to 2000 indicates monthly average minimum temperatures for the months of December, January, and February as being 0.33 degrees Celsius (°C), -1.33°C, and -0.06°C (32.6 degrees Fahrenheit [°F], 29.6°F, and 31.9°F), respectively. The monthly mean temperatures for the same months are 6.11°C, 4.3°C, and 6.1°C (43.0°F, 39.7°F, and 43.0°F), respectively. Ice formation in this locality on large bodies of water in Central North Carolina is expected to be limited to minor freezing along shorelines. It is not expected to be severe enough under any circumstances to jeopardize the operation of the safety-related structures.

3.7 Channel Migration or Diversion

As representative of the HNP site, evaluations for channel migration or diversion were adopted from the HAR COLA FSAR as detailed below.

There is no historical evidence of channel diversion above the Main Dam within Buckhorn Creek, Tom Jack Creek, Thomas Creek, Little White Oak Creek, White Oak Creek, or Cary Creek. Examination of US Geological Survey 1:24,000-scale topographic maps associated with the Buckhorn Creek drainage basin did not reveal evidence of natural channel diversions (e.g., oxbow lakes or broad, well developed floodplains). Creeks and streams within the watershed generally occur in well-defined valleys and, therefore, limit the possibility of water diversion into adjacent drainage basins.

Topographic characteristics and geological features of the drainage basin indicate there is no possibility for the occurrence of a landslide blocking or limiting streamflow into Harris Lake.

Because ice effects are expected to be limited to minor freezing, they are not expected to create flow diversion during winter months.

3.8 Combined Effect Floods

Combined Effect floods are events considered reasonably likely to occur at the same time at a given location, and to provide an adequate design flood basis. Recommended combinations of events are discussed in ANSI/ANS-2.8-1992 Section 9.2, and also in NUREG/CR-7046 Appendix H. Two combined effect events were considered for the HNP site: the probable maximum hurricane-induced probable maximum storm surge in combination with the maximum controlled reservoir levels (as detailed in Section 3.3 above) and the PMF with wind-wave activity resulting from the two-year wind speed.

3.8.1 PMF with Wind-Wave Activity

A two-year fastest-mile wind speed of 50 mph was taken from ANSI/ANS 2.8, 1992 for the site. This value was conservatively taken as applicable to a one-hour duration. The fetches determined in Section 3.3.2 were used for this evaluation. The wind speed was adjusted for each fetch to the length of time required to develop the maximum wave height for that fetch. Wind setup and wave runup were calculated in the same manner as in Section 3.3.4. The results are detailed in Table 5 in Attachment 1. The maximum (1%) wave runup elevations resulting from the PMF with two-year wind speed are 249.80 ft NGVD29 at the Main Dam and 259.34 ft NGVD29 at the Auxiliary Dam. The maximum runup near HNP is 257.64 ft NGVD29 on the Auxiliary Reservoir, and 246.94 ft NGVD29 on the Main Reservoir. These levels are below plant grade of 260.0 ft NGVD29 and the crests of the dams at elevation 260 ft NGVD29.

The watershed of the Auxiliary Reservoir is only 3.0 square miles in area and has no tributary streams more than two miles long; therefore, the potential for debris generation along the tributaries and shores is limited. The uncontrolled spillway is 170 ft long with no piers or superstructure. There are no structures with significant potential to trap debris. Since no overtopping was demonstrated, the evaluation was terminated and it can be concluded that the embankment is safe from hydrologic failure.

3.9 Dam Breaches and Failures

As representative of the HNP site, evaluations of upstream dams were adopted from the HAR COLA FSAR as detailed below.

There are no existing dams upstream or downstream of Harris Reservoir that can affect the site safety-related facilities or the availability of the cooling water supply. The National Hydrography Dataset was reviewed to identify impoundments in the Buckhorn Creek Drainage Basin. All impoundments other than the Auxiliary and Main Reservoirs were less than or equal to ten acres in size and were not considered to be large enough to affect safety-related facilities or the availability of the cooling water supply.

Additionally, failure of the Auxiliary Dam and Main Dam was considered in order to determine the availability of the Ultimate Heat Sink (UHS). ESW for safe shutdown of HNP can be drawn from either the Auxiliary Reservoir or the Main Reservoir. If either dam were to fail, the ESW supply could be challenged. As detailed in Sections 3.2, 3.3, and 3.8, the evaluations for flooding in streams and rivers, the analysis of PMH-induced storm surge, and the combined effect of two-year wind with the PMF show that no overtopping of either the Auxiliary or Main Dams will occur. Since no overtopping was demonstrated, it is concluded that the embankment is safe from hydrologic failure.

4.0 Comparison with Design Basis

For each flood hazard reevaluated, the result was compared to the DB flood hazard and protection and mitigation features to determine whether the safety-related SSCs would remain protected. The results are summarized in Table 6 in Attachment 1.

4.1 Local Intense Precipitation

The DB flood hazard levels for local intense precipitation is elevation 261.27 ft NGVD29. The reevaluated flood hazard levels are listed for buildings and structures on site in Table 6. Though the reevaluated flood hazard levels are slightly higher in the DFOSTB, it is protected up to 262 ft NGVD29. The flood hazard level is also higher than DB at the Waste Processing Building.

4.2 Flooding in Streams and Rivers

The DB flood hazard levels for flooding in streams and rivers at the Main Dam, Auxiliary Dam, and the HNP site are 238.9, 256.0, and 256.0 ft NGVD29, respectively. The reevaluated flood hazard levels for these areas are 243.84, 256.50, and 256.50, respectively. Though these values are slightly higher than DB, the areas are protected up to 260 ft NGVD29.

4.3 Storm Surge

The DB flood hazard level for PMH-induced storm surge was not evaluated for the Main Dam. The DB hazard levels at the Auxiliary Dam and the HNP site are 256.2 and 254.9 ft NGVD29, respectively. The reevaluated flood hazard levels for the Main Dam, Auxiliary Dam, and the HNP site are 233.43, 257.85, and 254.47 ft NGVD29. Though the value is slightly higher than DB at the Auxiliary Dam, all of these areas are protected up to 260 ft NGVD29.

4.4 Seiche

Seiche remains not applicable to the HNP site.

4.5 Tsunami

Tsunami remains not applicable to the HNP site.

4.6 Ice-Induced Flooding

Ice-induced flooding remains not applicable to the HNP site.

4.7 Channel Migration or Diversion

Channel migration or diversion remains not applicable to the HNP site.

4.8 Combined Effect Floods

The DB flood hazard levels for the combined effect flood at the Main Dam, Auxiliary Dam, and plant are 243.1, 258.0, and 257.7 ft NGVD29, respectively. The reevaluated flood hazard levels for these areas are 249.80, 259.34, and 257.64 ft NGVD29, respectively. Though these values are slightly higher than the DB at the Main Dam and Auxiliary Dam, these areas are protected up to 260.0 ft NGVD29.

4.9 Dam Breaches and Failures

The reevaluated flood hazard confirmed the evaluation in the DB that no existing dams upstream or downstream of Harris Reservoir that can affect the site safety-related facilities or the availability of the cooling water supply.

5.0 Interim Actions and Additional Actions

As indicated above, some reevaluated flood levels exceed the flood levels determined for the DB. This condition has been documented in the Corrective Action Program. An integrated assessment will be performed and an Integrated Assessment Report will be submitted by March 12, 2015, in accordance with the March 12, 2012, 50.54(f) Request For Information.

5.1 Local Intense Precipitation:

The reevaluated flood level for the DFOSTB exceeds the DB flood level by 0.14 ft but remains below the DFOSTB flood protection level by 0.59 ft. Therefore, no interim actions are required regarding the DFOSTB.

The reevaluated flood level for the WPB exceeds the DB flood level by 0.09 ft. Both the DB flood level and the reevaluated flood level exceed the WPB flood protection level at two entrances to the WPB. As discussed in the HNP FSAR, the two entrances to the WPB provide access to areas which house locker room, shower stalls and do not house any safety-related equipment. A conservative evaluation determined that approximately 1212 ft^3 of water could enter the WPB during the reevaluated PMP through small openings around the two entrance doors. Assuming the floor drains were clogged, this water would pond inside the WPB to a maximum depth of 0.07 ft in an area of the WPB that contains no safety-related equipment. Therefore, no interim actions are required regarding the WPB.

5.2 Probable Maximum Flood:

The reevaluated flood level for the Main Dam exceeds the DB flood level by 4.94 ft but remains below the Main Dam flood protection level by 16.16 ft. Therefore, no interim actions are required regarding the Main Dam.

The reevaluated flood level for the Auxiliary Dam exceeds the DB flood level by 0.5 ft but remains below the Auxiliary Dam flood protection level by 3.5 ft. Therefore, no interim actions are required regarding the Auxiliary Dam.

The reevaluated flood level for the HNP site exceeds the DB flood level by 0.5 ft but remains below the HNP site elevation by 3.5 ft. Therefore, no interim actions are required regarding the HNP site.

5.3 Probable Maximum Hurricane – Storm Surge:

The reevaluated flood level for the Auxiliary Dam exceeds the DB flood level by 1.65 ft but remains below the Auxiliary Dam flood protection level by 2.15 ft. Therefore, no interim actions are required regarding the Auxiliary Dam.

5.4 Combined Effects:

The reevaluated flood level for the Main Dam exceeds the DB flood level by 6.7 ft but remains below the Main Dam flood protection level by 10.2 ft. Therefore, no interim actions are required regarding the Main Dam.

The reevaluated flood level for the Auxiliary Dam exceeds the DB flood level by 1.34 ft but remains below the Auxiliary Dam flood protection level by 0.66 ft. Therefore, no interim actions are required regarding the Auxiliary Dam.

ATTACHMENT 1: TABLES

Table 1. HNP Site Basins

Basin ID	**Area** acres	**Note**
1	71.87	
2	6.28	
3	1.02	Cooling Tower
4	37.59	
5	0.66	Containment Building, NW
6	1.98	Containment Building, SW
7	22.10	
8	0.40	Containment Building, SE
9	11.22	
10	59.14	
11	7.94	
12	36.98	
13	3.60	
14	8.05	
15	6.39	
16	1.92	Containment Building, NE
17	8.02	
18	3.43	
Total	288.59	

Table 2. Local Intense Precipitation for the HNP Site

Duration	**Area** mi^2	**Multiplier**	**Applied to**	**Local Intense Precipitation** inches
1 hr	1	NA	NA	18.9 (HMR-52, Fig. 24)
30 min	1	0.741 (HMR-52, Fig. 38)	1-hr, 1 mi^2 PMP	13.9
15 min	1	0.514 (HMR-52, Fig. 37)	1-hr, 1 mi^2 PMP	9.7
5 min	1	0.327 (HMR-52, Fig. 36)	1-hr, 1 mi^2 PMP	6.18

Table 3. Water Level at Safety-Related Structures

Locations	**Reach**	**Cross-section**	**Velocity** ft/sec	**Froude Number**	**Water Level** ft NGVD29	**DB Flood Protected Elevation** ft, NGVD 29	**Level Above DB** ft
Waste Processing Building	Basin_4	XS_4_9	0.85	0.16	261.36	261.06*	0.30
Turbine Generator Building	Basin_7	XS_7_5	0.39	0.07	261.25	262**	N/A
Emergency Service Water & Cooling Tower Make-up Intake Structure	Basin_13	XS_13_5	0.66	0.15	260.82	262**	N/A
Emergency Service Water Intake Screen Structure	Basin_9	XS_9_8	1.27	0.33	260.52	262**	N/A
Diesel Generator Building	Basin_7	XS_7_3	1.23	0.25	261.12	262**	N/A
Diesel Fuel Oil Storage Tank Building	Basin_4	XS_4_14	0.06	0.01	261.41	262**	N/A

*DB flood is 261.27 ft NGVD29 as reported in the HNP FSAR.

**DB protected elevation against ponding through either artificial barriers such as watertight or airtight doors, or low structural barriers such as curbs at elevation 262 ft NGVD 29.

Table 4. Wind Setup and Wave Runup for the PMH-Induced Storm Surge

Location	**Fetch Number**	**Fetch length** mi	**Slope**	**Setup** ft	**Maximum (1%) Runup** ft	**Maximum (1%) elevation** ft NGVD29
Main Dam	1	4.29	0.5	1.55	11.88	233.43
Auxiliary Dam Downstream	2	4.29	0.4	1.55	10.92	232.47
Auxiliary Dam Upstream	3	1.17	0.4	0.63	5.22	257.85
Unit 1, Auxiliary Reservoir	1	0.76	0.2	0.42	2.05	254.47
Unit 1, Main Reservoir	2	4.33	0.2	1.56	5.46	227.02

Table 5. Wind Setup and Wave Runup for the PMF

Location	**Fetch Number**	**Fetch length** mi	**Slope, V/H**	**Setup** ft	**Maximum (1%) Runup** ft	**Overall PMF elevation,** ft NGVD29
Main Dam	1	4.29	0.5	0.29	5.67	249.80
Auxiliary Dam Downstream	2	4.29	0.4	0.29	5.09	249.23
Auxiliary Dam Upstream	3	1.17	0.4	0.135	2.70	259.34
Unit 1, Auxiliary Reservoir	1	0.76	0.2	0.09	1.05	257.64
Unit 1, Main Reservoir	2	4.33	0.2	0.30	2.80	246.94

Table 6. Summary of Comparison of Reevaluated Flood Hazards and DB

Hazard	Location	Protected Level ft NGVD29	DB Hazard ft NGVD29	Reevaluated Hazard ft NGVD29	Bounded by DB	Protected
Local Intense Precipitation	Diesel Fuel Oil Storage Tank Building	262.00	261.27	261.41	No	Yes
	Diesel Generator Building	262.00	261.27	261.12	Yes	Yes
	Waste Processing Building	261.06	261.27	261.36	No	No
	Part of Turbine Building	262.00	261.27	261.25	Yes	Yes
	Emergency Service Water Screening Structure	262.00	261.27	260.52	Yes	Yes
	Emergency Service Water and Cooling Tower Make-Up Water Intake Structure	262.00	261.27	260.82	Yes	Yes
Flooding in Streams and Rivers	Main Dam	260.00	238.9	243.84	No	Yes
	Auxiliary Dam	260.00	256.0	256.50	No	Yes
	Unit 1 Plant Island	260.00	256.0	256.50	No	Yes

Table 6. Summary of Comparison of Reevaluated Flood Hazards and DB (continued)

Hazard	**Location**	**Protected Level** (ft NGVD29)	**DB Hazard** (ft NGVD29)	**Reevaluated Hazard** (ft NGVD29)	**Bounded by DB**	**Protected**
Storm Surge	Main Dam	260.00	-	233.43	-	Yes
	Auxiliary Dam	260.00	256.2	257.85	No	Yes
	Unit 1 Plant Island	260.00	254.9	254.47	Yes	Yes
Combined Effect	Main Dam	260.00	243.1	249.80	No	Yes
	Auxiliary Dam	260.00	258.0	259.34	No	Yes
	Unit 1 Plant Island	260.00	257.7	257.64	Yes	Yes

ATTACHMENT 2: FIGURES

Figure 1. Delineated watershed for the HNP site including flowpaths

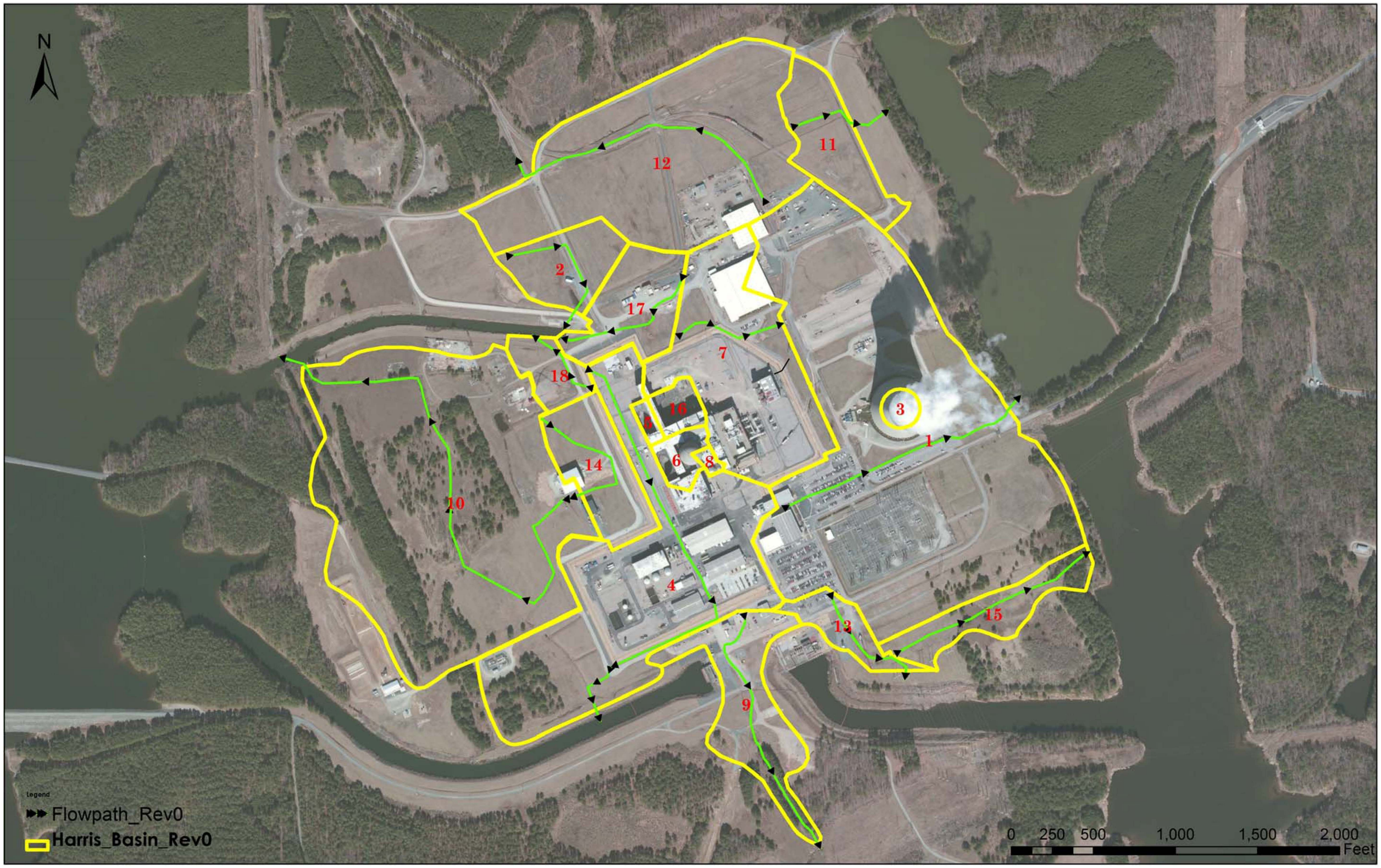

Figure 2. Delineated watershed for the HNP site with contributing area for each cross-section

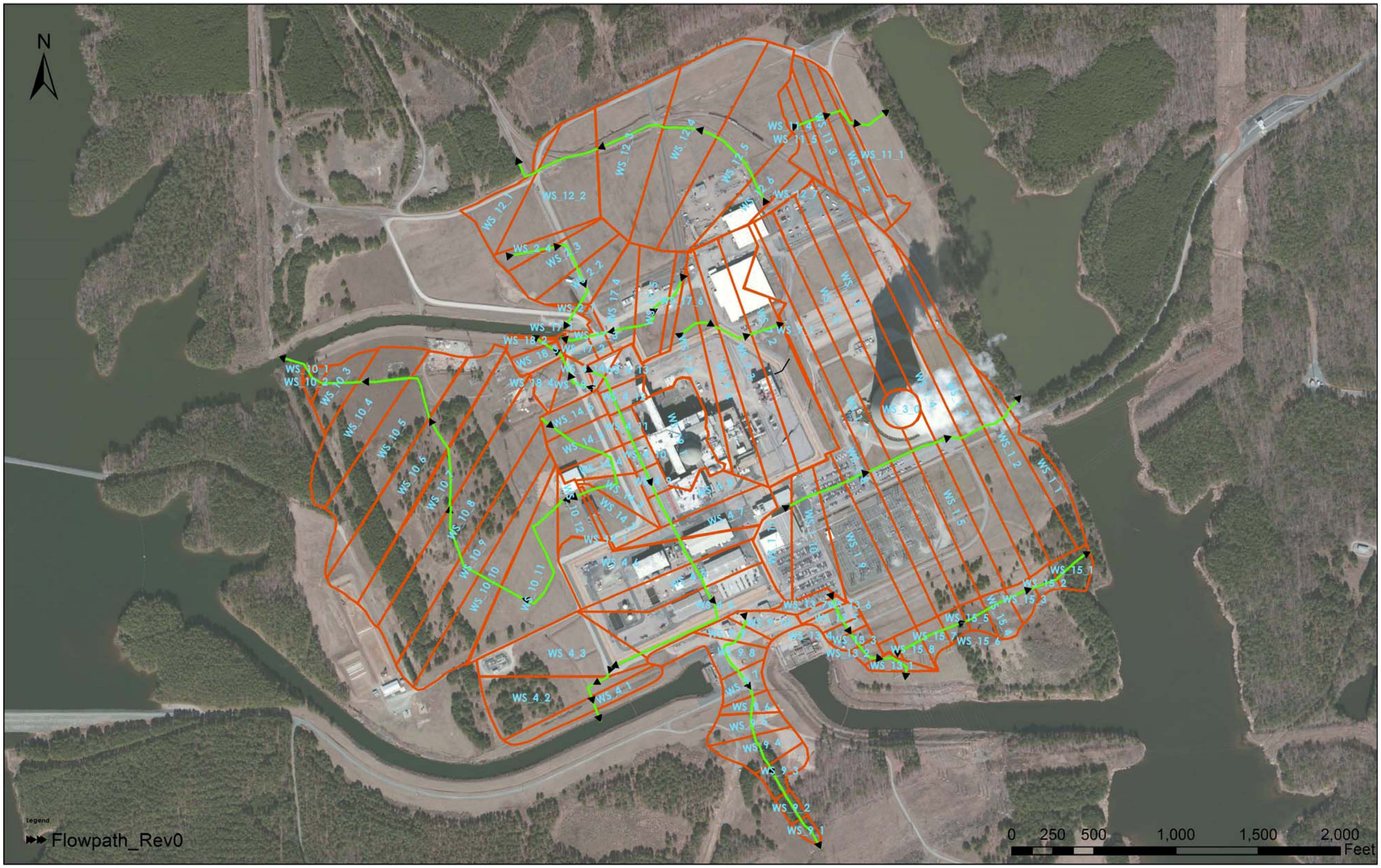

Figure 3. Delineated watershed for the HNP site with cross-sections and location of safety-related structures and buildings

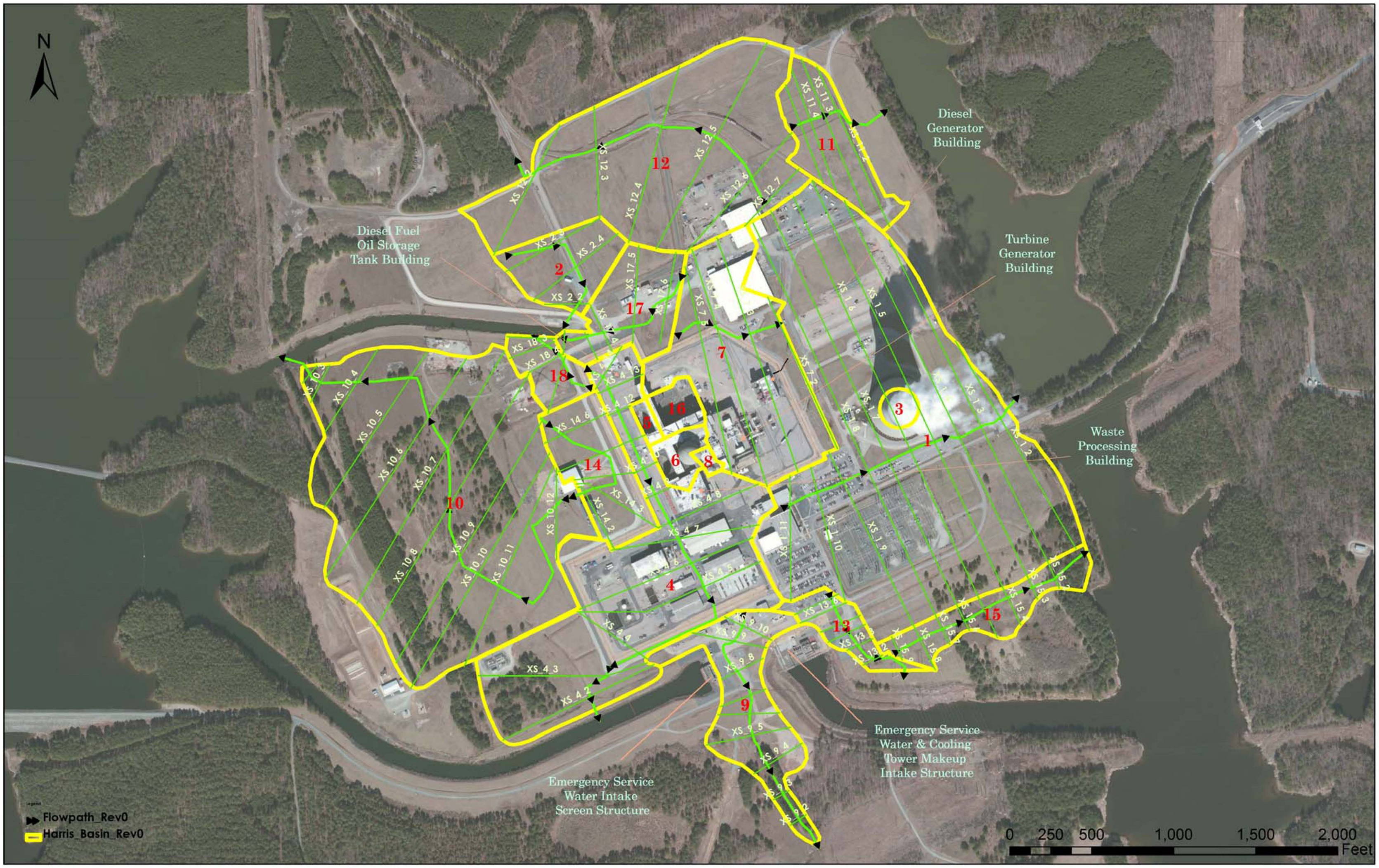

Figure 4. Rainfall Intensity Duration Frequency Curve for the Local PMP at HNP Site

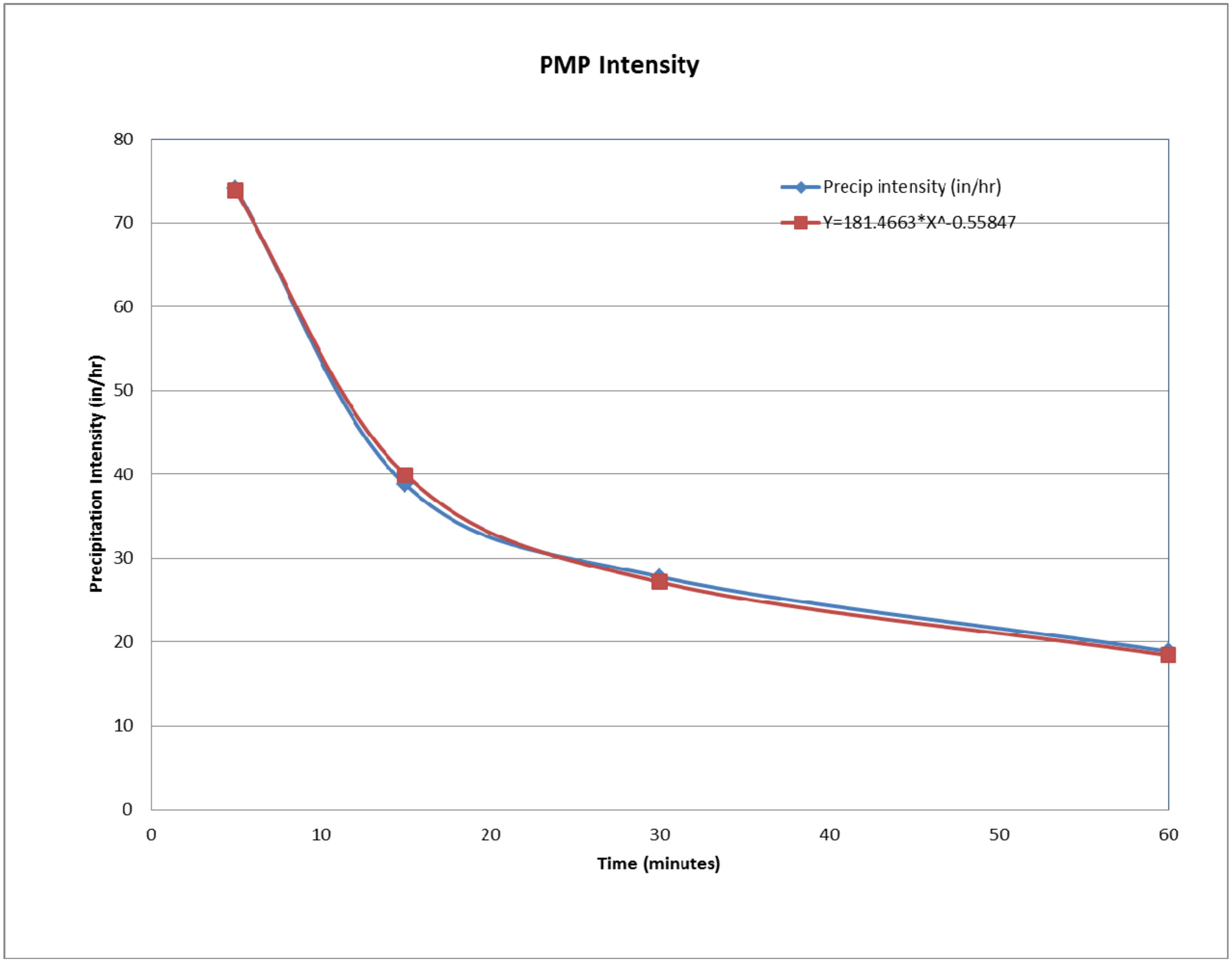

Figure 5. Cross-Sections for the HEC-RAS Model

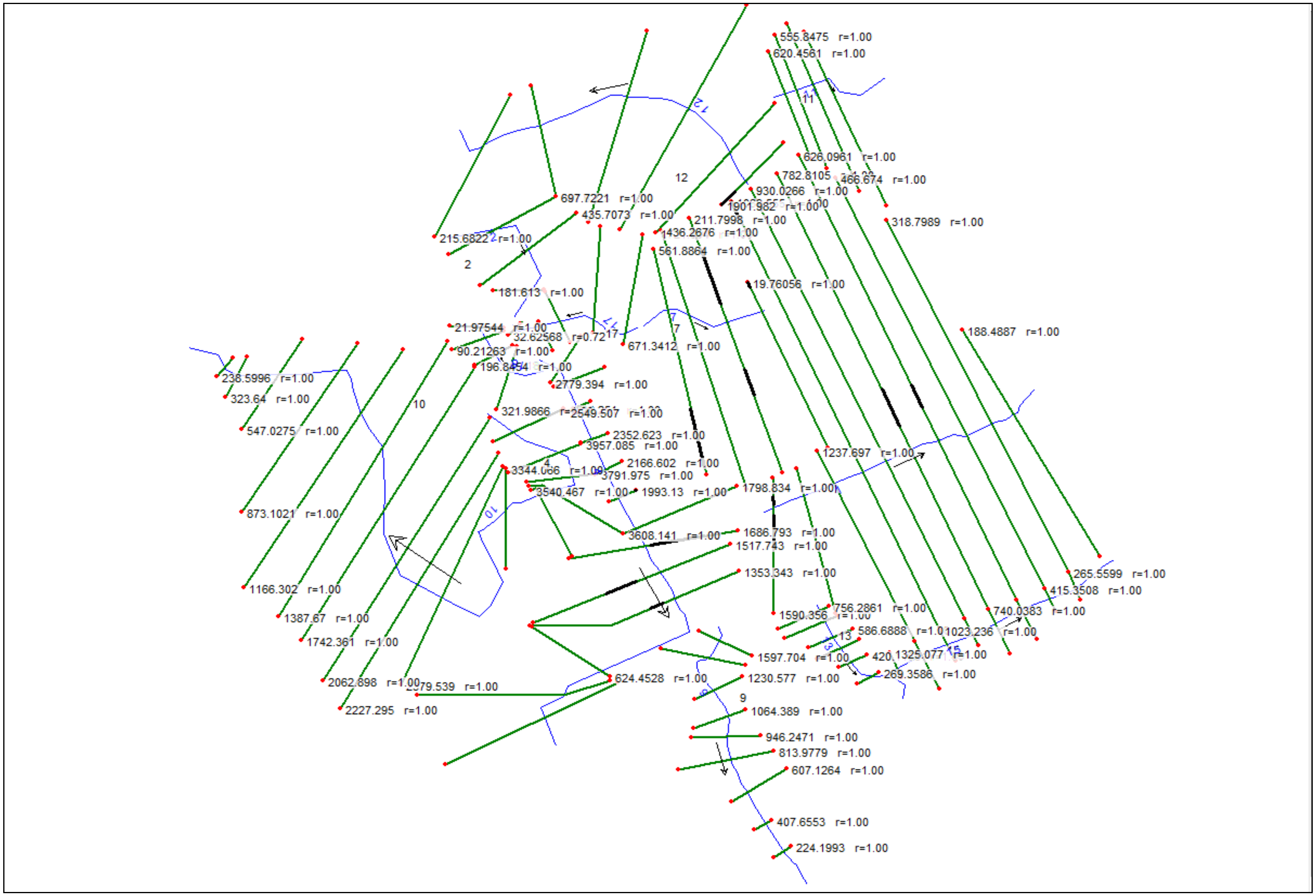

Figure 6. PMF Inflow, Outflow, Storage and Stage Hydrographs for the Auxiliary Reservoir

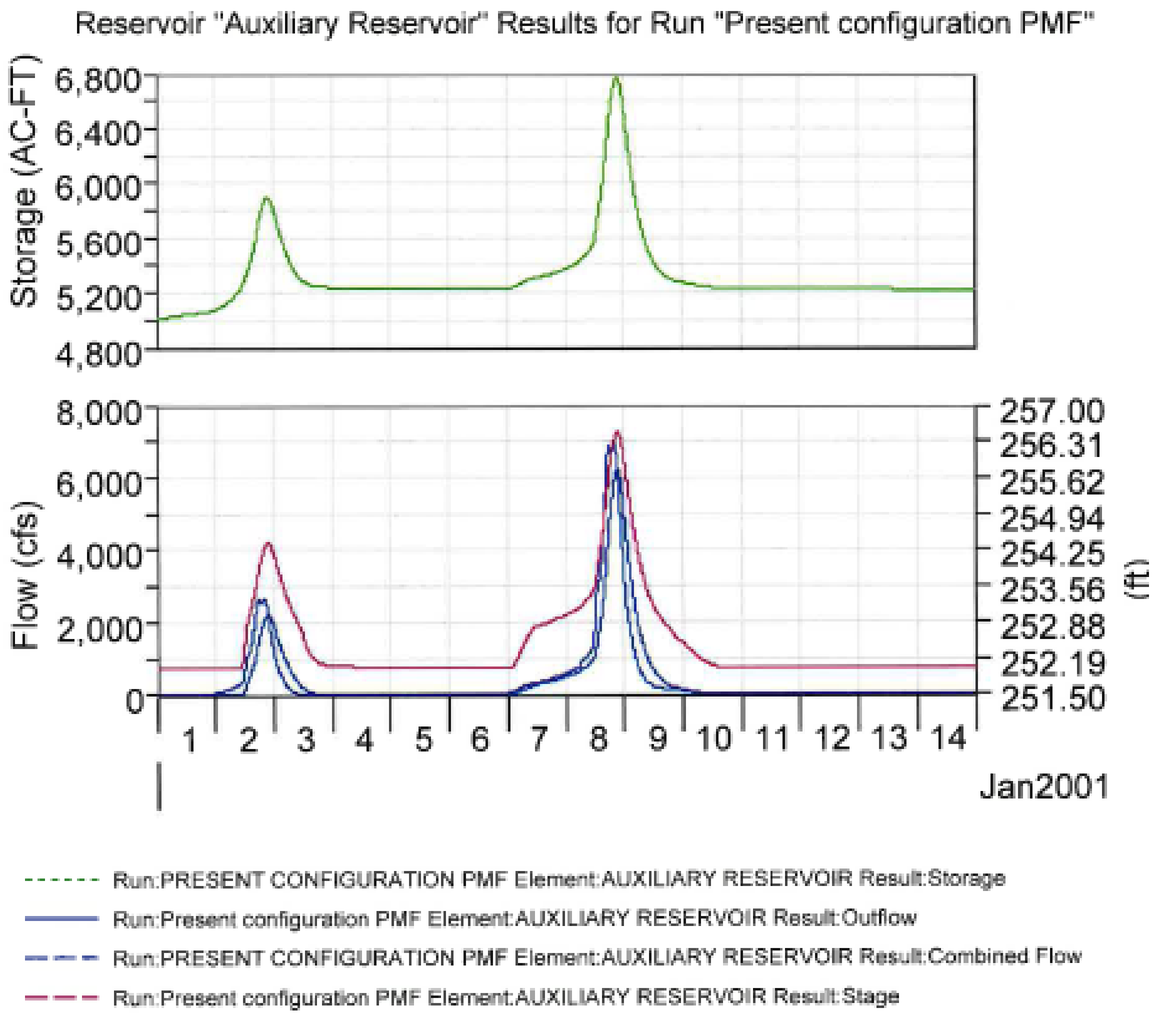

Figure 7. PMF Inflow, Outflow, Storage and Stage Hydrographs for the Main Reservoir

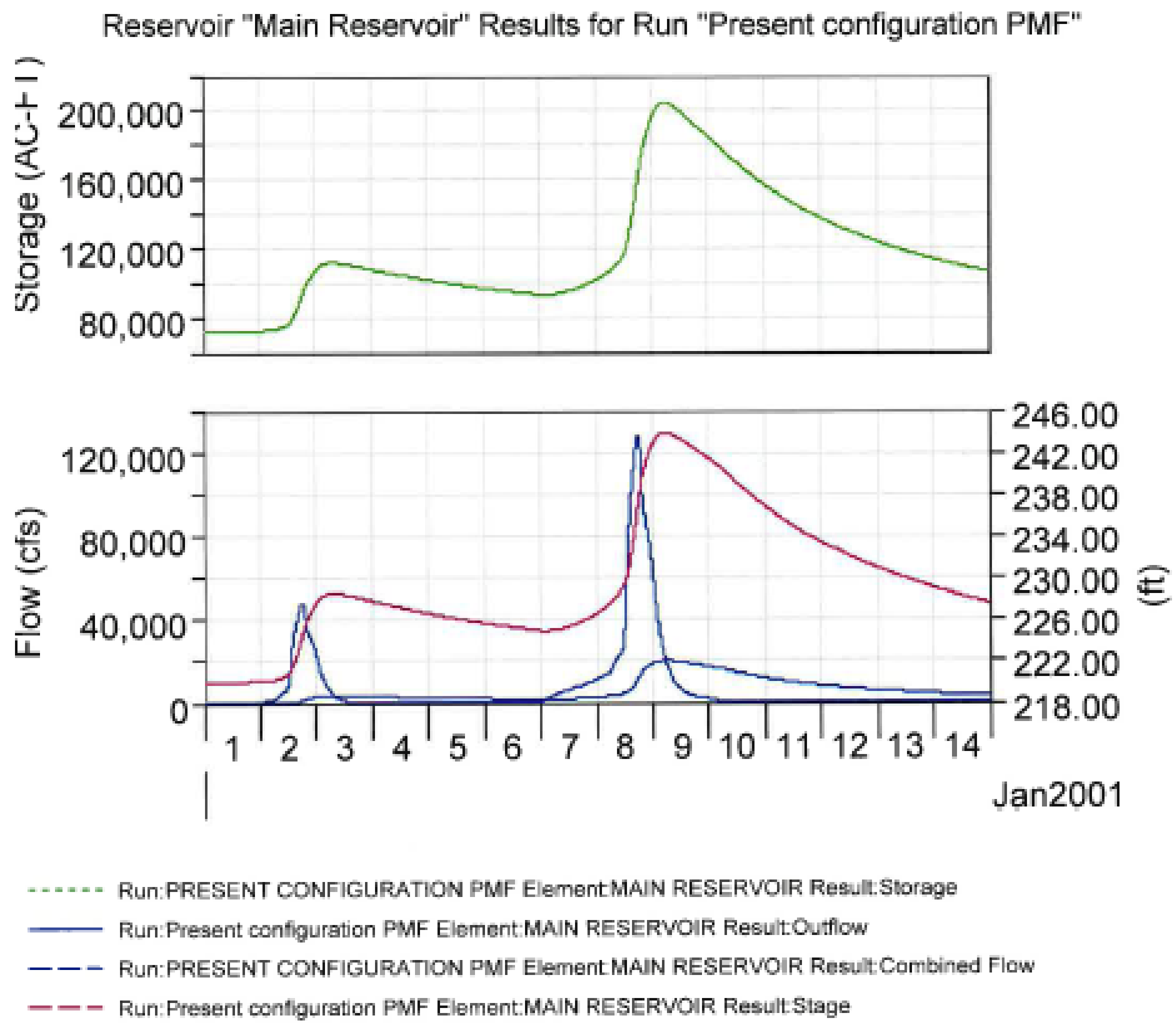

Figure 8. Fetch locations for dams

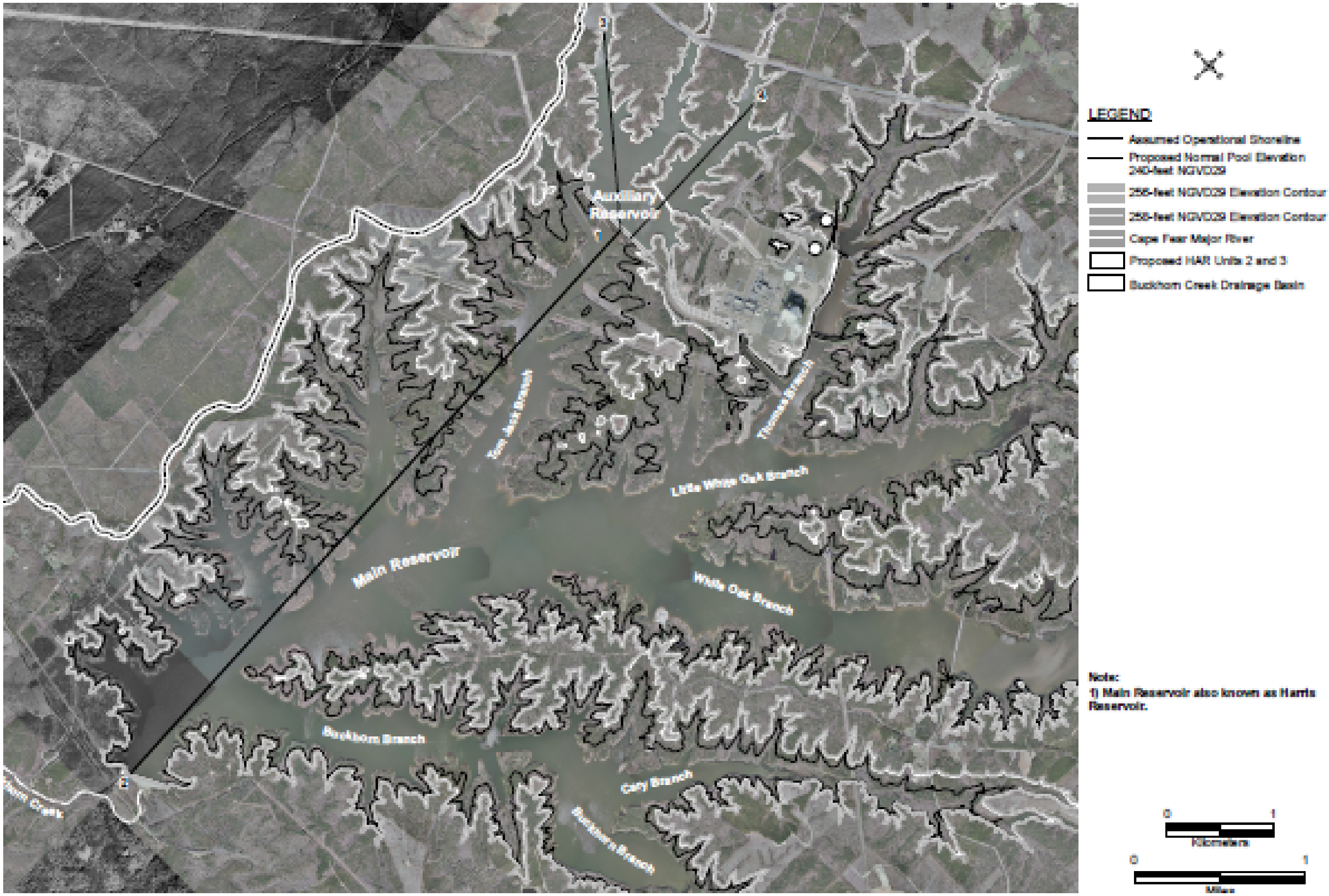

Figure 9. Fetch Locations for the HNP Site