DUKE ENERGY.

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10 CFR 50.54(f)

March 11, 2015
Serial: BSEP 15-0025

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

Subject: Brunswick Steam Electric Plant, Unit Nos. 1 and 2
Renewed Facility Operating License Nos. DPR-71 and DPR-62
Docket Numbers 50-325 and 50-324
Flood Hazard Reevaluation Report, Response to NRC 10 CFR 50.54(f) 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

References:

1. NRC Letter, Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, dated March 12, 2012 (Agencywide Document and Management System (ADAMS) Accession Number ML12053A340)
2. NRC Letter, Prioritization of Response Due Dates for Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Flooding Hazard Reevaluation 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 Number ML12097A509)
3. NRC Letter from David L. Skeen to Joseph E. Pollock (NEI), Trigger Conditions for Performing an Integrated Assessment and Due Date Response, dated December 3, 2012 (ADAMS Accession number ML12326A912)
4. BSEP Letter, Recommendation 2.3 Flooding Walkdown of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, dated November 27, 2012 (ADAMS Accession number ML12340A074)
5. NRC Letter, Supplemental Information to 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 March 1, 2013 (ADAMS Accession Number ML13044A561)

A010
NRR

Ladies and Gentlemen:

On March 12, 2012, the NRC issued Reference 1 to all power reactor licensees and holders of construction permits in active or deferred status. Enclosure 2 of Reference 1 requested that each licensee perform a reevaluation of all appropriate external flooding sources and report the results in accordance with the NRC's prioritization plan (i.e., Reference 2). The due date established for the Brunswick Steam Electric Plant (BSEP) was March 12, 2015. The enclosure to this letter contains the required Flooding Hazard Reevaluation Report for BSEP.

The attached flood hazard reevaluation report describes the approach, methods, and results from the reevaluation of flood hazards at BSEP. The eight (8) flood-causing mechanisms and a combined effect flood identified in Attachment 1 to Enclosure 2 of Reference 1 are described in the report along with the potential effects on BSEP.

The flood hazard reevaluation involved beyond-design-basis conditions, and the report shows that some flood levels are not bounded by the current design basis levels. These reevaluated, non-bounded hazards have been entered into the BSEP Corrective Action Program (CAP). The increased levels are the results of newer methodologies and guidance which are applicable to new reactor reviews and typically exceed the methodologies and guidance used to establish the design basis levels for existing plants. As such, these differences do not represent errors in the current BSEP flooding design or licensing basis. Therefore, consistent with Reference 5, the BSEP flood hazard reevaluation results and the reevaluated non-bounded hazards do not call into question the operability or functionality of any Structure, System, or Component (SSC) and are not reportable pursuant to 10 CFR 50.72 and 10 CFR 50.73.

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

As discussed in Reference 1, the NRC acknowledged that an accident with consequences similar to the Fukushima event is unlikely with nuclear power plants located in the United States. The NRC concluded that continued plant operation does not pose an imminent risk to the public health and safety. The flooding walkdowns of BSEP current flood protection features have been performed and the results were provided by Reference 4. In general, the flood walkdowns verified that the flood protection systems for BSEP are available, functional and implementable, and if necessary, any degraded or nonconforming flood protection features were entered in the BSEP CAP.

Section 6 of the flood hazard reevaluation report provides a discussion of the interim actions that have been taken to address the reevaluated non-bounded hazards relative to the design basis flood levels. These actions make possible the current capability to maintain the plant in a safe condition during postulated flooding events that exceed the design basis flood levels, and as a result, continued plant operation does not impose an imminent risk to the units and public health and safety while completing the Integrated Assessment.

There are no regulatory commitments associated with this submittal.

Should you have any questions concerning this letter or require additional information, please contact Mr. Lee Grzeck, Manager – Regulatory Affairs, at (910) 457-2487.

I declare under penalty of perjury that the foregoing is true and correct. Executed on March 11, 2015.

Sincerely,

William R. Gideon

Enclosure: BSEP Flooding Hazard Reevaluation Report

cc (with enclosures):

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ENCLOSURE

BSEP Flooding Hazard Reevaluation Report

DUKE ENERGY®

FLOOD HAZARD REEVALUATION REPORT

IN RESPONSE TO THE 50.54(f) INFORMATION REQUEST REGARDING SEVERE ACCIDENT MANAGEMENT FOR FUKUSHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1: FLOODING REEVALUATION

for the

BRUNSWICK STEAM ELECTRIC PLANT
UNIT NOS. 1 AND 2
RENEWED LICENSE NOS. DPR-71 & DPR-62

Prepared by:

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1. LIST OF ACRONYMS

°C	degree(s) Celsius (or Centigrade)
°F	degree(s) Fahrenheit
ac	acre(s)
ANS	American Nuclear Society
ANSI	American National Standards Institute
BNP	Brunswick Steam Electric Plant Unit Nos. 1 and 2
CFR	Code of Federal Regulations
CFL	Courant-Friedrichs-Levy
cfs	cubic (foot)feet per second
CVV	Cumbre Vieja Volcano
DEM	Digital Elevation Model
EM	Engineer Manual
ESP	early site permit
ESRI	Environmental Systems Research Institute
FEMA	Federal Emergency Management Agency
FUNWAVE-TVD	FUlly Nonlinear WAVE – Total Variation Diminishing Scheme
fps	feet per second
ft	foot (feet)
GIS	Geographic Information System
GHCN	Global Historical Climatology Network
GHCND	Global Historical Climatology Network Data
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HMR	Hydrometeorological Report
HSG	Hydrologic Soil Group
hr	hour(s)
in	inch
km	kilometer(s)
km^2	square kilometer(s)
Km^3	cubic kilometer(s)
LiDAR	Light Detection and Ranging
LIP	Local Intense Precipitation
m	meter(s)
mi^2	square mile(s)
mi	mile(s)
min	minute(s)
mm	millimeter(s)
mph	miles per hour
MSL	mean sea level
NAVD88	North American Vertical Datum of 1988
NDBC	National Data Buoy Center

NED	National Elevation Dataset
NGVD29	National Geodetic Vertical Datum of 1929
NHWAVE	Non-Hydrostatic WAVE
NID	National Inventory of Dams
NOAA	National Oceanic and Atmospheric Administration
NRC	United States Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
NTHMP	National Tsunami Hazard Mitigation Program
NTTF	Near Term Task Force
OBE	Operating Basis Earthquake
PMF	Probable Maximum Flood
PMH	Probable Maximum Hurricane
PMP	Probable Maximum Precipitation
PMSP	Probable Maximum Snowpack
PMS	Probable Maximum Seiche
PMSS	Probable Maximum Storm Surge
PMT	Probable Maximum Tsunami
PMWS	Probable Maximum Windstorm
RMSE	Root Mean Square Error
SAM	Severe Accident Management
SCS	Soil Conservation Service
SSCs	structures, systems, and components
SSE	Safe Shutdown Earthquake
SWAN	Simulating Waves Nearshore
UFSAR	Updated Final Safety Analysis Report
UHS	Ultimate Heat Sink
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
WSEL	Water Surface Elevation

2. PURPOSE

a. Background

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

On March 12, 2012, the NRC issued an information request pursuant to Title 10 of the Code of Federal Regulations, Section 50.54 (f) (10 CFR 50.54(f) or 50.54(f)) (Reference 2) which included six (6) enclosures:

1. [NTTF] Recommendation 2.1: Seismic

2. [NTTF] Recommendation 2.1: Flooding
3. [NTTF] Recommendation 2.3: Seismic
4. [NTTF] Recommendation 2.3: Flooding
5. [NTTF] Recommendation 9.3: Emergency Preparedness
6. Licensees and Holders of Construction Permits

In Enclosure 2 of Reference 2, the NRC requested that licensees "reevaluate the flooding hazards at their sites against present-day regulatory guidance and methodologies being used for early site permits and combined license reviews."

This report provides the information requested in the March 12, 50.54(f) letter; specifically, the information listed under the "Requested Information" section of Enclosure 2, paragraph 1 ("a" through "e") for the Brunswick Steam Electric Plant Unit Nos. 1 and 2 (BNP).

b. Requested Actions

Per Enclosure 2 of Reference 2,

> Addressees are requested to perform a reevaluation of all appropriate external flooding sources, including the effects from local intense precipitation (LIP) on the site, probable maximum flood (PMF) on streams and rivers, storm surges, seiches, tsunami, and dam failures. It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for early site permits (ESPs) and calculation reviews including current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The requested information will be gathered in Phase 1 of the NRC staff's two phase process to implement Recommendation 2.1, and will be used to identify potential "vulnerabilities" (see definition below).
>
> For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard with the hazard evaluation.
>
> Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them. The scope of the integrated assessment report will include full power operations and other plant configurations that could be susceptible due to the status of the flood protection features. The scope also includes those features of the ultimate heat sinks (UHS) that could be adversely affected by the flood conditions and lead to degradation of the flood protection (the loss of UHS from non-flood associated causes are not included). It is also requested that the integrated assessment address the entire duration of the flood conditions.
>
> *A definition of vulnerability in the context of [Enclosure 2] is as follows: Plant-specific vulnerabilities are those features important to safety that when subject to an increased demand due to the newly calculated hazard evaluation have not been shown to be capable of performing their intended functions.*

c. Requested Information

Per Enclosure 2 of Reference 2, the final report should be provided documenting results, as well as pertinent site information and detailed analysis, and include the following:

a. Site information related to the flood hazard. Relevant structures, systems, and components (SSCs) important to safety and the Ultimate Heat Sink (UHS) are included in the scope of this reevaluation, and pertinent data concerning these SSCs should be included. Other relevant site data includes the following:

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

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

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

iv. Changes to the watershed and local area since license issuance;

v. Current design basis flood protection and pertinent flood mitigation features at the site;

vi. Additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results, etc.)

b. Evaluation of the flood hazard for each flood causing mechanism, based on present-day methodologies and regulatory guidance. Provide an analysis of each flood causing mechanism that may impact the site including local intense precipitation and site drainage, flooding in streams and rivers, dam breaches and failures, storm surge and seiche, tsunami, channel migration or diversion, and combined effects. Mechanisms that are not applicable at the site may be screened-out; however, a justification should be provided. Provide a basis for inputs and assumptions, methodologies and models used including input and output files, and other pertinent data.

c. Comparison of current and reevaluated flood causing mechanisms at the site. Provide an assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood causing mechanism. Include how the findings from Enclosure 2 of the 50.54(f) letter (i.e., Recommendation 2.1 flood hazard reevaluations) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, include how this finding was determined.

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

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

3. SITE INFORMATION

a. Detailed Site Information

Site Location

BNP houses two boiling water nuclear reactors producing a total of 1,870 megawatts of electricity. The plant is located approximately two miles north of Southport, North Carolina. The confluence of the Cape Fear River and the Atlantic Ocean is approximately four miles south of the plant. Cape Fear River provides the plant with a cooling water source. The site location is shown in Figure 3-1.

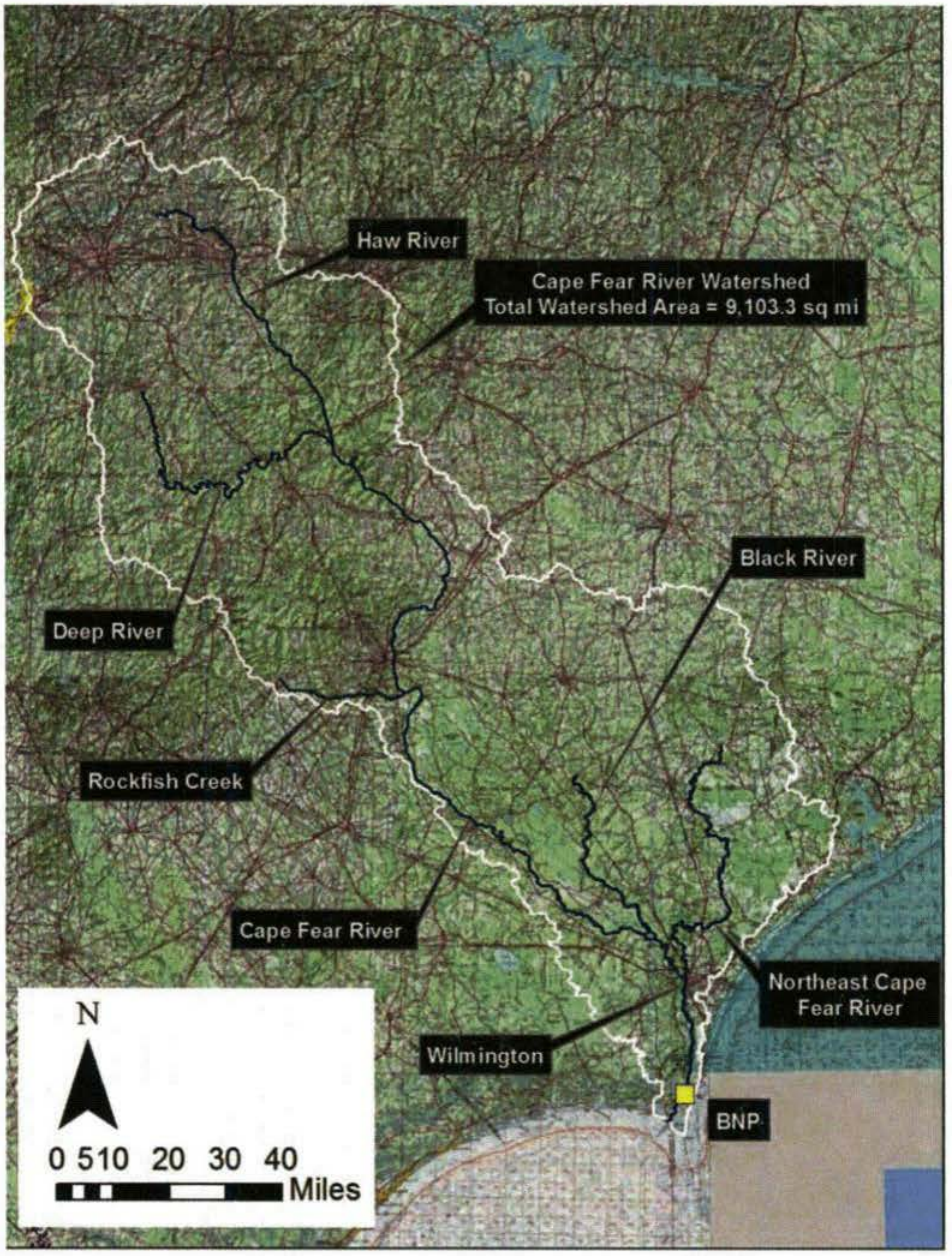

Figure 3-1: Site Location of BNP

Site Layout and Topography

The BNP site occupies an area of approximately 1,200 acres. The area adjacent to the plant is rural, consisting of farmland, swamps, marshes and woodland areas. The power block area, which consists of the reactor and turbine buildings, is predominantly impervious due to buildings, asphalt/concrete roads and walkways. Other major structures include the control building, the radwaste building, the diesel generator building, the nitrogen and off-gas services building, the circulating water intake structure and the service water intake structure. Additional buildings and structures are provided for plant auxiliaries, offices, and

warehouses. The service water intake structure is located to the southeast of the power block and draws cooling water from the intake canal. The nearest USGS stream gage (02105769) is located approximately 45 miles upstream of BNP on the Cape Fear River. The nearest tide gage (8659084) is located in Southport, North Carolina.

Based on the BNP's Updated Final Safety Analysis Report (UFSAR) (Reference 10) and documents and drawings provided by Duke Energy, the grade elevations at the buildings of interest in the National Geodetic Vertical Datum of 1929 (NGVD29 datum)* are:

- Control Building: 23.0 ft (Reference 21)
- Reactor Building: 20.0 ft (Reference 22, 23, 24, 25)
- Diesel Generator Building: 23.0 ft (Reference 26, 27)
- Service Water Building: 23.0 ft (Reference 28)
- Nitrogen and Augmented Off-Gas (AOG) Building: 22.33 ft (Reference 29)
- Radwaste Building: 23.0 ft (Reference 30)

Overall, the site is flat sloping, slightly toward the intake canal on the northeast side of the site and the discharge canal on the southwest side of the site. The topography of the plant is such that the runoff is directed away from the power block by natural drainage and by storm drains toward the intake and discharge canals.

* The UFSAR references most elevations in the Mean Sea Level (MSL) datum. However, Duke Energy verified that the MSL datum referenced in the UFSAR is equal to the NGVD29 datum. Therefore, all elevations in this report will be reported in the NGVD29 datum unless otherwise noted.

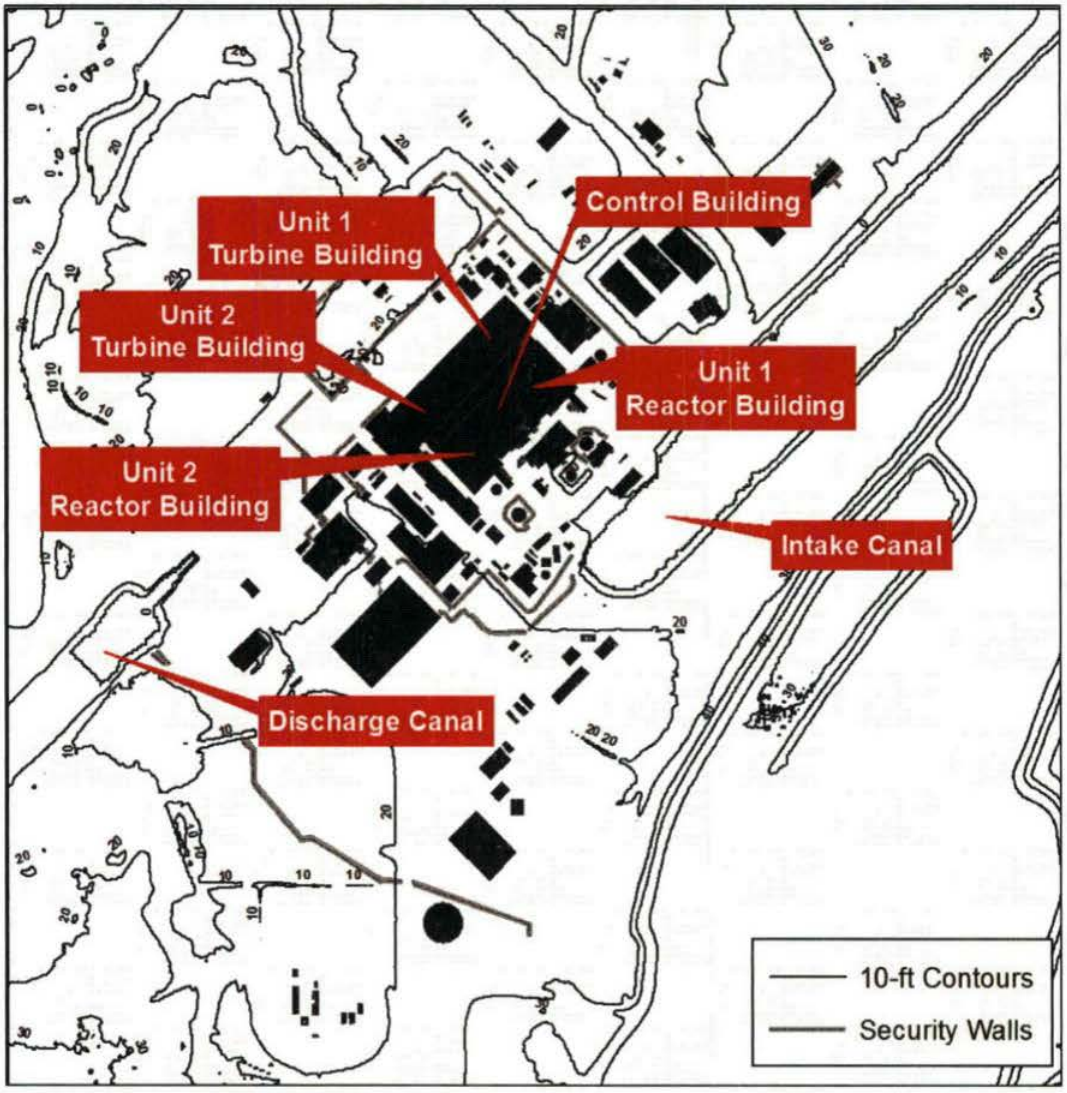

Figure 3-2: Site Layout and Topography of BNP

The dominant surface hydrological feature of the site's region is the Cape Fear River and its tributaries, which discharge to the Atlantic Ocean. The contributing drainage area of the Cape Fear River watershed to BNP is approximately 9,073 square miles. Approximately 6,600 square miles of the watershed are gaged continuously by the U.S. Geological Survey gaging stations. Based on these records, the average daily freshwater discharge rate of the Cape Fear River at the mouth to the Atlantic Ocean is estimated to be between 8,100 and 10,000 cubic feet per second (cfs). A delineation of the Cape Fear River watershed is provided in Figure 3-3 below.

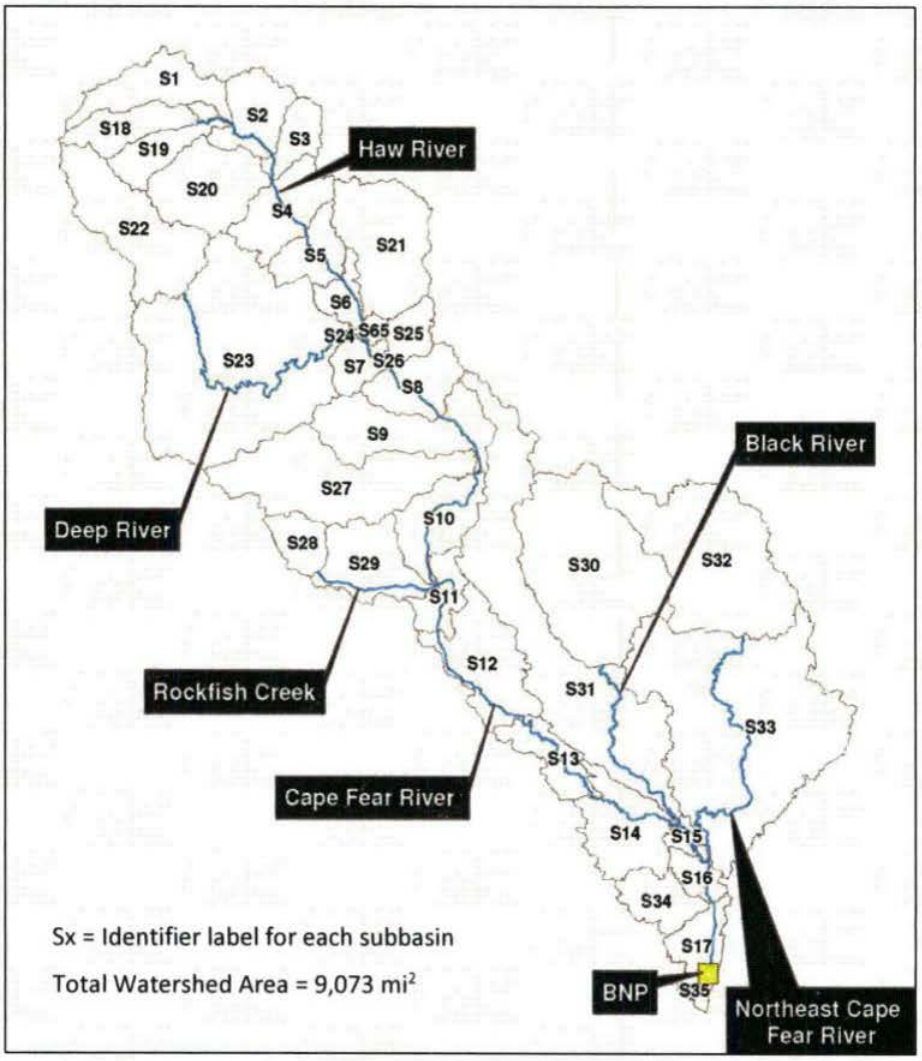

Figure 3-3: Delineation of the Cape Fear River Watershed with Corresponding Sub-Basins

Local drainage features identified in the UFSAR are the Green Swamp in the outlying areas of the plant, which extends approximately 40 miles west and northwest of the site. Drainage from the marshes and swamps is via several small creeks with Walden Creek north and east of the plant, and Dutchman Creek southwest of the plant. Walden Creek is fed by six primary tributaries: Governor's Creek, Fishing Creek, Nigis Creek, White Spring Creek, Nancys Creek, and Gum Long Branch. The primary streams flowing into Dutchman Creek are Jump and Run Creek, Calf Gully Creek, and Frazier Branch. These two creek systems are tidal estuaries with water levels varying ±2 ft at their mouths to ±0.5 ft at the headwaters. The watersheds for the creek systems are primarily wooded and marsh.

Other local drainage features include the Intake and Discharge Canals. The intake canal is approximately 2.5 miles long and portions of the canal intercept Gum Log Branch, which flows into Nancys Creek. The flow from Gum Log Branch upstream of its intersection with the canal, as well as that from several other small, unnamed streams, is carried into the intake canal. In summary, the intake canal has little effect upon the natural drainage of the Walden Creek system, and the water that is intercepted is carried into the canal.

The discharge canal intercepts the tributaries of Dutchman Creek as well as its headwaters. To provide drainage for the water that was normally carried off by these streams into Dutchman Creek, a drainage ditch is provided west of the Discharge Canal extending from the Intracoastal Waterway north to the headwaters of Dutchman Creek. With this configuration, the surface water is intercepted by the ditch and carried to the Intracoastal Waterway approximately 4,000 ft west of the mouth of Dutchman Creek.

SSCs Important to Safety

The safety-related structures at BNP that contain equipment critical for the operation and safety of the plant during a PMH are: the service water intake structure, the reactor buildings, the diesel generator building, and the control building (Reference 10).

b. Current Design Basis Flood Elevations for All Flood Causing Mechanisms

The design basis was reviewed to determine which flood-causing mechanisms are considered in the current design basis flood. Below is a summary of flood-causing mechanisms based on the design basis (Reference 10).

1. Local Intense Precipitation

Local intense precipitation was not addressed in the UFSAR (Reference 10).

2. Flooding in Streams and Rivers

Flooding in streams and rivers was not identified as an applicable flood hazard to BNP due to the PMH being defined as the PMF (Reference 10).

3. Dam Breaches and Failures

Flooding due to seismically induced dam breaches and failures was not identified as an applicable flood hazard to BNP. Sunny-day and overtopping dam breaches and failures were not addressed in the UFSAR (Reference 10).

4. Storm Surge & Seiche

The most severe flood conditions at the site were estimated to be a result of a PMH coinciding with peak local astronomical tides. The open coast stillwater surge was estimated to be 23.8 ft mean low water (MLW) or 21.6 ft NGVD29. The Cape Fear River transmits tidal flows very efficiently and, therefore, is expected to have a peak storm elevation of 25.3 ft MLW (23.3 ft NGVD29) on shore. This peak tide will not reach the BNP site since it will be intercepted by natural ground about one fourth of a mile east of the site near River Road. Therefore, no tide or wave action is expected to reach plant grade from an overland direction. In the intake canal, the stillwater level is expected to reach 22.0 ft NGVD29. From the open coast, the surge water level propagation up the Cape Fear River into the intake canal was evaluated with a resultant level of 24 ft MLW or 22 ft NGVD29. Given the nominal plant grade of 20 ft NGVD29, this results in 2 ft of water depth surrounding the plant during maximum surge conditions (Reference 10).

5. Tsunami

Tsunami flooding was not identified as an applicable flood hazard based on historical tsunamis along the East Cost of very low magnitude and frequency (Reference 10).

6. Ice Induced Flooding

Ice-induced flooding was not identified as an applicable flood hazard to BNP (Reference 10).

7. Combined Effects

The maximum sustained wave on site with 2 ft of water depth was estimated to be 1.6 ft high with a wave runup on a vertical wall of 3.6 ft. Waves greater than 1.6 ft coming from any overland direction would break when reaching the 2-ft depth overland. Therefore, the maximum instantaneous water level on safety-related buildings was estimated to be 25.6 ft NGVD29, which was then arbitrarily raised by 0.5 ft to 26.1 ft NGVD29 for use in the current design basis. Waves moving along the intake canal were determined not to have dynamic effects on the site safety-related structures due to the alignment of the intake canal.

Waves generated or propagated along the intake canal could potentially impact the service water intake structure. These waves were conservatively estimated to be 3.0 ft high. The runup due to these waves results in the maximum instantaneous water level of 28.3 ft NGVD29 at the intake structure.

The maximum values used in the current design basis are 26.1 ft NGVD29 and 28.3 ft NGVD29 for the site safety-related buildings and the intake structure, respectively. Additional detailed analyses have been performed for waves in the intake canal considering various fetch scenarios resulting in lower values. These were, however, not considered in the design basis (Reference 10).

c. Flood Related Changes to the Design Basis and Any Flood Protection Changes (including mitigation) Since License Issuance

Flood-Related Changes to the Design Basis since License Issuance

- Figure 2-25 "Time History of PMH at the Open Coast" in Chapter 2 of the UFSAR (Reference 10) has been revised due to an error in previous versions.
- The roll-up door at the diesel generator building has a leakage rate of 15 gpm. As a defense in depth measure, a partial height temporary metal plate is installed in front of the roll-up door during storm preparations to ensure that the leakage rate does not exceed 15 gpm (Reference 10).

Flood Protection Changes since License Issuance

- Review of personnel flood doors for safety related buildings noted some flood protection basis that were not required. For clarity, the UFSAR was updated to reflect correct door leakage requirements (Reference 45).
- A drain plug was installed in the deluge pit adjacent to the Diesel Generator Building to improve Diesel Generator Building basement flood protection (Reference 46).
- BNP issued a technical report for external event protection features to include a list of all flood protective features, document building leakage values assumed, and sump pump and check valve performance required (Reference 47).

d. Changes to Watershed and Local Area since License Issuance

The Upper Cape Fear River watershed is mainly farmland and forest and, therefore, it is reasonable to expect that no significant changes occurred since the license issuance. The Lower Cape Fear River watershed has likely experienced increase in impervious cover due to population growth and urbanization. However, the impervious cover remained fairly constant from 1996 to 2006 (Reference 36).

Maintenance dredging and improvements to the Cape Fear River channel occur regularly and are ongoing since license renewal. The comprehensive Wilmington Harbor-96 Act Project consists of channel improvements extending from the ocean entrance upstream to just above the Northeast Cape Fear River railroad bridge in Wilmington. Construction under the Wilmington Harbor-96 Act was initiated in August 1999 (Reference 44). As of April 2013, improvements that have been completed consist of deepening the

ocean bar and entrance channels from the authorized depth of 40 feet to 44 feet; deepening the authorized 38-foot project to 42 feet including the anchorage basin through the Cape Fear Memorial Bridge; widening the existing 400-foot wide channel to 600 feet over a total length of 6.2 miles including Lower and Upper Midnight and Lower Lilliput reaches; widening five turns and bends by 100 to 200 feet; and widening the Fourth East Jetty channel to 500 feet over a total length of 1.5 miles (Reference 44).

e. Current Design Basis Flood Protection and Pertinent Flood Mitigation Features

Seismic Class I safety-related structures, which include the Control Building (CB), Diesel Generator (DG) Building, Reactor Buildings 1 and 2 (RB1 and RB2), and Service Water Intake (SW) Building, are protected up to 22 ft NGVD29. Personnel and equipment access doors were provided with sills above the 22 ft NGVD29, or alternately were equipped with positive seals and closure devices where the sills were below 22 ft NGVD29. These structures and the equipment they house are also protected from wave action caused by the PMH (Reference 10). Incorporated passive features include wall penetrations seals, floor drains, roof drains, and manhole covers. Incorporated active features include credited water-tight doors, sump pumps, and check valves that prevent flood infiltration (Reference 11). Doors and door frames were designed to be complete with hardware, weather stripping, and gaskets to limit the in-leakage from the PMH to 5 gpm for personnel doors and 30 gpm for track doors (Reference 10). A partial height temporary metal plate is to be installed in front of the roll-up door at the diesel generator building to ensure that the leakage rate does not exceed 15 gpm.

Advance notice is expected to be available to site management and preparations for a design flooding event can be staged, as appropriate, before a threat is observed. Preparations for a flooding event are directed by procedure 0AI-68, "Brunswick Nuclear Plant Response to Severe Weather Warnings," procedure 0PEP-02.6, "Severe Weather," and abnormal operating procedure 0AOP-13.0, "Operation during Hurricane, Flood Conditions, or Earthquake." A meteorological service provider is contracted to notify BNP of National Oceanic and Atmospheric Administration (NOAA) hurricane watch and warning declarations affecting the plant. Additional notifications are made to provide the initiating criteria which trigger the procedures and activities for extreme hurricanes and are not relieved until official notification from NOAA data that the hurricane threat has passed (Reference 11).

4. SUMMARY OF FLOOD HAZARD REEVALUATION

The following is a summary evaluation of the flood hazard for each flood causing mechanism described in NUREG/CR-7046. These evaluations are based on acceptable industry standard methodologies and regulatory guidance. An analysis to identify each flood causing mechanism that may impact the site was performed including local intense precipitation and site drainage, flooding in streams and rivers, dam breaches and failures, channel migration or diversion, storm surge, seiche, tsunami, and combined effects.

a. Local Intense Precipitation

The local intense precipitation (LIP) is a measure of the extreme precipitation (high intensity/short duration) at a given location. Generally, the amount of extreme precipitation decreases with increasing duration and increasing area. NUREG/CR-7046 (Reference 3) specifies that the LIP should be equivalent to the 1-hr, 2.56-km^2 (1-mi^2) PMP at the plant site.

The LIP event was evaluated to determine the associated flooding elevations and velocities assuming the active and passive drainage features are non-functioning. The LIP evaluation was performed in accordance with NUREG/CR-7046.

The model was created with boundaries that encompass the local site drainage. The runoff caused by the LIP event was estimated using FLO-2D software (Reference 12). The software uses shallow water equations to route stormwater throughout the site. FLO-2D depicts site topography using a digital elevation model (DEM) to characterize grading, slopes, drainage divides, and low areas of the site. The methodology used within the FLO-2D software included the rainfall function and the levee function to incorporate site security features which could impact the natural drainage characteristics of the site. The DEM was produced from LiDAR data and supplemented with site drawings. Exterior door elevations and the surrounding areas of the safety related structures were based on a topographic survey.

The FLO-2D model uses Manning's n-values to characterize the site's surface roughness and calculate effects on flow depths and velocities. Manning's Roughness Coefficients (n-values) were based on the site's land cover. Per NUREG/CR-7046 recommendations, runoff losses were ignored during the LIP event in order to maximize the water elevation on site from the event. Furthermore, the majority of the site drainage areas consist of buildings/structures as well as paved areas and Hydrologic Soil Groups (HSG) C & D and, therefore, infiltration losses for these areas would be minimal. Ignoring runoff losses is considered to be the most conservative assumption. Only overland flow and open channel systems were modeled and considered in the LIP flooding analysis.

The 1-hr PMP event distribution was developed using HMR 52. The total PMP depth per square mile for the 1-hr event was extrapolated from the PMP depth contour map provided in Figure 24 of HMR 52 (Reference 13). The distribution of the 1-hr PMP was developed for the 5-, 15-, and 30-minute time intervals, with the 60-minute interval being the 1-hr PMP depth. The 1-hr PMP distribution is provided in Table 4-1 and Figure 4-1 below. The 1-hr PMP was modeled in FLO-2D to calculate the subsequent site flooding.

Table 4-1: 1-mi²/1-hr PMP Distribution

Time (minutes)	Percent Total PMP (%)	Cumulative Depth (inches)	Reference
0	0%	0.00	N/A
5	32.45%	6.19	Figure 36 of the HMR-52 manual
15	50.92%	9.72	Figure 37 of the HMR-52 manual
30	73.79%	14.08	Figure 38 of the HMR-52 manual
60	100%	19.08	Figure 24 of the HMR-52 manual

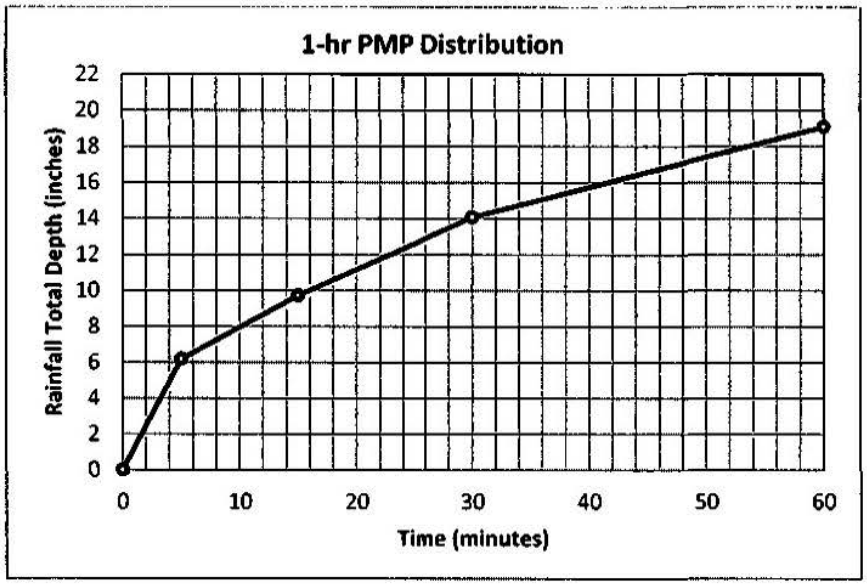

Figure 4-1: 1-mi²/1-hr PMP Distribution

To determine the flooding elevations associated with the LIP, the 1-mi²/ 1-hr storm was applied evenly across the site, and the model was allowed to run for 24 hours to ensure that only the areas of static ponding would remain inundated. The LIP evaluation was conducted independently of external high-water events (i.e., the LIP event was assumed to have occurred non-coincident to a river flood). Therefore, backwater or tailwater was not considered.

The LIP flooding evaluation (per Case 3 assumptions of NUREG/CR-7046, Section 3.2) estimated the maximum flooding depths, water surface elevations, velocities, resultant static loads, and resultant impact loads that could be expected during the LIP event, assuming the surface drainage system and storm sewer system are fully blocked.

A summary of the results of the analyses for buildings containing SSCs is provided in Table 4-2. A summary of the results of the analyses for buildings not containing SSCs is provided in Table 4-3. The location of the doors of interest is shown in Figure 4-2.

Table 4-2: LIP Flooding Results at Buildings Containing SSCs

Door ID	Building	Bottom of Door Elevation	Max. Water Surface Elevation	Max. Flooding Depth above Door Bottom	Flooding Duration above Door Bottom	Max. Velocity	Max. Resultant Impact Load	Max. Resultant Static Load
		ft (NGVD 29)		ft	hr	ft/sec	lb/ft	lb/ft
Door D-4	Diesel Generator Building	20/23*	20.88	0.88/0	7.8/0	0.88	2.15	48.99
Door D-5		23	21.53	0	0	0.6	0.15	0.43
Door D-6		23	20.99	0	0	0.84	1.04	29.43
Door D-7*		20/23*	21.02	1.02/0	7.5/0	0.56	26.23	78.2
Door D-8		23	22.49	0	0	0.84	0.68	0.52
Door D-2	Reactor Building	20	20.79	0.79	3.6	0.55	0.97	19.38
Door D-3		20	21.07	1.07	6.3	1.01	3.56	39.77
Door D-13	Service Water Intake Building	23	20.61	0	0	0.34	0.94	0.62
Door D-14		23	19.87	0	0	5.05	61.84	29.86

*Bottom of door elevations for D-4 and D-7 locations indicate the exterior and interior elevations of the respective doors. The interior doors at elevation 23 ft NGVD29 are credited as flood protection features.

Table 4-3: LIP Flooding Results at Buildings Not Containing SSCs

Door ID	Building	Bottom of Door Elevation	Max. Water Surface Elevation	Max. Flooding Depth above Door Bottom	Flooding Duration above Door Bottom	Max. Velocity	Max. Resultant Impact Load	Max. Resultant Static Load
		ft (NGVD 29)		ft	hr	ft/sec	lb/ft	lb/ft
Door D-9	AOG Building**	22.33	20.71	0	0	0.78	11.03	42.46
Door D-10		22.33	20.64	0	0	0.79	10.57	21.36
Door D-11		22.33	20.63	0	0	0.44	10.42	21
Door D-12		22.33	20.72	0	0	0.5	10.15	34.08
Door D-15	Flex Storage Building	30.17	28.99	0	0	0.41	4.19	11.13
Door D-16		30.17	29.03	0	0	0.34	3.62	7.33
Door D-1	Radwaste Building	23	21	0	0	0.72	71.19	123.98
Door D-17	Turbine Building	20	21.45	1.45	6.6	0.68	52.15	76.86
Door D-18		20	21.45	1.45	6.4	0.88	51.11	74.24
Door D-19		20	21.43	1.43	6.8	0.54	62.56	75.61
Door D-20		20	21.44	1.44	6.7	0.46	33.21	79.12
Door D-21		20	21.56	1.56	5.8	1.25	88.71	116.71
Door D-22		20	21.45	1.45	5.8	2.64	56.85	70.98
Door D-23		20	21.45	1.45	5.8	2.64	56.85	70.98

Door ID	Building	Bottom of Door Elevation	Max. Water Surface Elevation	Max. Flooding Depth above Door Bottom	Flooding Duration above Door Bottom	Max. Velocity	Max. Resultant Impact Load	Max. Resultant Static Load
Door D-24	Turbine Building	20	21.69	1.69	10.8	0.9	59.22	103.86
Door D-25		20	21.69	1.69	6.2	0.54	58.58	101.37
Door D-26		20	21.65	1.65	4.7	0.55	53.68	85.36

** AOG Building is a safety-related structure not containing equipment requiring flood protection

Figure 4-2: Location of Doors of Interest

b. Flooding in Streams and Rivers

The PMF in the Cape Fear River and Nancys Creek adjoining the site was determined by applying the PMP to the flooding source drainage basins in which the site is located. The PMF is based on a transformation of PMP rainfall on a watershed to flood flow. The PMP is a deterministic estimate of the theoretical maximum depth of precipitation that can occur at a time of year of a specified area. A rainfall-to-runoff transformation function, as well as runoff characteristics, based on the topographic and drainage system network characteristics and watershed properties are needed to appropriately develop the PMF hydrograph. The PMF hydrograph is a time history of the discharge and serves as the input parameter for the hydraulic model which develops the flow characteristics including flood flow and elevation.

Cape Fear River

The precipitation driven PMF discharge was determined from the evaluation of three combined-effect flood alternatives defined by NUREG/CR-7046, Appendix H.1 *Floods Caused by Precipitation Events* (Reference 3). A deterministic HEC-HMS model was developed and used to evaluate the combined-effect floods under Alternative 1. Due to the geographic location of BNP, a qualitative assessment of the Probable Maximum Snowpack (PMSP) with the snow-season PMP was performed to evaluate whether the extreme rain on snow events of Alternative 2 and Alternative 3 are bound by the Alternative 1 event. The HEC-HMS model was based on the best available geospatial data and was calibrated to two severe storm events, March 1998 and September 1999. The March 1998 storm was used to calibrate the model in the upper and middle portions of the Cape Fear River watershed where it had a greater impact. The September 1999 storm was used to calibrate the model for the lower portion of the Cape Fear River watershed. The April 2003 storm was selected to verify the combined model because it was the largest storm event and generated the highest streamflow at the most downstream gage on the Cape Fear River since the September 1999 storm.

The three precipitation driven combined-effect flood alternatives evaluated in this report are:

- Alternative 1 – Combination of:
  - Mean monthly base flow;
  - Median soil moisture;
  - Antecedent or subsequent rain: the lesser of (1) rainfall equal to 40 percent of PMP and (2) a 500-year rainfall; and
  - PMP.
- Alternative 2 – Combination of:
  - Mean monthly base flow;
  - Probable maximum snowpack; and
  - A 100-year, snow-season rainfall.
- Alternative 3 – Combination of:
  - Mean monthly base flow;
  - A 100-year snowpack; and
  - Snow-season PMP.

Please note that the effects of waves induced by 2-year wind speed applied along the critical direction will be discussed in Section 4.i Combined Effect Flood for the governing Alternative. The evaluation was performed consistent with the following guidance documents:

- NRC Office of Standards Development, Regulatory Guide: RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977 (Reference 35).
- American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8-1992) (Reference 4).
- NUREG/CR-7046 "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," publication date, November 2011 (Reference 3).

Hydrologic parameters, inputs, and assumptions were selected based on recommendations provided in NUREG/CR-7046. Hydrologic parameters, inputs, and assumptions based on Federal regulatory guidance from other agencies (i.e., NRCS, USGS, and USACE), previous studies, and engineering judgment were used when NUREG/CR-7046 did not provide guidance.

The purpose of this evaluation was to determine the governing riverine PMF peak discharge at BNP. The PMF alternative producing the greatest calculated peak discharge of the three alternatives evaluated (listed above) is identified as governing. For clarity, evaluation of hydrologic dam failures is provided in Section 4.c.

The following sub sections describe the inputs, assumptions, methodology, and results of the Cape Fear River analysis.

Dam and Reservoir Screening

Section 5.5 of ANS 2.8 states: "All dams above the plant site shall be considered for potential failure, but some may be eliminated from further consideration because of low differential head, small volume, distance from plant site, and major intervening natural or reservoir detention capacity (Reference 4)." The dams in the Cape Fear River Watershed upstream of BNP were evaluated in accordance with Guidance for Assessment of Flooding Hazards Due to Dam Failure (JLD-ISG-2013-01) (Reference 1). Dams identified to have a potential impact at the site were considered to be critical and were modeled individually in estimation of the PMF at the site.

According to the North Carolina Division of Energy, Mineral and Land Resources (NCDEMLR) database (Reference 6) there are 1,290 dams in the Cape Fear River watershed upstream of BNP. The database was used to obtain their location and characteristics. Data gaps were closed with information from the USACE National Inventory of Dams (NID) database (Reference 8). The resulting dataset was used for the dam screening analysis to remove inconsequential dams from consideration, in accordance with Section 3.1 of JLD-ISG-2013-01 (Reference 1). Inconsequential dams are defined as dams having minimal or no adverse failure consequences beyond the owner's property.

According to NCDEMLR, a "Low" Hazard Classification dam must meet the following criteria:

- Interruption of road service, low volume roads: less than 25 vehicles per day.
- Economic damage: less than $ 30,000.

Based on this criterion, "Low" Hazard Classification dams were removed from consideration and 407 dams, characterized by "High" or "Intermediate" Hazard Classification, were screened using the volume method described JLD-ISG-2013-01 (Reference 1) to identify potentially critical dams. The volume method assumes the total upstream dam storage volume is simultaneously transferred to the site without attenuation. Additionally, the method assumes the only available floodplain storage is between the lowest safety-related structure finished floor and the 500-year water surface elevation.

As part of the screening process, the dams were ranked by storage volumes. The storage volumes were cumulatively added from the lowest to the highest rank. Once the cumulative storage volume exceeded the volume between the lowest safety-related structure finished floor and the 500-year flood stage, any dams exceeding the volume and ranked higher were identified as potentially critical. In this case, only Jordan Lake Dam was identified as potentially critical and modeled individually and used in the calibration process. In addition to Jordan Lake Dam, Shearon Harris Dam was also modeled individually since it is owned by Duke Energy and stage-storage and stage-discharge information is readily available. The remaining dams were identified as non-critical and modeled as hypothetical clusters of dams in the dam breach analysis.

Calibrated Hydrologic Model

The USACE Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) software, version 4.0 (Reference 16) was used to simulate the hydrologic processes of the watershed and to estimate the PMF

peak flow rates during a PMP event. The HEC-HMS model was calibrated to recorded stream gage data during and following the March 1998 and September 1999 storm events.

Preliminary values of model parameters (loss, rainfall-runoff transformation, flow routing, and baseflow parameters) were manually adjusted during the calibration to reproduce the observed streamflow data. In particular, the model representing the March 1998 storm event was calibrated against the streamflow data at USGS gage Cape Fear River At Lock #1 Near Kelly, NC (Gage 02105769, as shown in Figure 4-3) and all gages upstream of it, because the storm mainly affected the upper and middle portions of the Cape Fear River watershed, producing the largest historical peak flow at Gage 02105769. The calibration of the September 1999 model focused on the streamflow data in the Black River and Northeast Cape Fear River subwatersheds (Gages 02106500, 02108000, and 02105769, as shown in Figure 4-3), because the storm mainly affected the lower portion of the Cape Fear River watershed, producing the highest historic streamflows in the Black River and Northeast Cape Fear River subwatersheds. Once the HEC-HMS model was calibrated to each of the two storm events, all calibrated parameters were reviewed and combined to establish a set of parameters for a single combined model, such that a watershed-wide response to a significant storm event is conservatively represented. In particular, the parameters calibrated for the 1998 storm are used in the combined model for the portion of the Cape Fear River watershed upstream of Gage 02105769. For the rest of the watershed, the parameters calibrated for the 1999 storm are used.

Following the development of the final combined model, both the March 1998 and September 1999 storms, as well as the April 2003 storm, were re-evaluated in order to verify that the combined model provides a conservative yet realistic estimate of peak flows in the Cape Fear River watershed

Stream flow data from eleven (11) stream gages within the Cape Fear River Watershed was used in the calibration process. Stream flow gage locations are presented in Figure 4-3.

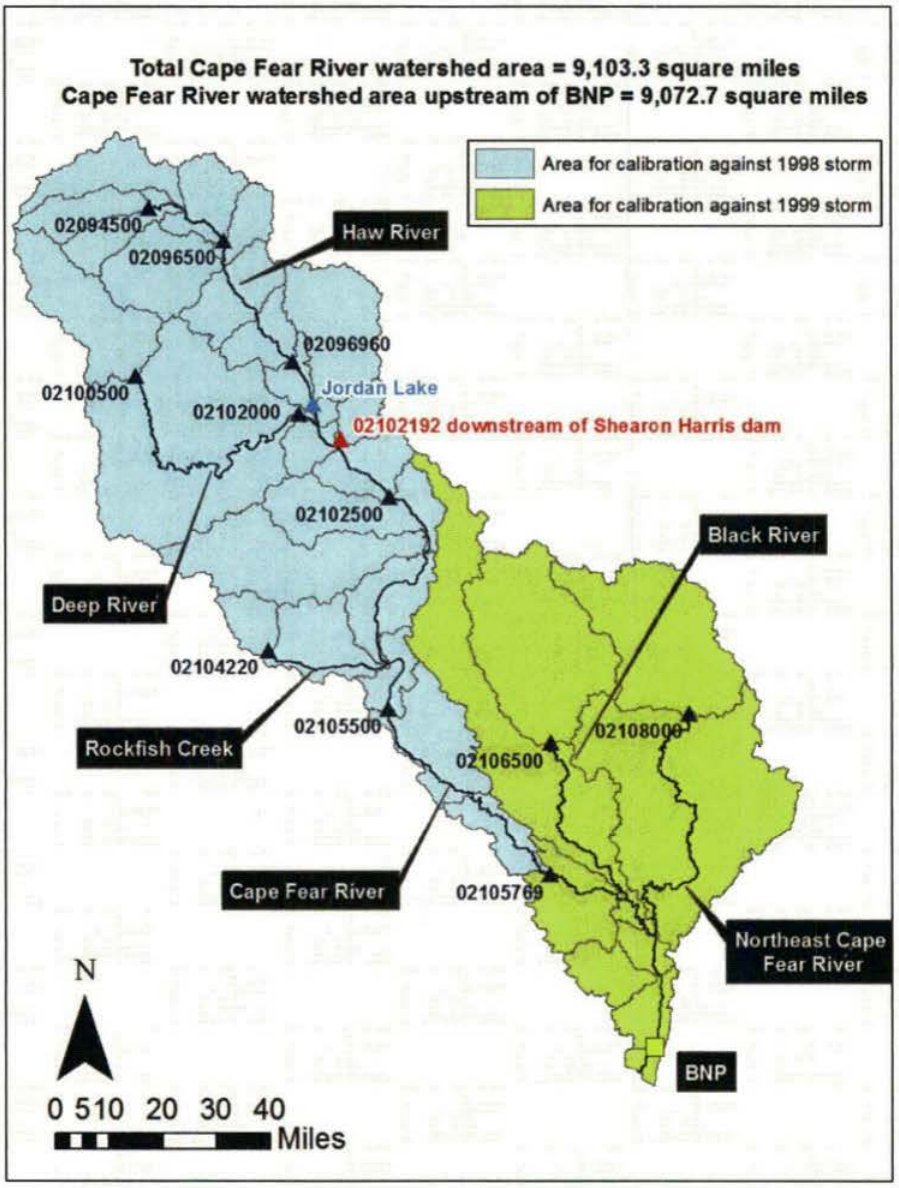

Figure 4-3: Streamflow Gage Locations

The Cape Fear River Watershed was divided into 36 sub-basins, ranging in area from approximately 3 square miles to 1,130 square miles. The sub-basins were mostly delineated at each USGS stream gage, at the confluence of major tributaries, and "critical" dams.

The initial lag time inputs for hydrograph routing were based on the NRCS Technical Release 55 (TR-55) methodology using the segmental velocity approach along the longest flow paths. The Muskingum Method was used to estimate routing in the channel reaches; the Muskingum K (representing the travel time through each reach) and X (representing the storage factor) parameters were adjusted during the calibration process to reflect the watershed response during the calibration event.

The initial and constant loss method was used to calculate the subbasin losses. The initial loss specifies the amount of precipitation loss in the watershed before surface runoff begins. The constant loss rate is the estimation of the rate of infiltration that occurs once the initial loss is satisfied. The initial losses were calibrated to reproduce the volume and shape of the rising limb of the observed hydrograph. The constant loss rates were calibrated within the range limited to those that could be experienced in the specific subbasin using saturated hydraulic conductivity associated with the corresponding Hydrologic Soils Group (HSG).

The recession method was selected for the baseflow methodology. The recession method is designed to approximate the typical behavior observed in watersheds when channel flow recedes exponentially after an event. The initial baseflow at the beginning of the simulation was based on observed flow rate data for the simulated storm events. The values of recession constant (describing the rate at which baseflow recedes between storm events) and ratio (determining how to reset the baseflow during a storm event) were calibrated to observed values.

The calibration of the individual storm event models (1998 and 1999) was performed to reproduce the observed USGS stream flow hydrographs at the different streamflow gages. The calibration results for observed volume and peak flow were generally within the 5% to 10% range, which according to Donigian et al. (Reference 9) indicates good calibration. Furthermore, the performance of the combined model during the 1998, 1999, and 2003 storm events indicates that the model conservatively estimates flows in the Cape Fear River watershed during an extreme event. Therefore, the combined model was used to evaluate extreme precipitation event alternatives in support of the flooding hazard re-evaluation.

Alternative 1 PMF Evaluation

Per NUREG/CR-7046, Appendix H.1 *Floods Caused by Precipitation Events* (Reference 3), the Alternative 1 combined-effect precipitation was evaluated to include:

- Mean monthly base flow;
- Median soil moisture;
- Antecedent or subsequent rain: the lesser of (1) rainfall equal to 40 percent of PMP and (2) a 500-year rainfall; and
- PMP.

As described previously, the evaluation of effects of waves induced by 2-year wind speed is discussed in Section 4.i Combined Effect Flood. Alternative 1 was evaluated using the calibrated hydrologic model and derived precipitation inputs to reflect the combination of mean monthly base flow, antecedent rain, and the PMP event, as well as median soil moisture conditions.

Baseflow was estimated by averaging the mean monthly discharges obtained from USGS for each month of the year for the entire period of record at the USGS streamflow gages and dividing by the drainage area, to obtain values of baseflow per unit drainage area. The March average flow resulted in the highest baseflow throughout the year and was used to represent the average monthly baseflow. The values of baseflow per unit drainage area reported in Table 4-3 were applied to the Cape Fear River watershed subbasins according to their location in the watershed. A constant value of baseflow was used for each subbasin during the PMP simulation.

Table 4-4: Baseflow per Unit Drainage Area

USGS Streamflow Gage Number	USGS Streamflow Gage Name	Reach	HEC-HMS ID	Drainage Area (sq mi)	March Mean Monthly Discharge (cfs)	Baseflow Per Unit Drainage Area (cfs/mi²)
02094500	REEDY FORK NEAR GIBSONVILLE, NC	Reedy Fork	J18	131.0	163	1.24
02096500	HAW RIVER AT HAW RIVER, NC	Haw River	J2	606.0	1,000	1.65
02096960	HAW RIVER NEAR BYNUM, NC	Haw River	J5	1,275.0	2,320	1.82
02100500	DEEP RIVER AT RAMSEUR, NC	Deep River	J22	349.0	626	1.79
02102000	DEEP RIVER AT MONCURE, NC	Deep River	J23	1,434.0	2,750	1.92
02102192	BUCKHORN CREEK NR CORINTH, NC	Buckhorn Creek	Shearon Harris Reservoir	76.3	112	1.47
02102500	CAPE FEAR RIVER AT LILLINGTON, NC	Cape Fear River	J8	3,464.0	6,430	1.86
02104220	ROCKFISH CREEK AT RAEFORD, NC	Rockfish Creek	J28	93.1	137	1.47
02105500	CAPE FEAR R AT WILM O HUSKE LOCK NR TARHEEL, NC	Cape Fear River	J11	4,852.0	8,720	1.80
02105769	CAPE FEAR R AT LOCK #1 NR KELLY, NC	Cape Fear River	J13	5,255.0	9,690	1.84
02106500	BLACK RIVER NEAR TOMAHAWK, NC	Black River	J30	676.0	1,330	1.97
02108000	NORTHEAST CAPE FEAR RIVER NEAR CHINQUAPIN, NC	Northeast Cape Fear River	J32	599.0	1,170	1.95

The 72-hr, all-season PMP for the Cape Fear River Watershed was obtained from HMR 51 (Reference 14) and then spatially and temporally distributed using HMR 52 software with an ArcGIS interface.

NOAA Atlas 14 (Reference 19) was used to determine a spatially averaged 500-year/3-day rainfall value for the Cape Fear River Watershed for comparison with the spatially-averaged 40% PMP. The spatially averaged 40% PMP for each sub-basin was computed by multiplying the 3-day total PMP depth of 18.18 inches by 0.40 resulting in 7.27-inch basin-wide 40% PMP. The 500-year 3-day rainfall depth for the Cape Fear River Watershed is 13.94 inches. The comparison between the 500-year NOAA Atlas 14 rainfall at the site and the 40% PMP shows that the 40% PMP is the lesser of the two storms. Both antecedent and subsequent storm events were considered per NUREG/CR-7046 (Reference 3).

HMR 52 software with an ArcGIS interface was used to evaluate storm configurations producing the highest basin-averaged rainfall over the watershed. The following storm centers were identified for the analysis: (1) Basin Centroid representing the geometric center of the Cape Fear River Watershed; (2) Centroid of the upper portion of the Cape Fear River Watershed; and (3) Centroid of the lower portion of the Cape Fear River Watershed. The locations of the three storm centers are provided in Figure 4-4.

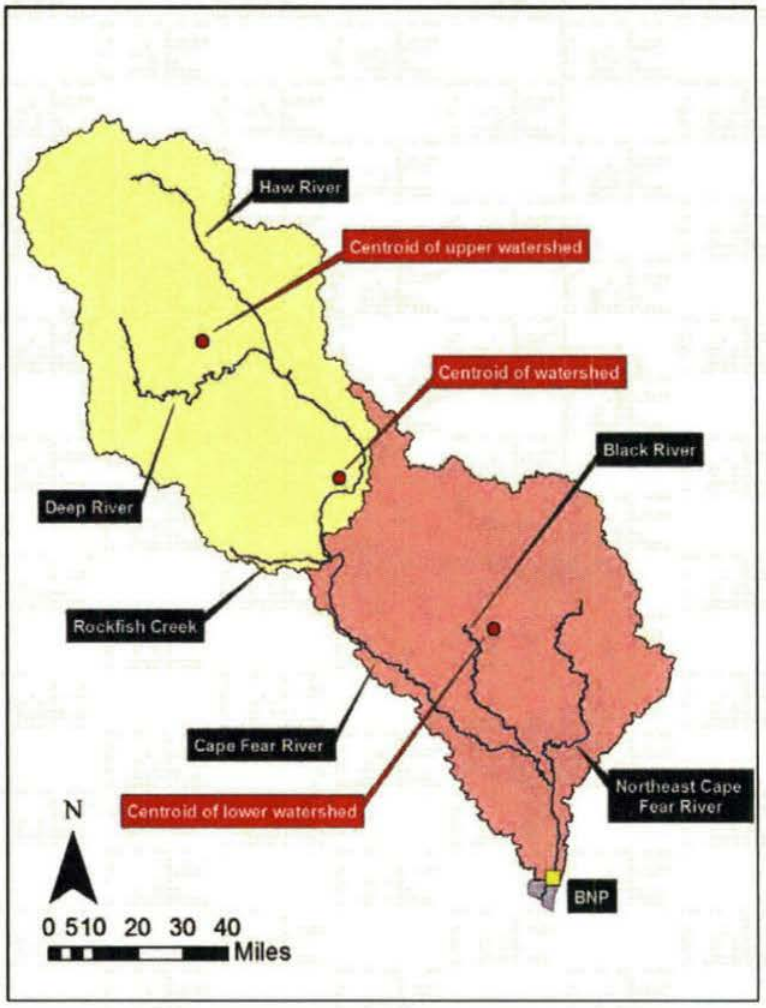

Figure 4-4: Location of Storm Centers

Various storm orientations and sizes for each storm center were evaluated to determine the maximum rainfall depth over the watershed. Optimal combinations of storm orientation and size for input into HEC-HMS were determined by identifying the maximum average 72-hr depth and maximum rainfall depth at the storm peak 6-hr interval. Figure 4-5 shows the map of the isohyetals for the most critical PMP storm.

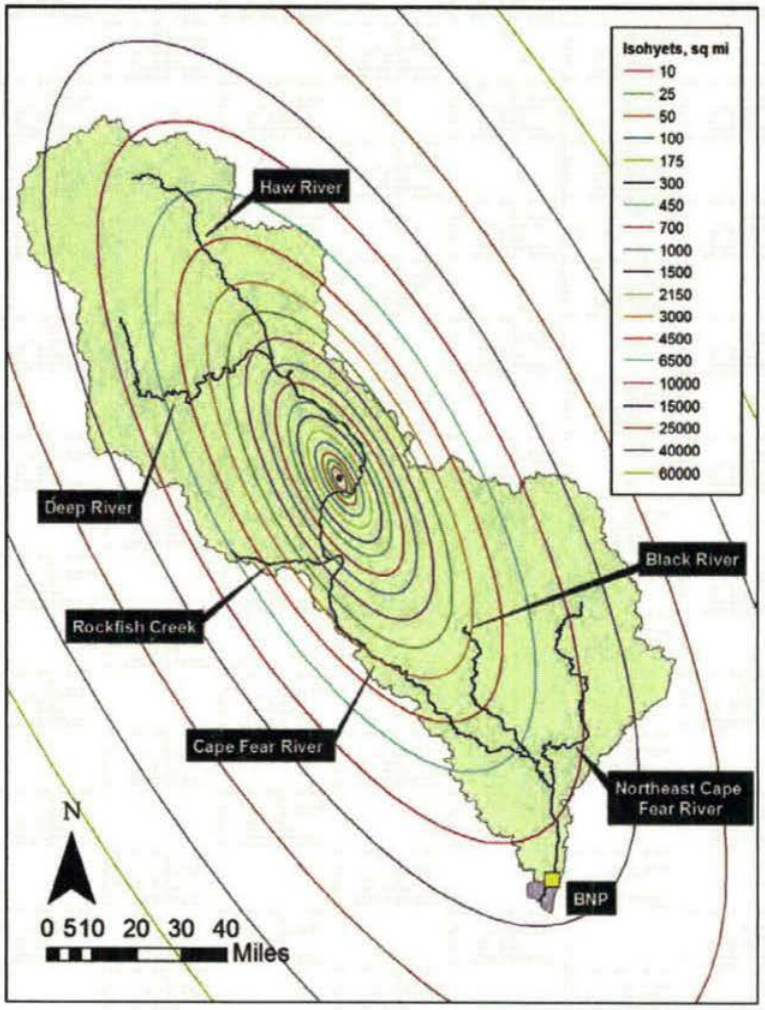

Figure 4-5: Isohyetals for the Most Critical PMP Storm

The calibrated HEC-HMS model was then used to determine the runoff at BNP resulting from the various orientations and sizes of the PMP event. The PMP event centered over the centroid of the Cape Fear River watershed with orientation of 150 degrees and basin-averaged rainfall of 18.18 inches, generated the highest discharge at BNP (444,460 cfs). The hyetographs of antecedent and subsequent storms (40% PMP) were included in the HEC-HMS model runs. A three-day dry period between the antecedent or subsequent storms and PMP was used for the precipitation runs. Various temporal distributions of the PMP event, with the peak rainfall increment occurring 30 hours, 36 hours, 42 hours, 54 hours, and 72 hours following the start of the PMP precipitation, were evaluated using the HEC-HMS model. The temporal distributions for the critical PMP event are presented in Figure 4-6.

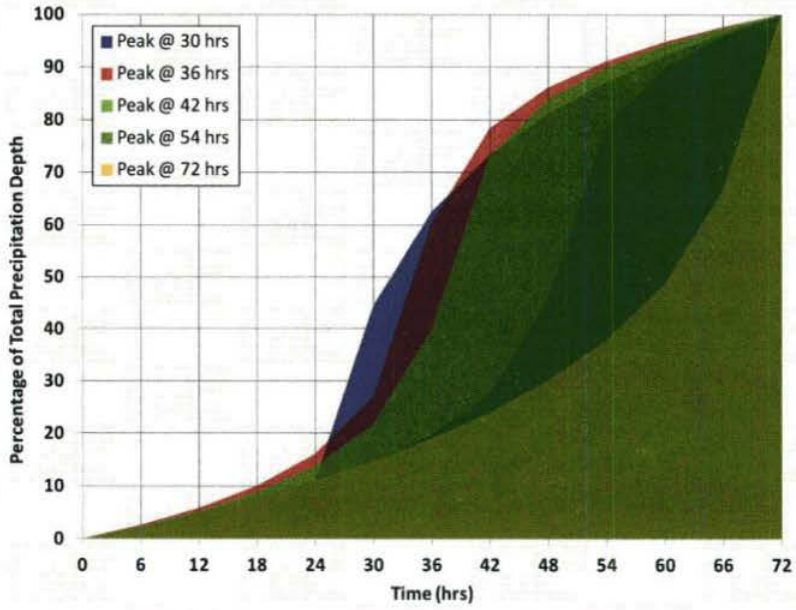

Figure 4-6: Temporal Distributions of the Critical PMP Event

The critical Alternative 1 PMF peak discharge is a result of a 40% antecedent PMP followed by a 2/3-loaded PMP. The critical Alternative 1 PMF results in a peak discharge of 512,000 cfs at BNP. The critical Alternative 1 PMF hydrograph is shown in Figure 4-7. The results of the various model runs are provided in Table 4-5.

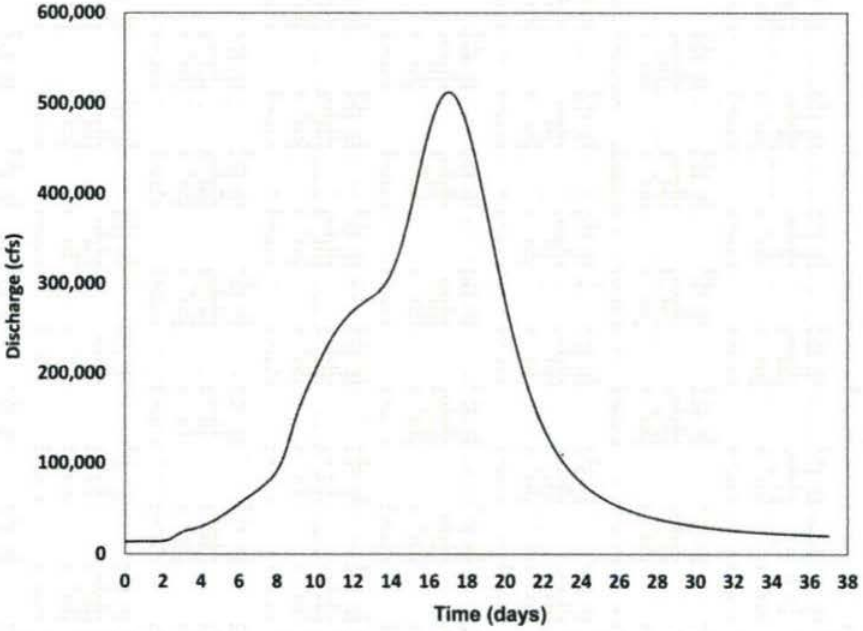

Figure 4-7: Critical Alternative 1 PMF Hydrograph

Table 4-5: Alternative 1 PMF Discharge results

Hyetograph Temporal Distribution	Peak Flow at BNP (cfs)
Front-loaded	503,692
1/3-loaded	508,121
Center-loaded	510,036
2/3-loaded	512,000
Back-loaded	508,951

The peak flow for the critical Alternative 1 PMF was used in the steady flow analysis to determine the governing PMF discharge and stage at BNP, as discussed in the "Governing Riverine PMF Discharge and Stage" and "Dam Breaches and Failures" sections.

Alternative 2 and Alternative 3

Per NUREG/CR-7046, Appendix H.1 *Floods Caused by Precipitation Events* (Reference 3), the Alternative 2 combined-effect precipitation was evaluated to include:

- Mean monthly base flow;
- Probable maximum snowpack; and
- A 100-year, snow-season rainfall.

The Alternative 3 combined-effect precipitation was evaluated to include:

- Mean monthly base flow;
- A 100-year snowpack; and
- Snow-season PMP.

A qualitative assessment was used to evaluate whether a combination of PMSP and snow-season PMP is bound by Alternative 1 combined-effect precipitation events. The snow-season PMP was estimated based on the comparison of the 72-hr, 10-square-mile all-season PMP reported in HMR 51 (Reference 14) to the 72-hr, 10-square-mile January/ February PMP reported in HMR 53 (Reference 15). The PMP depths derived from HMR 51 and HMR 53 are presented in Table 4-6.

Table 4-6: Inputs for Alternatives 2 and 3 from HMR 51 and HMR 53

HMR 53 Snow-Season 10-mi^2, 72-hr PMP (January/February)	28.00 in
HMR 51 10-mi^2, 72-hr PMP (All-Season)	46.00 in
Snow-Season PMP/All Season PMP Ratio	0.61
HMR 52 Basin-Averaged, 72-hr PMP (All-Season)	18.18 in

The publicly available NOAA Global Historical Climatology Network (GHCN) dataset was used to make a conservative estimate of the maximum daily snow depth for the Cape Fear River watershed. For the Alternative 2 and 3 analysis, the Cape Fear River watershed was divided into seven (7) sub-watersheds. A single NOAA station was considered for each of the subwatersheds, selected based on the closest distance from the subwatershed centroid and availability of a sufficiently large number of snow depth data. Figure 4-8 shows the subwatersheds, centroids, and NOAA stations considered for the historic daily snow depth analysis. The custom-generated GHCN daily snow depth data series were then analyzed to obtain the

maximum daily snow depth for each month. Table 4-7 summarizes the values of maximum daily snow depth for each subwatershed by month. An overall maximum daily snow depth of 22.01 inches was recorded at the Siler City station (Gage GHCND: US1NCCH0006 in the SW1 sub-basin, as shown in Figure 4-8). This value was used as a conservative estimate of the probable maximum storm in the Cape Fear River watershed.

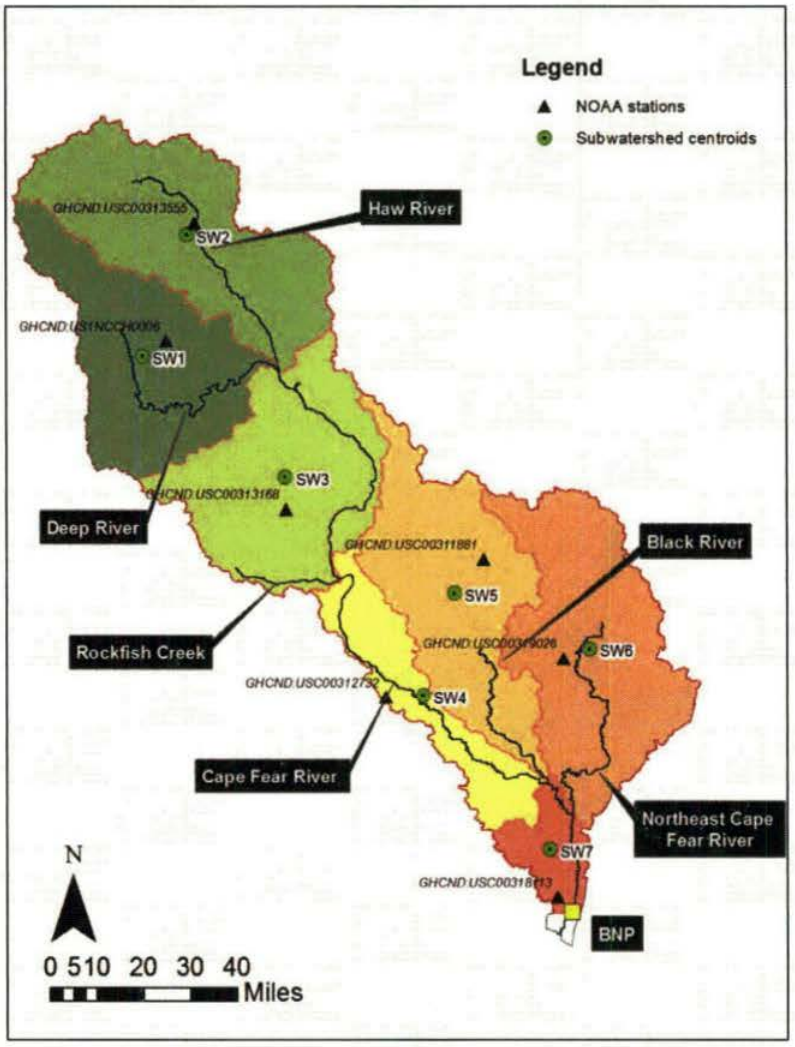

Figure 4-8: Subwatersheds, Centroids, and NOAA Stations Considered for Historic Daily Snow Depth Analysis

Table 4-7: Historical Maximum Daily Snow Depths

Subwatershed ID	Data Available (Years)	Maximum Historical Daily Snow Depth, in					
		January	February	March	November	December	Maximum by Station, in
SW1	1916-2014	22.01	10.98	7.99	0.98	7.99	**22.01**
SW2	1903-2014	12.99	15.00	17.99	0	10.98	17.99
SW3	1964-2010	7.01	5.00	0	0	0	7.01
SW4	1911-2014	7.99	7.99	0	0.98	7.01	7.99
SW5	1971-2014	5.98	7.99	10.00	0	10.00	10.00
SW6	2000-2014	2.99	2.99	0	0	0	2.99
SW7	1900-2010	4.02	4.02	7.99	0	0	7.99
Maximum by Month, in		**22.01**	15.00	17.99	0.98	10.98	

The basin-averaged snow-season PMP for the Cape Fear River watershed was estimated by applying the snow-season/all-season PMP ratio (0.61) to the HMR 52 basin-averaged all-season PMP (18.18 inches), resulting in a basin-averaged snow-season PMP of 11.07 inches. The snow water equivalent of the PMSP was calculated using the following equation:

$$Snow\ Water\ Equivalent = \frac{\rho_s}{\rho_w} d_s = 4.40\ inches$$

Where:
ρ_s = Desity of snowpack (conservatively assumed 0.2 g/cm^3);
ρ_w = Density of water (1 g/cm^3); and
d_s = Depth of snowpack (conservatively assumed 22.01 inches).

The combined snow-season PMP (11.07 inches) and snow water equivalent of the PMSP (4.40 inches) was calculated to be 15.47 inches, which is less than the all-season PMP of 18.18 inches. The all-season PMP is significantly greater than a conservative estimate of the combined PMSP and snow-season PMP and, therefore, the all season PMP would produce greater peak discharges than an unlikely rain on snow event. Further evaluation of Alternative 2 and Alternative 3 was not performed since these events are bounded by the Alternative 1 event.

Governing Riverine PMF Stage

A steady-flow hydraulic HEC-RAS model was developed to calculate maximum flood elevations from the Alternative 1 PMF discharge presented above. The same model was also used for estimation of maximum flood elevations due to dam breaches and failures discussed in Section 4.c of this report.

The channel and overbank geometry of the HEC-RAS model was based on USGS NED data, LiDAR and bathymetric survey data. The governing precipitation-driven peak discharge was modeled in HEC-RAS to determine the associated flood stage.

The ground surface DEM was developed by combining available USGS NED, LiDAR, and bathymetric data to reflect current topography of the stream channel and reach overbanks. Manning's n-values were based on available orthoimagery. The Cape Fear River is not gaged in the vicinity of BNP and, therefore, Manning's n-values could not be calibrated against observed data. Conservative Manning's n-values of 0.05 and 0.20 were used for channel and overbank areas, respectively. Both values correspond to the upper end of the range of acceptable values per HEC-RAS User Manual (Reference 18).

The HEC-RAS model downstream boundary condition was set to a known water surface elevation of 2.63 ft NGVD29 corresponding to 10% exceedance high tide in accordance Section H.3.2 of NUREG CR/7046 (Reference 3).

The predicted water surface elevation at the BNP intake for the governing precipitation-driven peak discharge is 11.86 ft NGVD29 .

Nancys Creek

Nancys Creek is located just to the north and adjacent to BNP and is within in the Cape Fear watershed. Figure 4-9 shows the location of the 1.9-square-mile Nancys Creek watershed relative to the BNP site.

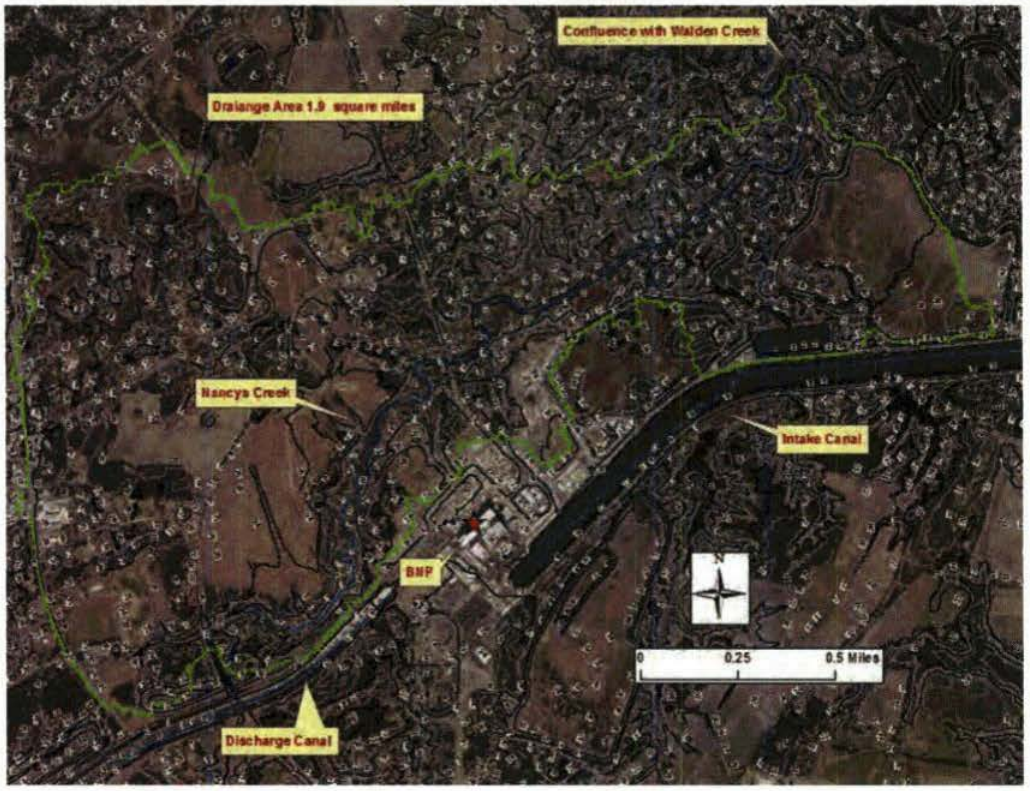

Figure 4-9: Nancys Creek Watershed

The peak discharge and associated flood levels of Nancys Creek were evaluated to determine if flooding from Nancys Creek would challenge the site during an extreme storm event. The following subsections describe the PMF hydrologic and hydraulic analyses performed for Nancys Creek.

Nancys Creek PMF Hydrologic Analysis

HEC-HMS was used to simulate the hydrologic processes of the watershed. The model was developed using a single basin. No stream gage data is available for Nancys Creek to allow for model calibration. Therefore the HEC-HMS model was developed assuming no infiltration (initial and constant) losses in the watershed model to maximize runoff and evaluate the most conservative water surface of Nancys Creek adjacent to the site during PMP storm event. The Snyder transform method was used assuming conservative inputs for

the watershed (Peaking Coefficient of 0.8, and C_t of 3.0) resulting in a conservative lag time and peak discharge. According to the USACE NID database there are no dams in the Nancys Creek watershed.

Baseflow for the PMF event was based on the March average flow rate. Considering the size and response of the watershed, a 72-hr PMP is not applicable. Therefore, 6-hr and 12-hr PMP storms were developed for PMF study using USACE HMR-52 software with an ArcGIS interface.

Both 6-hr and 12-hr PMP storms were run in the HEC-HMS model. The 12-hr PMP generated a higher discharge at the outlet of Nancys Creek. Therefore, the 12-hr PMP was selected to provide a conservative estimate for the PMF. Table 4-8 summarizes the results of the model runs and shows time to peak, peak discharge, and discharge volume. Figure 4-10 shows the rainfall depth, base flow and peak flow for the controlling run in HEC-HMS (Model Run 4).

Table 4-8: Summary of HEC-HMS Runs for Nancys Creek

Model Run	PMP Storm	Scenario	Peak Time	Peak Q (cfs)	Discharge Volume (in)
1	6-hr	Base	7:45	2,941	30.18
2	12-hr	Base	11:10	3,186	35.41
3	12-hr	No Loss	11:15	3,344	37.88
4	12-hr	No Loss, Highest Cp	10:55	6,300	43.16

Base Scenario: Base simulation with loss rates estimated from NRCS soil data
No Loss Scenario: Simulation with initial loss and loss rates set to zero to consider the most conservative response
No Loss, Highest Cp Scenario: Simulation with initial loss and loss rates set to zero, Cp set at highest end of range to consider the most conservation peak flow

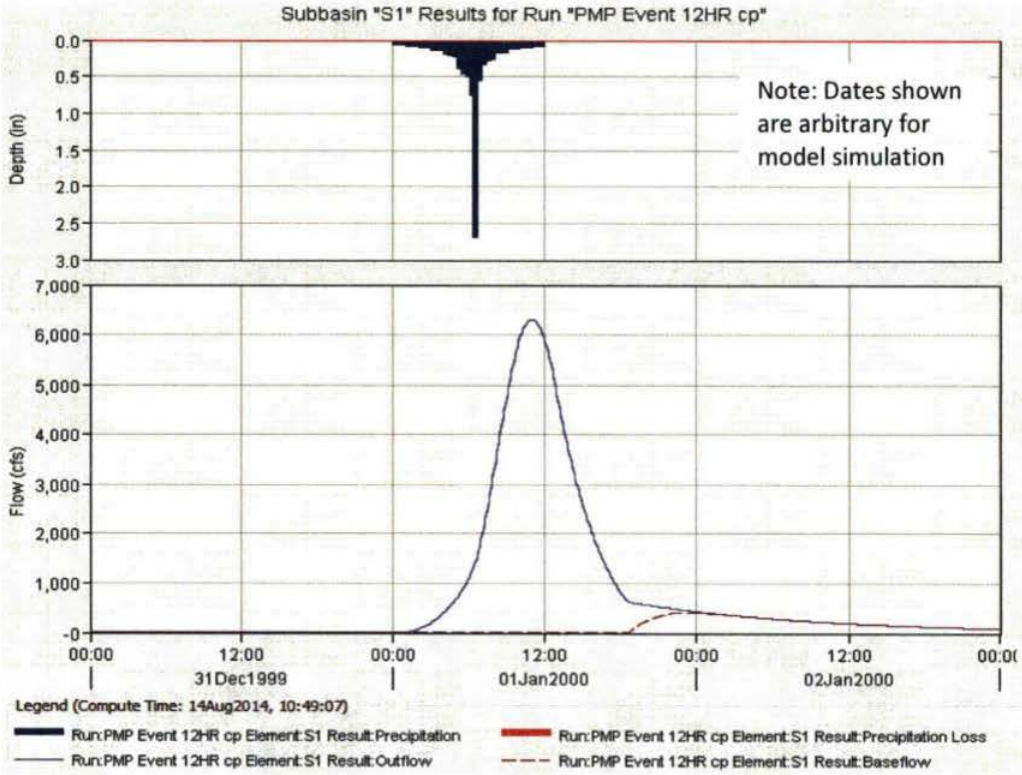

Figure 4-10: Critical PMF Discharge for Nancys Creek (Model Run 4)

Nancys Creek PMF Hydraulic Analysis

A steady-flow, one-dimensional hydraulic analysis of Nancys Creek was conducted using HEC-RAS. The model was based on the 2008 FEMA Effective Hydraulic Model study, which was developed to evaluate and map the 100-year floodplain for Nancys Creek. The cross sections from the FEMA Model were not adjusted, and as a result some cross sections do not contain the full PMF floodplain. This results in a conservative calculation of the PMF peak water surface elevations for cross sections that do not envelope the PMF since the HEC-RAS model assumes a vertical wall in these instances and, therefore, does not account for full storage in the floodplain. Manning's n-values (Table 4-9) used in the FEMA model were reviewed and considered applicable for use in this study.

Table 4-9: Manning's Roughness Coefficients (source FEMA model) for Nancys Creek

Stream	Channel n-values	Overbank n-values
Nancys Creek	0.041 to 0.045	0.084 to 0.140

A normal depth slope of 0.0009 ft/ft was used as the downstream slope boundary condition. The sensitivity of the Nancys Creek HEC-RAS model downstream boundary condition was evaluated through setting the downstream starting water surface elevation to 11.6 ft NGVD29, which is 10.5% greater than the normal depth starting elevation. The maximum water surface elevation at the site corresponding to the increased boundary condition is 15.65 ft NGVD29, and 0.19 ft greater than the maximum elevation estimated using the normal depth as the downstream condition. Based on this sensitivity evaluation, the maximum water surface elevation at the site is not sensitive to the downstream starting elevation and, therefore, the

normal depth slope of 0.0009 ft/ft is an appropriate downstream boundary condition for this study. The PMF peak water surface elevation at the site corresponding to the peak PMF discharge of 6,300 cfs is 15.46 ft NGVD29, which is 4.54 ft below plant grade of 20 ft NGVD29.

c. Dam Breaches and Failures

Breach or failure of artificial barriers used to impound water for multiple possible functions, including flood control (attenuation), recreation, water supply, hydroelectric, sediment storage, aquatic habitat, stormwater (quantity/quality) management, or a combination thereof, located within the watershed of an adjacent stream/river or upslope of SSCs important to safety were evaluated. Flood waves resulting from the breach of upstream dams considered domino-type or cascading dam failures.

Upstream dam breaches and failures were evaluated for sunny day, seismic, and overtopping breach events in accordance with NUREG/CR-7046, Section 3.9 and Appendix H.2 (Reference 3). The dam failure analysis was performed using the calibrated HEC-HMS model.

The deterministic HEC-HMS model initially developed was used to estimate the flood hydrographs associated with overtopping dam failure, seismic dam failure, and sunny day dam failure. The combined model provides an accurate representation of the Cape Fear River watershed response to an extreme rainfall event by appropriately combining the parameters from the individually calibrated models (loss, rainfall-runoff transformation, flow routing, and baseflow parameters) to account for the spatial distribution and magnitude of each storm event considered.

The evaluation of upstream dam failures for the Cape Fear River watershed is described in the subsections below.

Combined Storage Volume of Small Dams within the Watershed

According to the North Carolina Division of Energy, Mineral and Land Resources (NCDEMLR) database there are 1,290 dams in the Cape Fear River watershed upstream of BNP. Their location and characteristics were obtained from the NCDEMLR database and any data gaps were closed with information from the USACE NID database. The resulting dataset was used for the dam screening analysis to remove inconsequential dams from consideration, per Section 3.1 of JLD-ISG-2013-001 (Reference 1). Inconsequential dams are defined as dams having minimal or no adverse failure consequences beyond the owner's property.

The dam screening evaluation identified Jordan Lake Dam as the only potentially critical dam for BNP. This dam was modeled individually in HEC-HMS. In addition, Shearon Harris Dam, owned by Duke Energy was also modeled individually in HEC-HMS. Table 4-10 provides the characteristics of the two dams.

Table 4-10: Characteristics of Dams Modeled Individually

Individual Dam	Dam Height (ft)	Maximum Storage Volume (acre-ft)	Top of Dam Length (ft)
Jordan Lake Dam	(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)		
Shearon Harris Dam			

The remaining dams were grouped in clusters, each represented by a composite dam. The grouping of the dams and their representation by a single composite dam with storage volume equal to the sum of the storage volume values of all dams in each cluster is conservative, because the attenuation of any individual breached hydrograph from one reservoir to the next is not taken into account. Table 4-11 reports the

characteristics of the composite dams. The characteristics of the composite dams were established using the following assumptions:

- The storage volume of each composite dam was based on the sum of the storage volume values of all dams in each cluster.
- The height of each composite dam was based on the highest dam in each cluster.
- The length of the crest of each composite dam was based on the length of the crest of the highest dam in each cluster.
- Each composite dam was placed in the HEC-HMS model at the downstream end of the subwatershed associated with the corresponding cluster of dams.
- The elevation-storage relationship for each composite dam was based on a simplified linear relationship.
- The elevation-discharge relationship for each composite dam was based on a spillway formulation with spillway elevation equal to top of the dam; length equal to the length of the crest; and weir coefficient equal to 2.8. The selected coefficient is on the conservative end of the range of values between 2.0 and 3.0 for US Customary units recommended in the HEC-HMS User's Manual Version 4.0 (Reference 16).

Table 4-11: Characteristics of Composite Dams

Composite Dam	Number of Dams in Cluster	Dam Height (ft)	Total Storage Volume (acre-ft)	Top of Dam Length (ft)
C1	88	102	175,178	2,895
C2	162	72	216,344	1,350
C3	131	50	50,211	600
C4	2	23.5	47	325
C5	9	30	3,131	10,000
C6	26	25	5,543	315
C7	0	N/A	N/A	N/A
C8	14	29.5	2,223	630
C9	2	30	3,900	1,584

Location of dams and drainage areas for composite dams are shown in Figure 4-11.

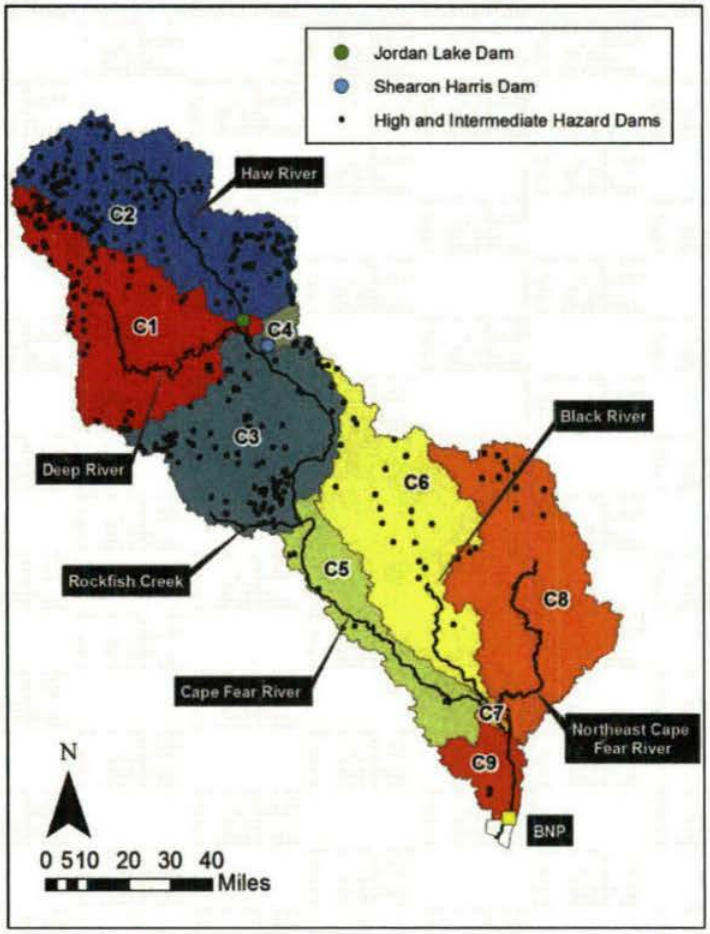

Figure 4-11: Individual Dams and Drainage Areas for Dam Cluster Grouping

Sunny-Day Dam Failure

In the sunny-day dam breach analysis the subbasins in the HEC-HMS model were updated to remove any baseflow. The starting water surface elevations for the critical dams identified through the dam screening processes were set to the maximum normal pool elevation (i.e., crest of the auxiliary spillway).

Breach parameters for the composite dams were calculated assuming water surface at the top of the dam. For Jordan Lake Dam, (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)
(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) For Shearon Harris Dam,
(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

For each dam, individual or composite, the breach parameters were estimated using methodologies by Froehlich (Reference 20) and Xu & Zhang (Reference 5), and were applied on a dam-by-dam basis, depending on which methodology produced the most conservative breach formation time.

All dams were assumed to fail at the same time due to a common cause (piping failure). Dam breach by piping was used to reflect a potential mode of failure caused by a sunny-day failure.

The resulting HEC-HMS hydrographs for the sunny-day dam failure scenario were used as inputs for the steady-flow HEC-RAS model to estimate the Cape Fear River PMF discharge and stage at BNP.

The peak flow rate for sunny-day dam breach scenario in the Cape Fear River was calculated to be [redacted] cfs. The sunny-day failure peak discharge is bounded by the seismically-induced and overtopping dam failure scenarios, which do not challenge the site and therefore is not evaluated further.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Seismically-Induced Dam Failure

Appendix H.2 of NUREG/CR-7046 (Reference 3) and Section 9.2.1.2 of ANS-2.8 (Reference 4) provide the following two (2) alternative combinations for seismically-induced dam failure:

1. 25-year flood, dam failure caused by the safe shutdown earthquake (SSE) coincident with the peak flood, and 2-year wind speed applied in the critical direction.
2. ½ PMF or 500-year flood, whichever is less, dam failure caused by the operating basis earthquake (OBE) coincident with the peak flood, and 2-year wind speed applied in the critical direction.

The best available interpretation of the above requirements is that seismically-induced dam failure occurs coincident with the peak pool levels for the 25-year flood, 500-year flood, or ½ PMF entering the dam; implying that runoff downstream of the dam to the site is not included.

This approach is reasonable for a single upstream dam but is inappropriate for multiple dams. It would not be reasonable to assume that peak flood pool levels for all upstream dams occur coincident with an earthquake. Developing floods for multiple upstream dams is best accomplished by using watershed-wide precipitation events, not upstream flood-flow, to represent the above combinations. Therefore, the seismically-induced dam failure analysis was based on the following precipitation-based combinations:

- Alternative 1 – 25-year precipitation throughout the site's watershed; dam failure resulting from a Safe Shutdown Earthquake (SSE); and waves induced by 2-year wind speed applied along the critical direction.
- Alternative 2 – the lesser of the ½ PMP or 500-year precipitation throughout the site's watershed; dam failure resulting from an Operating Basis Earthquake (OBE); and waves induced by 2-year wind speed applied along the critical direction.

This is consistent with NRC expectations that, as stated in NUREG/CR-7046 (Reference 3) and ANS-2.8 (Reference 4), combinations are thought to have a probability-of-exceedance of less than 1×10^{-6}. Also, using watershed-wide precipitation events is conservative because it includes runoff downstream of the dams to the site. Since all dams are assumed to fail during the lower-magnitude (OBE) earthquake, only the Alternative 2 combination was included in the analysis. Alternative 2 uses a higher precipitation event as input, resulting in higher discharge at BNP.

The simulations for seismic dam failure scenario were performed using the lesser of the 500-year precipitation and ½ PMP. The ½ PMP was calculated by dividing in half the watershed-averaged 72-hr All Season PMP, and was compared to the watershed-averaged 72-hr 500-year average precipitation. The watershed-averaged 72-hr ½ PMP rainfall depth (9.09 in) is less than the watershed-averaged 72-hr 500-year rainfall depth (13.94 in), and was therefore used as precipitation in the seismic dam failure evaluation.

Note that all dams are assumed to fail when the earthquake occurs (the timing of which is established based on optimal impact to the site), which may not result in a particular dam failing at its peak water level. Dam breach by piping was used to reflect a potential mode of failure caused by a seismic event. Seismically-

induced soil liquefaction can lead to embankment failure for earthen dams or foundation failure for other types of dams (Reference 1).

Breach parameters for the composite dams were calculated assuming the reservoir water surface is at the top of the dam. For the individual dams, breach parameters were calculated for the maximum values of water volume and depth in the reservoir obtained from a simulation in absence of dam breach.

For each dam, individual or composite, the breach parameters corresponding to the lowest breach formation time, as determined using the regression-based methods by Froehlich (Reference 20) and Xu and Zhang (Reference 5), were then used in the dam breach analysis.

Five precipitation temporal distributions were simulated in absence of dam breaches to determine the most critical distribution. HMR 52 (Reference 13) methodology was applied to temporally distribute the ½ PMP with the peak rainfall increment occurring 30 hours (front-loaded distribution), 36 hours (1/3-loaded distribution), 42 hours (center-loaded distribution), 54 hours (2/3-loaded distribution), and 72 hours (back-loaded distribution) following the start of the ½ PMP precipitation. Per HMR 52 (Reference 13), the four highest 6 hour increments were not placed within the first 24-hr period of the storm sequence. It was determined that the distribution producing the largest peak at BNP is the 2/3-loaded distribution.

In order to determine the critical peak flow based on a seismic dam failure, several breach times were evaluated for simultaneous failure by piping of all dams in the watershed. Table 4-12 presents the seismic dam breach scenario results at BNP.

Table 4-12: Flood Arrival Time and Peak Flow at BNP for the Seismic Dam Failure Scenarios

Trigger Time of Simultaneous Breaches (time after start of ½ PMP) (hrs)	Peak Discharge Time from the Start of ½ PMP Event (hrs)	Peak Discharge (cfs)
(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)	(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)	(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Based on the results in Table 4-12, a simultaneous breach of all dams [redacted] hours after the start of rainfall produces a critical peak flow rate of [redacted] cfs at BNP. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

The HEC-RAS model developed for the Cape Fear River was run with the governing seismic peak flow to determine the peak water surface elevation at the site. The calculated governing seismic dam failure flood peak water surface elevation is [redacted] ft NGVD29 at the BNP intake. This is [redacted] ft below the plant grade elevation (20.00 ft NGVD29) and therefore does not challenge the site. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Overtopping Dam Breach

For the overtopping dam breach analysis, the HEC-HMS model used to estimate the peak discharge resulting from a seismic dam breach was modified with breach parameters specific to an overtopping breach event and with the critical precipitation alternatives consisting of a 40% PMP antecedent event followed by a PMP event.

The initial water surface elevation in the Jordan Lake was set equal to the conservation pool level of 216 ft. The conservation pool elevation was selected as the initial water surface elevation for routing of the flood hydrograph since the dam is operated by USACE as a flood control structure and there is a reasonable justification that the conservation pool would be maintained in the anticipation of a large rainfall event, such as the PMP. In addition, a review of the actual lake level data prior to the March 1998 and September 1999 storms showed that the reservoirs were maintained close to conservation pool levels at 216.19 ft NGVD29 and 216.57 ft NGVD29, respectively. The initial water surface elevation in the Shearon Harris reservoir was set equal to the maximum controlled elevation in the main reservoir, equal to 220 ft NGVD29, which is the main reservoir spillway crest elevation. Initial water surface elevation for the composite dams was conservatively assumed equal to the top of dam.

Breach parameters for the composite dams were calculated assuming water surface at the top of the dam. For the individual dams, which are not overtopped during the PMF, breach parameters were calculated using an iterative process. In the first scenario (Scenario A), the triggering water surface elevation for overtopping dam breach was set equal to the elevation of the top of the dam for the composite dams, whereas individual dams were set not to breach. This allowed for evaluating the time evolution of water surface elevation and water volume in the reservoirs of the individual dams. In the second scenario (Scenario B), all individual dams were set to breach at the time of maximum volume in the reservoir (determined in Scenario A), computing the breach parameters for the actual values of water volume and depth in the reservoir.

For each dam, individual or composite, the breach parameters corresponding to the shortest breach formation time, as determined using the regression-based methods by Froehlich (Reference 20) and Xu & Zhang (Reference 5), were then used in the dam breach analysis

Simulations for overtopping dam failure were performed using a time step of 1 minute, in order to accurately model the short and quickly varying hydrographs associated with dam breaches. Each dam was set in the HEC-HMS model to fail when the maximum stage level is reached in the reservoir during the PMF storm event.

The peak flow at BNP for the dam breach scenarios simulated is provided in Table 4-13. Table 4-13 also presents the flood arrival time, computed as the time between the start of the rainfall and the peak flow. A comparison of hydrographs for the various dam breach scenarios is provided in Figure 4-12. A comparison of hydrographs for the critical dam failure scenario and Alternative 1 PMF discharges is provided in Figure 4-13.

Table 4-13: Flood Arrival Time and Peak Flow at BNP for the Overtopping Dam Breach Scenarios

Scenario	Dam Breach Triggering Criteria	Flood Arrival Time from the Start of PMP Event (hrs)	Peak Flow (cfs)
Dam Breach Scenario A	Composite dams: Water surface elevation in the reservoir equal to top of dam elevation Individual dams: no breach	(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)	
Dam Breach Scenario B	Composite dams: Water surface elevation in the reservoir equal to top of dam elevation Individual dams: breaching set at the time of maximum water surface elevation in the reservoir		

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Figure 4-12: HEC-HMS Hydrographs at BNP for Various Dam Breach Scenarios

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Time (days)

Figure 4-13: HEC-HMS Hydrographs at BNP for Alternative 1 PMF and Critical Overtopping Dam Failure (Scenario B) Discharges

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Based on the results in Table 4-13, the governing dam breach peak discharge at BNP resulting from overtopping dam failure is equal to cfs.

The peak water surface elevation corresponding to the governing discharge was determined using a steady-flow HEC-RAS model for the Cape Fear River. The calculated governing overtopping dam failure flood peak

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

water surface elevation is [redacted] at the BNP intake. This is [redacted] ft below the nominal plant grade elevation (20.00 ft NGVD29) and, therefore, does not challenge the site.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

d. Storm Surge and Seiche

Probable Maximum Storm Surge (PMSS) from a Probable Maximum Windstorm (PMWS), typically a Probable Maximum Hurricane (PMH), and from a Probable Maximum Seiche (PMS) were evaluated using a hydrodynamic and wave model Delft3D, versions 4.01.000.hm and 4.01.01.rc.03. The calibrated model was used to compute the wind, pressure and wave driven storm surge at BNP.

In order to quantify the site-specific ocean circulation, best available knowledge of tidal circulation and atmospheric forcing in the region was used in the Delft3D model. The values for the physical parameters in the model have been determined based on literature review of recommended and nominal values for the area of interest and the North Atlantic Ocean. Modeling time steps for each nested grid have been selected based on stability and accuracy considerations derived from the Courant-Friedrichs-Levy (CFL) condition for each nested grid and the momentum and advection schemes used.

Swell propagation and wave generation under action of the wind were modeled using the third-generation numerical wave model Delft3D-WAVE, which represents an implementation of the Simulating WAves Nearshore (SWAN) wave model that is enabled for coupling with other Delft3D modules.

The bathymetric grid was developed using nested rectangular grids in spherical (geographic) coordinates to represent the area of interest with a high degree of detail, while reasonably constraining computational times. As illustrated Figure 4-14, a series of nested grids are developed from a resolution of approximately 1,500 m down to a resolution of approximately 18.5 m. The finest grids allowed a detailed representation of water flow in the vicinity of BNP while the largest grid provided a wide enough area to cover the developing storm surge.

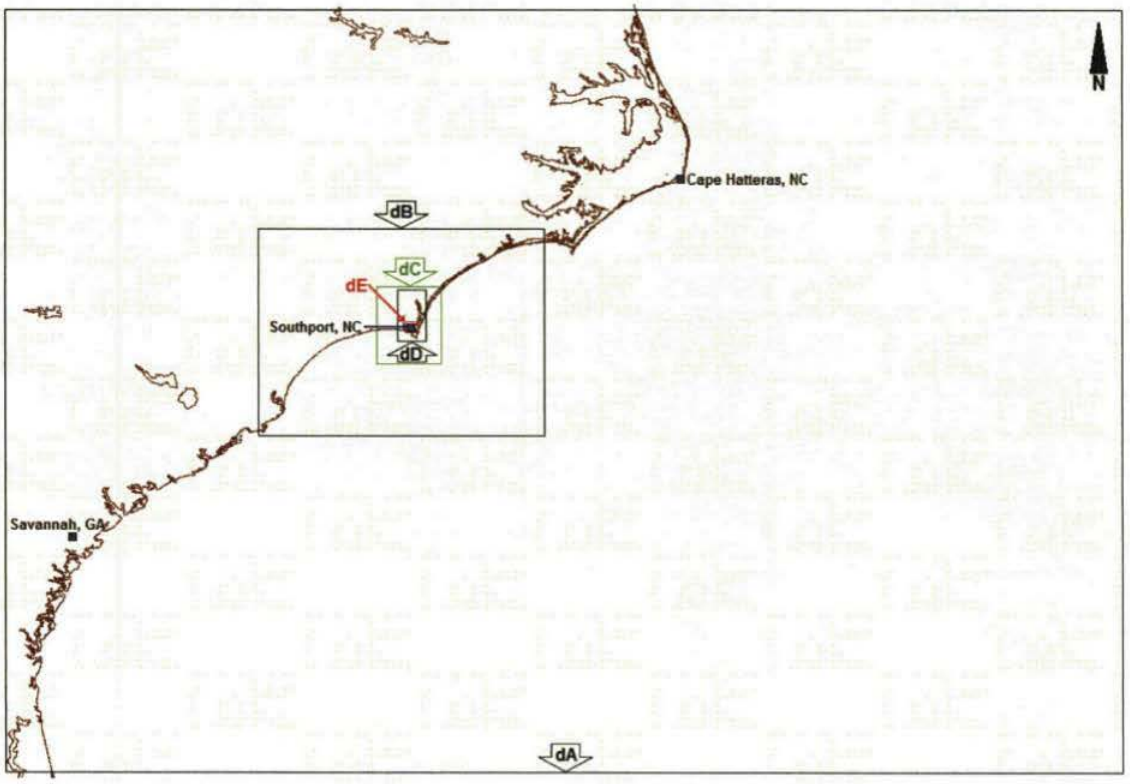

Figure 4-14: Computational Model Grid

Calibration of the hydrodynamic model and derivation of historical storm surge estimates was performed using hourly water levels from the NOAA Center for Operational Oceanographic Products and Services (CO-OPS) online archive for five sites in the vicinity of BNP. Historical wave observations during hurricane Ernesto from the Frying Pan Shoals buoy (41013) of the NDBC (National Data Buoy Center) were used to validate the numerical wave model.

The guiding document for determining the PMH was NOAA Technical Report NWS 12 (TR23) (Reference 39). The document includes a list of tropical storms that have affected the Pacific, Atlantic and Gulf of Mexico coasts of the United States and provides a description of methods used to calculate the various parameters for a PMH. Additional data was obtained from HURDAT reanalysis dataset for all tropical cyclones in the Atlantic Ocean, Gulf of Mexico, and Caribbean Sea since 1851. The four key parameters that were determined as part of the analysis were the minimum central pressure, the radius of maximum winds, the forward speed and the gradient wind.

The Holland method (Reference 40) was used to develop a hurricane wind field based on a Rankine vortex. The model used the best track hurricane parameters to calculate the surface wind and pressure fields over a spherical grid and the results were formatted for input to the coupled hydrodynamic and waves models, Delft3D and SWAN. The Holland hurricane model was implemented as part of the Delft3D-WES (Wind Enhance Scheme for cyclone modeling) module.

The simulated wind speed, wind direction, and atmospheric pressure were in good agreement with the timing and magnitude of the observations at three meteorological stations on the coast (Myrtle Beach, Wrightsville, Cape Lookout) during the September 2006 Hurricane Ernesto, as shown in Figure 4-15. The

wind speed and magnitude of the pressure drop are particularly well reproduced in areas closer to the center of the storm track (Wrightsville), while they are overestimated at locations further away on both sides of the track (Myrtle Beach and Cape Lookout), due to the inherent limitations in the hurricane wind and pressure model to represent conditions near and over land areas.

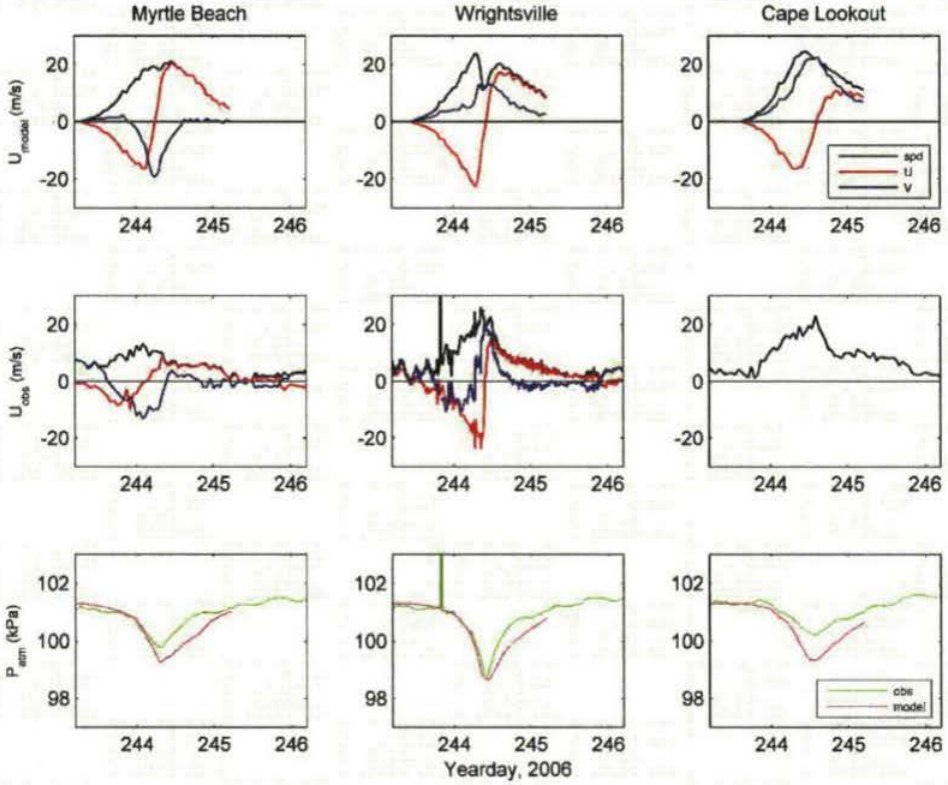

Figure 4-15: Comparison of observed (U_{obs}) and predicted (U_{model}) wind speeds and atmospheric pressures (P_{atm}) for Hurricane Ernesto in 2006. The U plots provide the indication of the magnitude of velocity (spd), the u-component of velocity (U), and v-component of velocity (V).

The numerical model performance was validated against observations during Hurricane Ernesto, which produced one of the most significant surge events of tropical origin at the Southport tide gage over the observation period available. Moreover, Hurricane Ernesto was the only storm for which the full set of meteorological and oceanographic parameters were observed near the site, including water levels at Southport; wave properties at the Frying Pan Shoals location; and wind and atmospheric pressure data at several coastal locations. Time series and statistics of surge levels at four coastal locations are shown in Figure 4-16 and Table 4-14 while plots of significant wave height, peak wave period, wind speed and direction are presented in Figure 4-17.

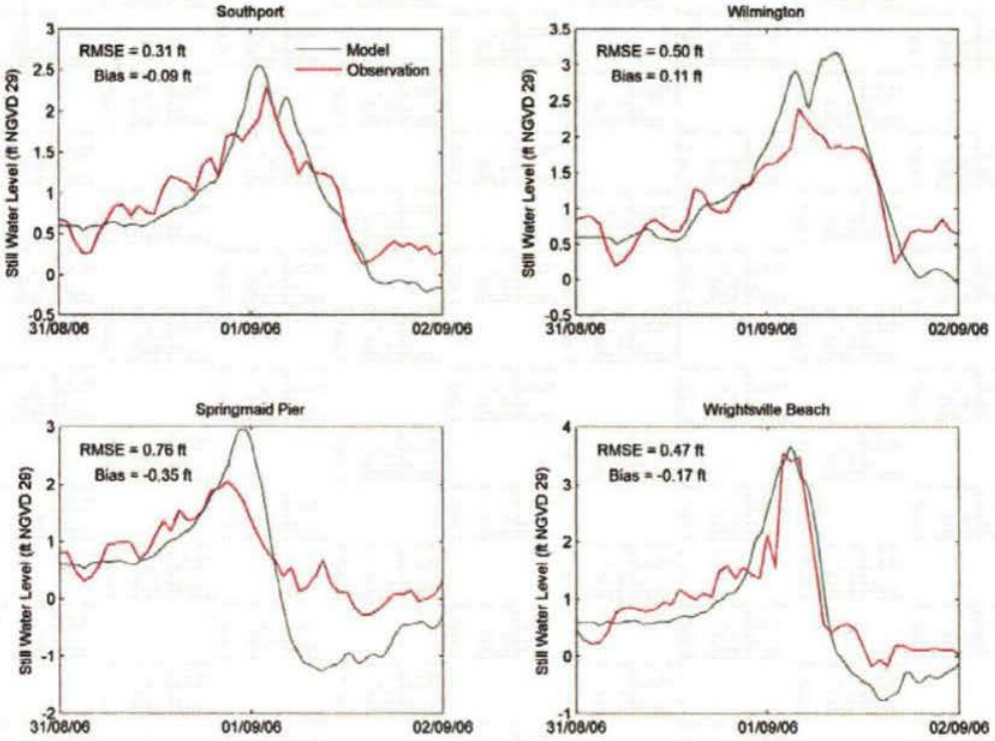

Figure 4-16: Comparison of observed and predicted storm surge levels at four stations on the coast for Hurricane Ernesto. The plots provide indications of the Root Mean Square Error (RMSE) and bias of the mean.

Table 4-14: Model Error Statistics and Peak Surge Values for Hurricane Ernesto

Station	RMSE (ft)	Bias (ft)	Model Max (ft)	Observed Max (ft)
Southport	0.31	-0.09	2.55	2.28
Wilmington	0.5	0.11	3.16	2.39
Springmaid Pier	0.76	-0.35	2.95	2.05
Wrightsville Beach	0.47	-0.17	3.64	3.54

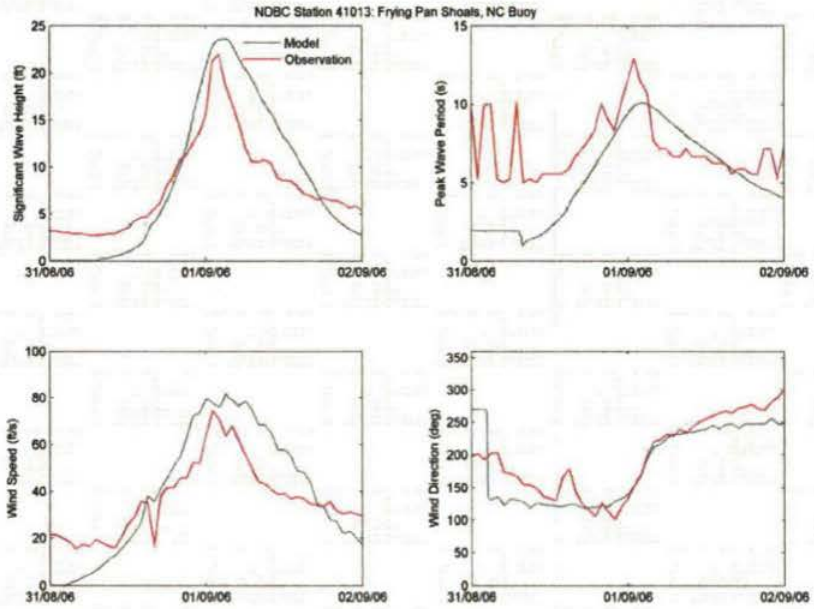

Figure 4-17: Comparison of observed and predicted wave and wind parameters at the Frying Pan Shoals buoy for Hurricane Ernesto

The PMSS is determined from the PMH as defined in Appendix E of NUREG/CR 7046 (Reference 3), using the calibrated hydrodynamic model. As a simplifying assumption, the track of the PMH was assumed to be straight line throughout the storm duration. Conservative simplifying assumptions were made for two of the storm parameters to constrain the number of plausible combinations. For wind speed, it was assumed that higher values would yield higher surge levels. In addition, early tests showed that larger values for the wind radius also yield higher surge levels, due to the energy of a storm increasing with the square of the radius for the same peak winds. As a result, three parameters remained which did not have a straightforward relation to surge levels: forward speed, direction of motion and location of landfall.

The worst-case PMH scenario was determined by screening 79 different scenarios. For landfall locations, scenarios were run in 9-mile increments toward the west, relative to the mouth of the Cape Fear River estuary. Due to the location of the plant, a storm making landfall northeast of the mouth of the estuary would bring its highest surge on the barrier islands north of the site along with northwesterly winds over the site, which would tend to fend off the surge. Most severe surge events were associated with landfalls in the range of 19-37 miles to the west of the mouth of the estuary.

For forward motion, scenarios were run for speeds of 16, 22, 28 and 34 mph. The results showed that the most severe surge events were associated with forward speeds near the middle of the range. For direction at landfall, the full range was considered; however, the impact of direction greatly depended on the other two parameters.

The PMH candidate event that produced the highest surge has a maximum wind speed of 145 knots, radius of 30 mi, forward speed of 30 mi/h, bearing of 310 degrees, landfall point 18.6 mi west of Cape Fear River mouth, and minimum central pressure of 12.96 psi. Figure 4-18 shows the track of the PMH. It should be noted that several storms compared closely to the top candidate producing only a slightly lower total water level at several points near the BNP site.

The topography used in the model reflected bare earth without representation of buildings and structures at the BNP site. Therefore, the effects of flooding and waves on the site for this evaluation are conservative since they do not reflect the dissipation effects that would be expected from on-site buildings and structures.

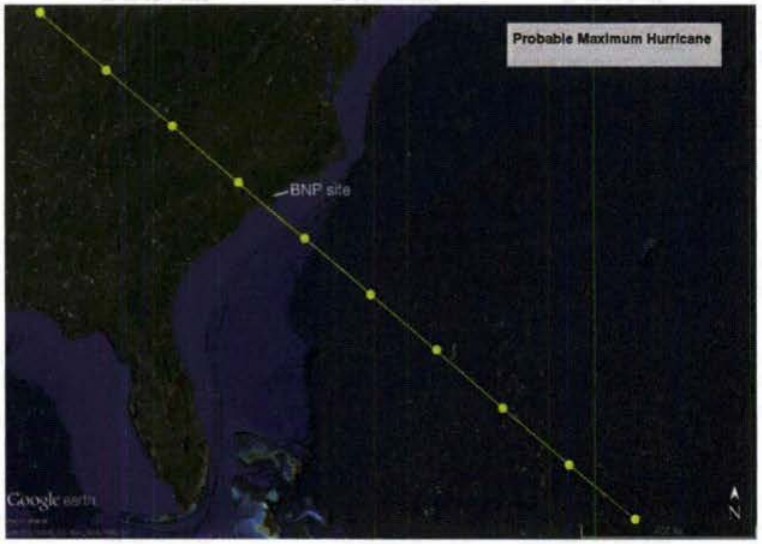

Figure 4-18 PMH track

Figure 4-19 shows the inundation due to the PMH at the BNP site. The composite maximum water level shows the extent of the flooding, covering the northern part of the BNP site and reaching a maximum water level of 21.0 ft north of Unit 1 Turbine Building. At the site, the composite maximum water depth varies spatially with a maximum depth of 1.3 ft over the site grade.

Flooding on site comes from the southwest from Nancys Creek and the discharge canal. As the storm approaches landfall, the water levels along the coast are elevated. The water is then pushed inland, elevating water levels surrounding the BNP site, including Nancys Creek and the Cape Fear River. The flooding during the PMH affects a portion of the site along the northeastern side of the Turbine Building. As the storm leaves the vicinity of the site, water recedes slowly. Due to the topography of the site, water remains trapped on site for several hours before draining.

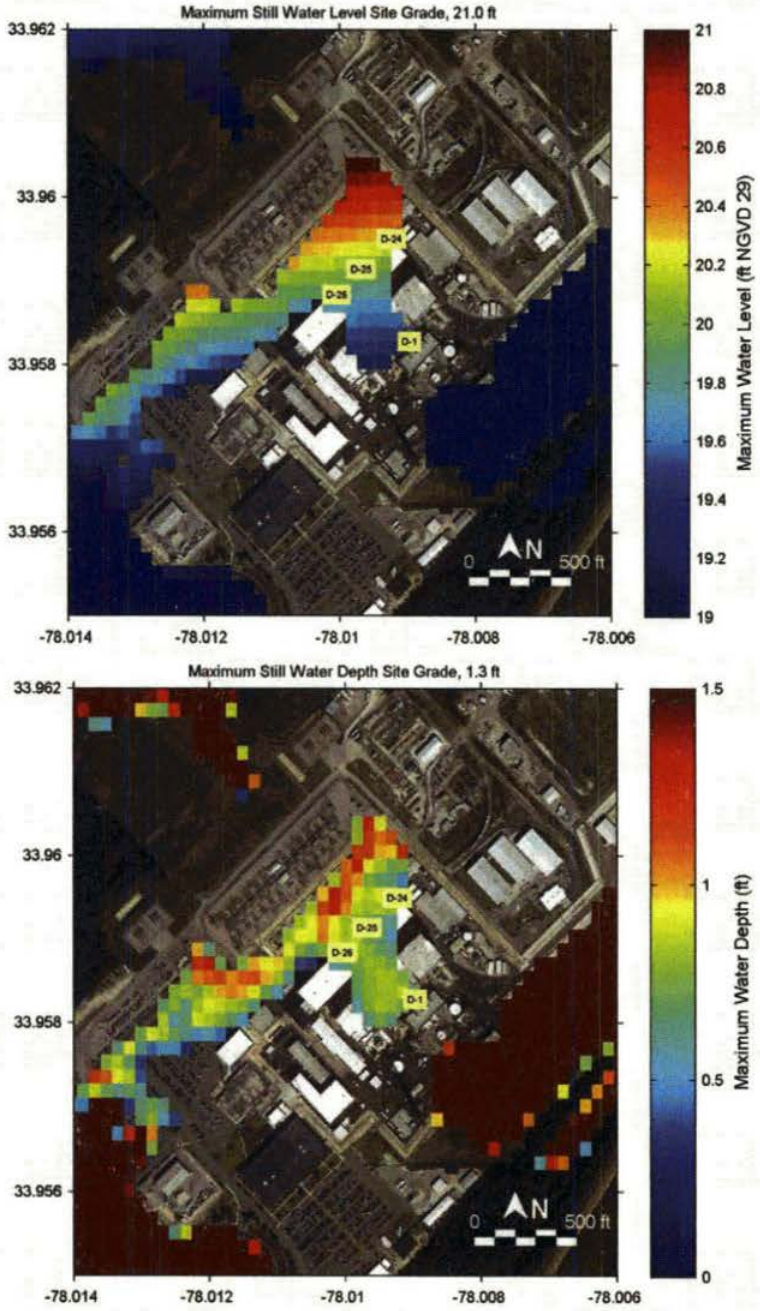

Figure 4-19 Composite water level (upper panel) and water depth (lower panel) at BNP

Table 4-15 summarizes the maximum still water level, flood duration, and flooding above finished floor level for buildings containing SSCs at the door locations shown in Figure 4-2. Table 4-16 summarizes this information for buildings not containing SSCs. The Turbine Building doors are flooded at stillwater level with a maximum depth above finished floor elevation of 0.31 ft at door D-24. The duration of floods vary at each location from 0.55 hours to as long as 1.2 hours. The maximum duration of flooding corresponds to the time when the flood levels in the adjacent water bodies recede below plant grade and only water

trapped by the topography remains. The maximum wave heights at the site are insignificant. The model water depths at the site are not sufficiently deep to allow locally generated waves to develop. The spatial extent of the flooding on the site (Figure 4-19) is confined as well to a small area limiting the fetch that wind can blow to develop significant waves.

Table 4-15: Summary of PMH Results at Buildings Containing SSCs

Door ID	Building	Adjacent Ground Level	Finished Floor Level	Still Water Level	Depth Above Ground Level (Finished Floor Level)	Max Wave Height	Max Wave Run Up[2]	Flood Duration Above Adjacent Ground
		ft (NGVD29)			ft			hr
D-2	Reactor Building	20.00	20.00	-	-	-	-	-
D-3		19.94	20.00	-	-	-	-	-
D-4	Diesel Generator Building	19.63	23.00	-	-	-	-	-
D-5		21.41	23.00	-	-	-	-	-
D-6		20.02	23.00	-	-	-	-	-
D-7		19.44	23.00	19.92	0.48 (-)	0.00	0.00	0.58
D-8		22.36	23.00	-	-	-	-	-
D-13	Service Water Intake Building	20.47	23.00	-	-	-	-	-
D-14		18.89	23.00	19.14	-	0.00	0.00	0.58

Table 4-16: Summary of PMH Results at Buildings Not Containing SSCs

Door ID	Building	Adjacent Ground Level	Finished Floor Level	Still Water Level	Depth Above Ground Level (Finished Floor Level)	Max Wave Height	Max Wave Run Up[2]	Flood Duration Above Adjacent Ground
		ft (NGVD29)			ft			hr
D-1	Radwaste Building	19.01	23.00	19.61	0.6 (-)	0.00	0.00	1.20
D-9	AOG Building**	19.54	22.33	-	-	-	-	-
D-10		19.81	22.33	-	-	-	-	-
D-11		19.81	22.33	-	-	-	-	-
D-12		19.67	22.33	-	-	-	-	-
D-15[1]	Flex Storage Building	28.39	30.17	-	-	-	-	-
D-16[1]		28.54	30.17	-	-	-	-	-
D-17	Turbine Building	19.88	20.00	-	-	-	-	-
D-18		19.91	20.00	-	-	-	-	-
D-19		19.87	20.00	-	-	-	-	-
D-20		19.85	20.00	-	-	-	-	-
D-21		19.63	20.00	-	-	-	-	-
D-22		19.94	20.00	20.23	0.29 (0.23)	0.00	0.00	1.20
D-23		19.94	20.00	20.23	0.29 (0.23)	0.00	0.00	1.20
D-24		19.87	20.00	20.31	0.44 (0.31)	0.00	0.00	1.20
D-25		19.89	20.00	20.10	0.21 (0.10)	0.00	0.00	1.20
D-26		20.00	20.00	-	-	-	-	-

[1] Location is not shown in Figure 4-2, [2] Max wave run-up = 2.5 * Max wave height (Reference 41

** AOG Building is a safety-related structure not containing equipment requiring flood protection

e. Tsunami

Probable Maximum Tsunami (PMT) flood elevation resulting from a landslide-generated tsunami event, a volcanic cone collapse, and a subduction zone co-seismic event was evaluated and estimated.

Potential sources of tsunami hazard for the U.S. East Coast have been recently reviewed by Brink et al (Reference 33). This resource and further studies of individual events also served as the basis for performing a tsunami hazard assessment and the development of inundation and evacuation maps for the U.S. East Coast regions by the National Tsunami Hazard Mitigation Program (NTHMP). The NTHMP has developed low-resolution estimates of tsunami impact along the entire U.S. East Coast for a number of events that could qualify as PMT's for the region. Figure 4-20 illustrates such estimates for three events: a large volcanic cone collapse in the Canary Islands, referred to as the Cumbre Vieja Volcano (CVV), with a failure volume of 450 km^3 (108 mi^3); a smaller CVV event with a failure volume of 80 km^3 (19 mi^3); and a continental margin landslide off the coast of Maryland/Delaware with characteristics modeled after the Currituck Slide, studied in detail in ten Brink et al (Reference 33). The region around Cape Fear, encompassing the BNP, is located around the 1600 km mark in the plot on the left in Figure 4-20, where the axis represents along-coast distance starting from southern Florida. The figure indicates that the region is a

relative hot spot, generally due to refractive focusing of waves by the offshore features of Cape Fear. However, the orientation of the coast in this location provides a shelter compared to the Outer Banks region of North Carolina, which is relatively more exposed and fronted by a much narrower continental shelf.

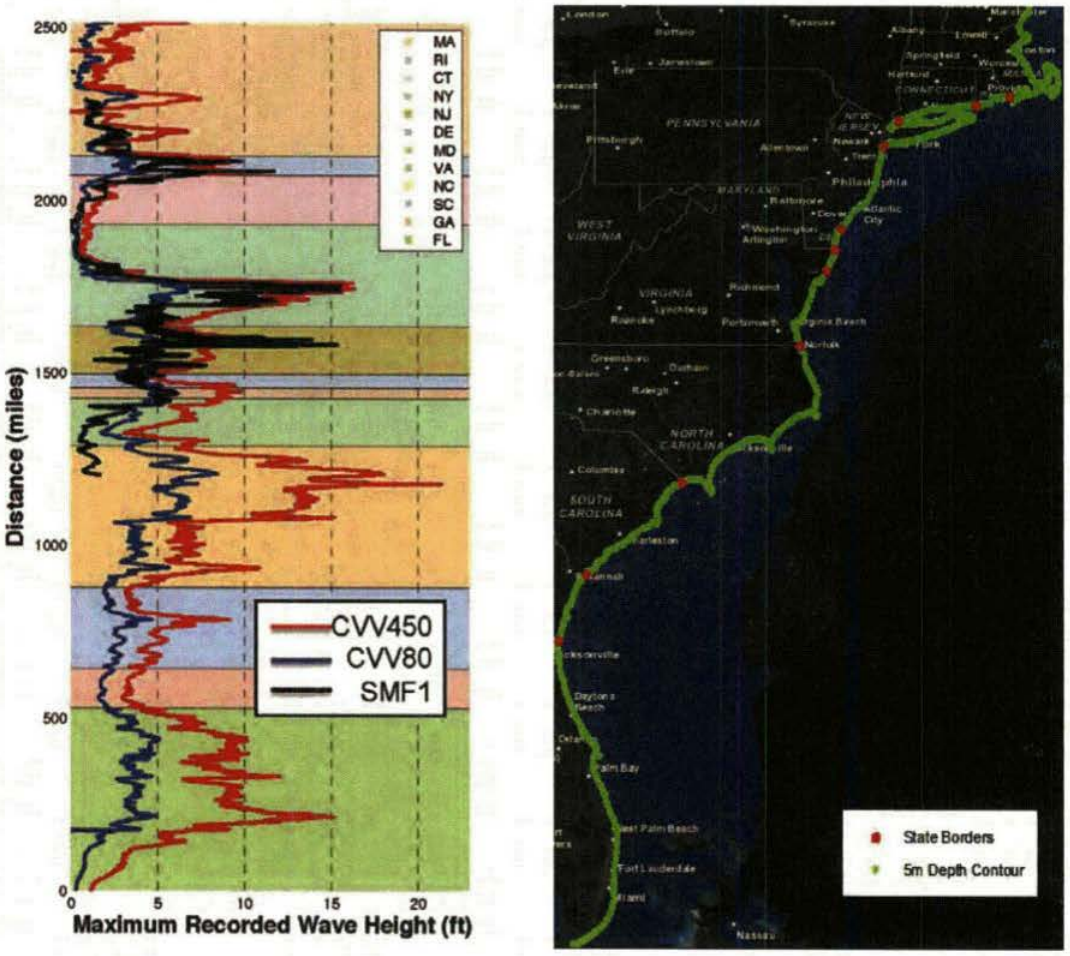

Figure 4-20: Comparison of Nearshore Tsunami Wave Heights along U.S. East Coast for a Large CVV, Small CVV and Currituck-Like Submarine Landslide Event

The tsunami evaluation utilized the NTHMP procedures in several ways. First, the modeling methodology was identical, starting with the initial generation of tsunami waves to the final high resolution simulation of coastal inundation, and computations were carried out using the same set of models. Secondly, for distant tsunami sources, the study used archived model results stored for general usage at the outer boundary of Grid A, represented by the entire region shown in Figure 4-21. This approach allowed for performing high-resolution inundation studies using distant sources - CVV and a subduction zone event in the Puerto Rico (PR) Trench –without the need to reproduce the detailed modeling of the CVV volcanic collapse (Grilli and Grilli, Reference 34) or the ocean basin-scale propagation calculations for each of the events.

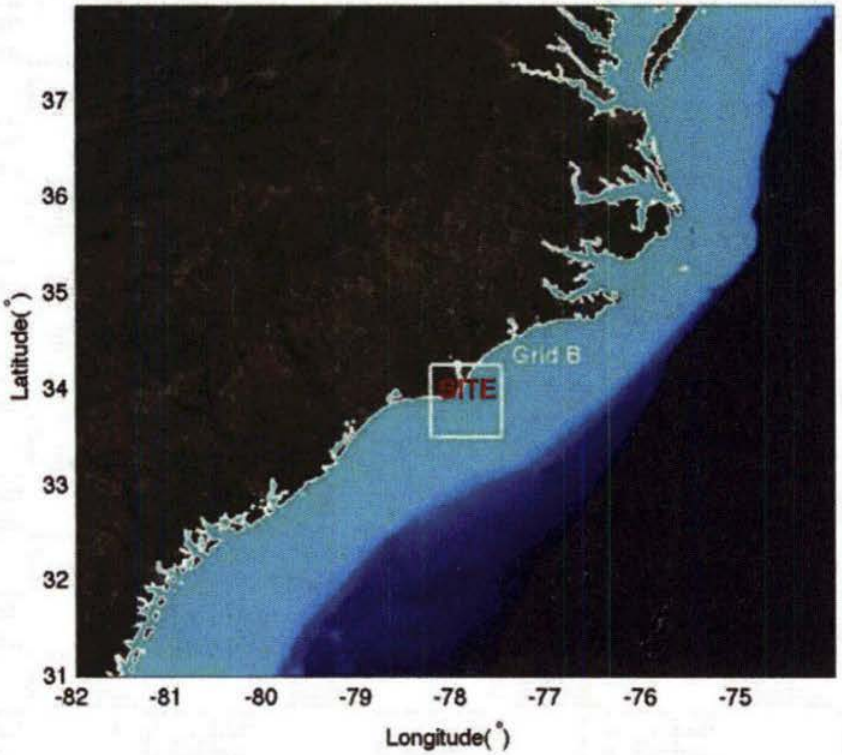

Figure 4-21: The Extent of Grid A (W82°-W74°,N31°-N38°) for CVV and PR Simulations

The flood level due to tsunami inundation near the BNP site was estimated using the numerical wave model FUNWAVE-TVD (Fully Nonlinear Wave – Total Variation Diminishing Scheme), Version 2.1 (Reference 37). For landslide sources, the modeling approach employed a dynamic source, which specifies both surface displacement and depth-varying horizontal velocity fields. This source was computed from the slide geometry using the model NHWAVE (Non-Hydrostatic Wave), Version 1.1 (Reference 38).

Table 4-17 presents the results for the Cape Fear landslide case, the two versions of the Cumbre Vieja Volcano source, and the Puerto Rico Trench co-seismic source. Numerical wave gage 3 is located at the Intake Canal, while gages 1 and 2 are located to the south of BNP at the Cape Fear River estuary (Figure 4-22). Based on the results presented in Table 4-17, none of the potential sources are able to produce tsunami events which would cause water surface elevations to rise above the BNP plant grade of 20 ft NGVD29. The larger CVV event produces the highest simulated water surface runup at the Intake Canal (gage 3) of 10.21 ft NGVD29. The maximum simulated water surface runup of 14.98 ft NGVD29 associated with this event was simulated at gage 1; however, due to the shore topography the tsunami waves do not have the ability to reach the BNP site. Figure 4-22 and Figure 4-23 present the visual representation of the tsunami runup and the time series of the tsunami runup resulting from the larger CVV event. The Cape Fear slide event, which represents an estimate of a large-scale continental shelf margin failure, also produces tsunami runup values which would significantly impact the Cape Fear area but which fall short of inundating the plant site. The smaller CVV event and the PR Trench event produce significantly less runup than the two larger events.

Table 4-17: Maximum Tsunami Water Surface Elevations in the Vicinity of BNP

	Gage 1	Gage 2	Gage 3
	WSEL, ft NGVD29		
CVV (450 km³)	14.98	13.25	10.21
CVV (80 km³)	7.49	6.38	5.03
PR Trench	6.14	5.5	4.18
Cape Fear Slide	10.76	9.29	9.2

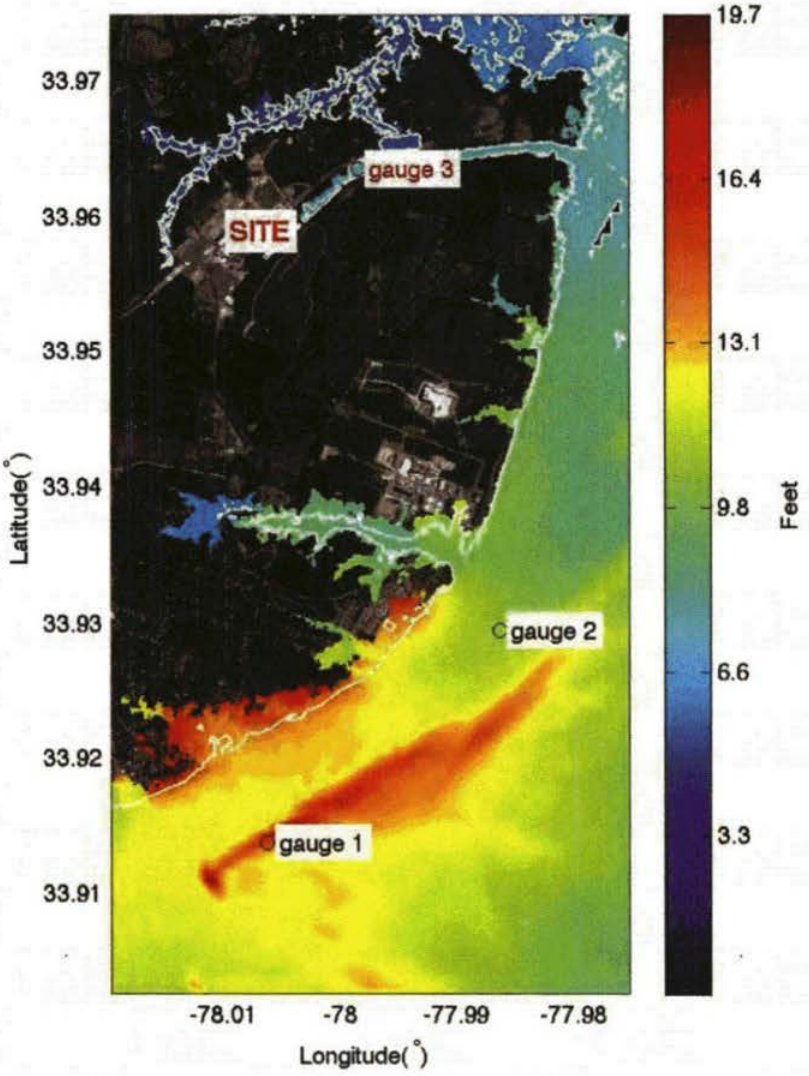

Figure 4-22: Maximum Water Surface Elevations (ft NGVD29) during the CVV (450 km³) Event in Grid D

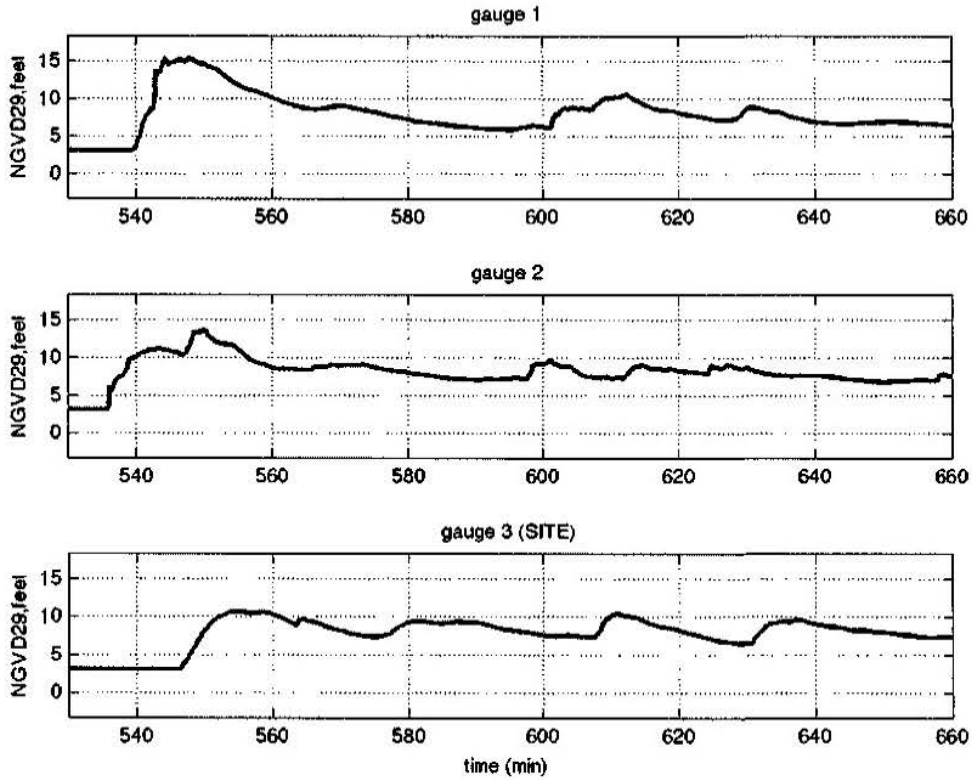

Figure 4-23: Time Series of Water Surface Elevations for the CVV (450 km³) Event

Based on the results presented above, PMT contribution to potential flooding of BNP is not plausible, even when events which fall at the extreme high end of possible Atlantic Ocean tsunami events are considered.

Because there is no indication of tsunami waves impacting the site, further analysis of velocities or debris impacts that could potentially damage the plant facilities during an inundation event was not performed.

f. Ice Induced Flooding

Ice jams and ice dams can cause flooding by impounding water upstream of a site and subsequently collapsing or downstream of a site impounding and backing up water. There is no method to assess a probable maximum ice jam or ice dam. Therefore, historical records are generally used to determine the most severe historical event in the vicinity of the site. The USACE Ice Jam Database was reviewed and there are no historic occurrences of ice jams identified along the Cape Fear River. In addition, based on a review of available water temperature data for the Cape Fear River, water temperature at all stations along the Cape Fear River in 2005 ranged from 3.7°C (38.7°F) to 31.1°C (88.0°F) (Reference 31). Available water temperature measurements from the NOAA National Buoy Center stations near the site were also reviewed with the lowest available measured temperature of 10.2°C (50.4°F) in 2013 recorded at Wilmington, NC (Station WLON7) (Reference 32). This range is safely above the freezing temperature and therefore, the formation of fragile ice is not expected to occur. Based on the historic review of ice jam occurrences and water temperature, ice induced flooding or blockage of the Intake Structure is not expected to occur and therefore this flood causing mechanism is not considered applicable to the site.

g. Channel Migration or Diversion

The flood hazard associated with channel diversion is due to the possible migration either toward the site or away from it. For natural channels adjacent to the site, historical and geomorphic processes should be reviewed for possible tendency to meander. For man-made channels, canals or diversions used for the conveyance of water located at a site, possible failure of these structures should be considered.

As indicated in NUREG/CR-7046 Section 3.8 (Reference 3), there are no well-established predictive models for channel diversions and, therefore, it is not possible to postulate a probable maximum channel diversion event. A qualitative evaluation of the tendency for Nancys Creek and the Cape Fear River to meander was based on a review of available historical data. The channel alignment of Nancys Creek and the Cape Fear River in the vicinity of the BNP site appears stable and has not shown a historical tendency to meander. Furthermore, hydrogeomorphological data was also used to determine whether Nancys Creek or the Cape Fear River exhibited the tendency to meander towards the site. Maximum flow velocities in Nancys Creek and in the Cape Fear River were compared to the permissible velocities of the channel banks based on soil and vegetation types.

For Nancys Creek, the flow velocity near the site was calculated to be 3.88 ft/s. The soils near Nancys creek are mapped as Longshoal muck. Longshoal series consists of very deep, very poorly drained soils with moderately rapid permeability. These soils formed in thick herbaceous plant remains over marine and fluvial sediments and are in brackish coastal marshes of the Lower Coastal Plain. Additionally, the area appears to be covered with grass/herbaceous cover. These conditions have permissible velocity of 3-4 ft/sec and the calculated velocity in Nancys Creek is within the permissible velocities of the channel banks minimizing any potential for significant erosion.

The average velocities of flow for the Cape Fear River from the cross section at the BNP intake is 5.31 ft/s in the main channel (the deep, dredged channel), 0.65 ft/s in the left overbank area, 0.77 ft/s in the right overbank area, and 1.95 ft/s in the total cross section. The banks of the Cape Fear have limited vegetative cover and the soils adjacent to the Cape Fear River are Bohicket silty clay loam. The Bohicket series consists of very poorly drained, very slowly permeable soils that formed in marine sediments in tidal marshes. These soils are flooded twice daily by sea water. These conditions have a permissible velocity of 1.75 – 2.25 ft/sec and the calculated velocities in the overbank areas for the Cape Fear River are well within the permissible velocities minimizing any potential for significant erosion.

In addition, North Carolina State Port Authority reports that the Cape Fear River is dredged to maintain a 42-foot deep navigational channel (Reference 7), which further minimizes any potential for channel migration. Maintenance dredging and improvements to this channel occur regularly and are ongoing reducing the likelihood of channel migration away from or toward the site.

h. Combined-Effect Floods

The combined-effect floods were evaluated for the critical combination of floods caused by precipitation events (Alternative 1) and for three applicable combinations of floods along the shores of open and semi-enclosed bodies of water, as recommended by NUREG/CR-7046, Appendix H.1 and H.3.2 (Reference 3), respectively.

Floods Caused by Precipitation Events

As indicated in the Section 4.c [redacted] (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F) [redacted] Waves induced by the 2-year wind speed applied along the critical direction were evaluated on top of the Alternative 1 scenario per NUREG/CR-7046, Appendix H.1.

The 2-year wind speed of 50 mph was obtained from Figure 1 of ANSI/ANS-2.8-1992. The significant wave height was calculated to be 1.33 ft. The maximum water surface elevation of the combined Alternative 1 stillwater elevation and the crest of the significant wave height is [redacted] ft NGVD29, which will not challenge the site. (b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

Floods along the Shores of Open and Semi-Enclosed Bodies of Water

Combined sea-related and riverine flooding was determined for the following alternative combinations for a streamside location, as defined in Appendix H.3.2 of NUREG/CR-7046 (Reference 3):

- Alternative 1
  - One-half PMF or 500-year flood in the stream, whichever is less.
  - Surge and seiche from the worst regional hurricane or windstorm with wind-wave activity.
  - Antecedent 10% exceedance high tide.
- Alternative 2
  - PMF in the stream.
  - 25-year surge and seiche with wind-wave activity.
  - Antecedent 10% exceedance high tide.
- Alternative 3
  - 25-year flood.
  - Probable maximum surge and seiche with wind-wave activity.
  - Antecedent 10% exceedance high tide.

The PMS is produced by the PMH. Similarly, the worst regional hurricane is considered to be conservatively represented by the standard project hurricane (SPH) defined in the TR23 guidance document (Reference 39), in which the storm track characteristics are consistent with the PMH, but with a lower central pressure and wind speed.

The combined sea-related and riverine flooding effect were simulated in the Delft3D model. The resulting combined effects from the combined hurricane and riverine flooding were calculated conservatively by assuming that the peak storm surges coincide with the peak riverine flow at the site. Therefore, all modeled alternatives represent the upper limit of the combined sea-related and riverine flooding in their respective category.

Alternative 1 and 3 were calculated based on the SPH and PMH events, respectively. Alternative 2 was calculated assuming that the combined effects of PMF at 10% exceedance high tide, and the 25-year surge and seiche add up linearly. The 25-year surge of 4.70 ft NGVD29 is the maximum of the 25-year surge levels calculated at the Southport and Wilmington tide gages using extremal analysis.

From the scenarios that inundate the site, derived quantities such as wave run-up and flood duration at the doors of interest were also estimated. The wave runup was computed based on the wave height at the structure toe. The upper limit of wave runup on a smooth vertical slope was estimated as 2.5 times the wave height at the toe structure. The maximum duration of flooding corresponds to the time when the flood levels in the adjacent water bodies recede below plant grade and only water trapped by the topography remains.

Alternatives 1 and 2 did not result in flooding of safety-related structures (Figure 4-24b and Figure 4-24c, respectively). Alternative 3 (Figure 4-24d) is the governing combined-effect scenario, which results in flooding of the entire BNP site. The composite maximum water levels (Figure 4-25) show the extent of the

flooding with maximum water levels of 22.9 ft NGVD29 north of Unit 1 Turbine Building. The composite maximum water depths vary spatially with a maximum value of 3.9 ft in the vicinity of Radwaste Building.

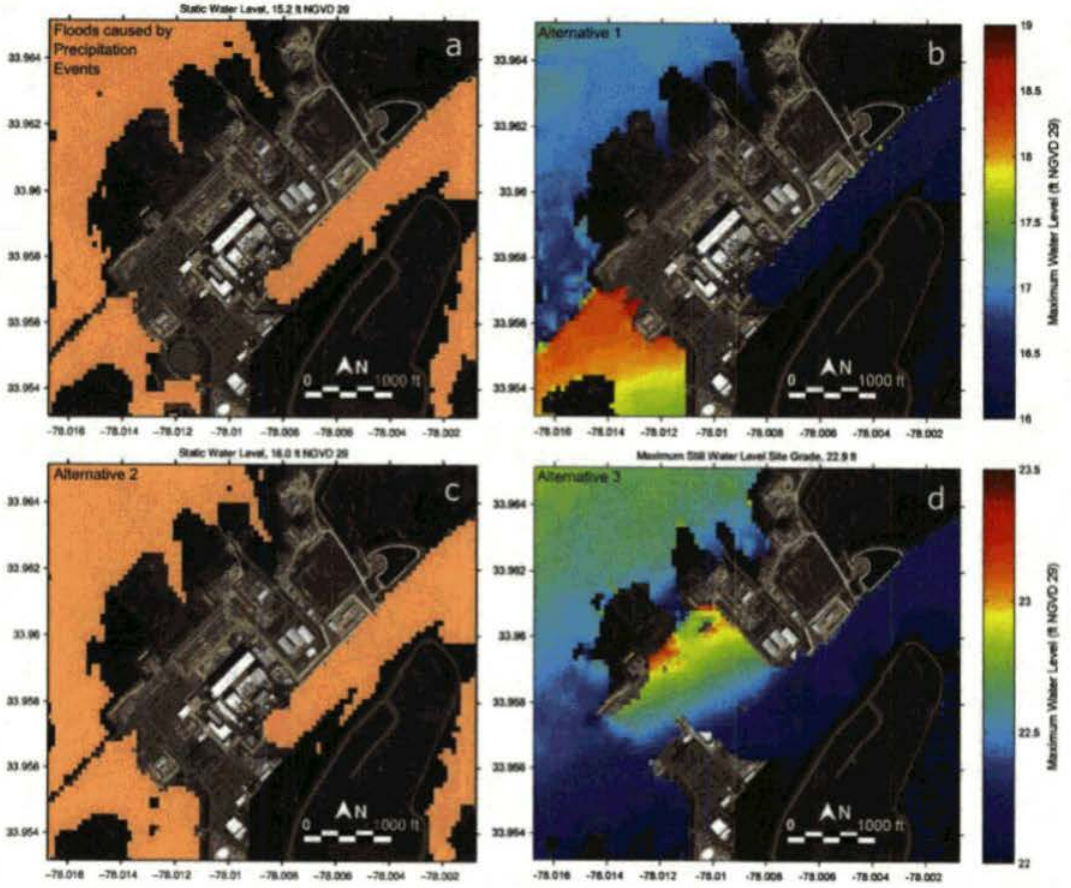

Figure 4-24: Calculated Maximum still water level for the 4 scenarios considered. A) and c) Static water level for the case of riverine flooding combined events PMF and dam failure scenarios and Alternative 2 scenario, and b) and d) Composite maximum water level calculated for Alternative 1 and 3 scenarios.

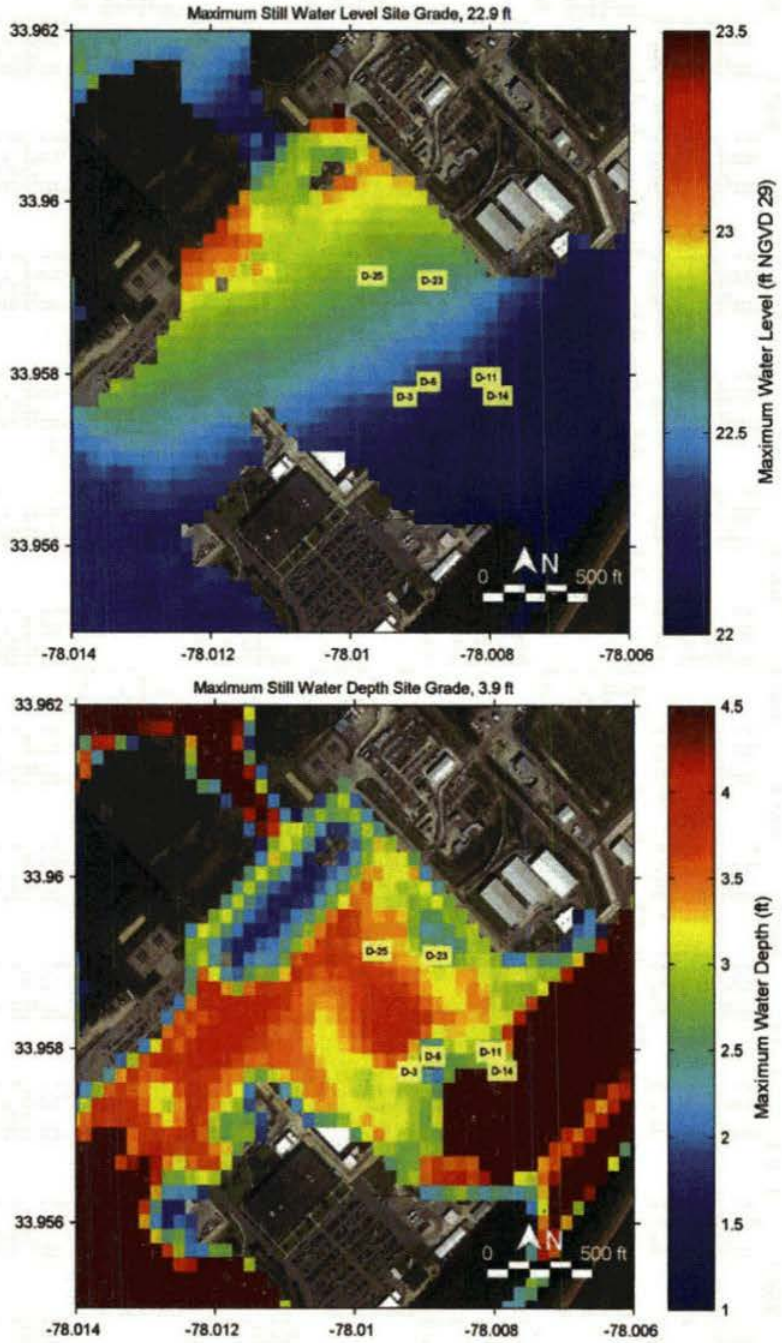

Figure 4-25: Composite water level (upper panel) and water depth (lower panel) at BNP. Locations of 6 doors are shown.

Table 4-18 summarizes the maximum still water level, flood duration, and flooding above finished floor level at the door locations for buildings containing SSCs shown in Figure 4-2. Table 4-19 summarizes these

results for buildings not containing SSCs. The maximum still water depths above finished floor elevation are predicted at door D-24 (2.78 ft). Maximum wave run-up of 4.47 ft is calculated at several locations.

Table 4-18: Summary of the Governing Combined-Effect Floods (Alternative 3) Results at Buildings Containing SSCs

Door ID	Building	Adjacent Ground Level	Finished Floor Level	Still Water Level	Depth Above Ground Level (Finished Floor Level)	Max Wave Height	Max Wave Run Up	Flood Duration Above Adjacent Ground	Max Wave Height Instantaneous Water Surface Elevation	Max Wave Run Up Instantaneous Water Surface Elevation
		ft (NGVD29)			ft			hr	ft (NGVD29)	
D-2	Reactor Building	20.00	20	22.46	2.46 (2.46)	1.76	4.40	3.67	23.34	26.86
D-3		19.94	20	22.42	2.48 (2.42)	0.92	2.30	1.35	22.88	24.72
D-4	Diesel Generator Building	19.63	23	22.50	2.87 (-)	1.76	4.40	3.67	23.38	26.90
D-5		21.41	23	22.40	0.99 (-)	1.73	4.32	0.5	23.27	26.72
D-6		20.02	23	22.39	2.37 (-)	0.92	2.30	3.67	22.85	24.69
D-7		19.44	23	22.39	2.95 (-)	0.92	2.30	0.83	22.85	24.69
D-8		22.36	23	22.36	-	-	-	-	-	-
D-13	Service Water Intake Building	20.47	23	22.16	1.69 (-)	0.92	2.29	0.92	22.62	24.45
D-14		18.89	23	22.18	3.29 (-)	0.92	2.30	3.67	22.64	24.48

Table 4-19: Summary of the Governing Combined-Effect Floods (Alternative 3) Results at Buildings Not Containing SSCs

Door ID	Building	Adjacent Ground Level	Finished Floor Level	Still Water Level	Depth Above Ground Level (Finished Floor Level)	Max Wave Height	Max Wave Run Up	Flood Duration Above Adjacent Ground	Max Wave Height Instantaneous Water Surface Elevation	Max Wave Run Up Instantaneous Water Surface Elevation
		ft (NGVD29)			ft			hr	ft (NGVD29)	
D-1	Radwaste Building	19.01	23	22.48	3.47 (-)	1.79	4.47	3.67	23.38	26.95
D-9	AOG Building**	19.54	22.33	22.38	2.84 (0.05)	1.79	4.47	3.67	23.28	26.85
D-10		19.81	22.33	22.27	2.46 (-)	1.79	4.47	1.33	23.17	26.74
D-11		19.81	22.33	22.28	2.47 (-)	0.92	2.30	1.17	22.74	24.58
D-12		19.67	22.33	22.38	2.71 (0.05)	1.79	4.47	3.67	23.28	26.85
D-15[1]	Flex Storage Building	28.39	30.17	27.75	-	-	-	-	-	-
D-16[1]		28.54	30.17	28.14	-	-	-	-	-	-
D-17	Turbine Building	19.88	20	22.56	2.68 (2.56)	1.79	4.47	3.67	23.46	27.03
D-18		19.91	20	22.56	2.65 (2.56)	1.79	4.47	3.67	23.46	27.03
D-19		19.87	20	22.55	2.68 (2.55)	1.79	4.47	3.67	23.45	27.02
D-20		19.85	20	22.56	2.71 (2.56)	1.79	4.47	3.67	23.46	27.03
D-21		19.63	20	22.70	3.07 (2.70)	1.76	4.40	3.67	23.58	27.10
D-22		19.94	20	22.67	2.73 (2.67)	1.42	3.56	3.67	23.38	26.23
D-23		19.94	20	22.67	2.73 (2.67)	1.42	3.56	3.67	23.38	26.23
D-24		19.87	20	22.78	2.91 (2.78)	1.51	3.77	3.67	23.54	26.55
D-25		19.89	20	22.77	2.88 (2.77)	1.42	3.56	3.67	23.48	26.33
D-26		20.00	20	22.71	2.71 (2.71)	1.70	4.26	3.67	23.56	26.97

[1] Location is not shown in Figure 4-2, [2] Max wave run-up = 2.5 * Maximum significant wave height (Reference 41)

** AOG Building is a safety-related structure not containing equipment requiring flood protection

The associated effects (e.g., hydrostatic and hydrodynamic forces, and debris impact loads) during the combined-effect floods were also calculated. Table 4-20 provides a summary of the predicted hydrostatic and hydrodynamic forces and debris impact loads.

Table 4-20: Maximum Predicted Hydrostatic and Hydrodynamic Forces and Debris Impact Loads for Buildings Containing SSCs

Door ID	Building	Maximum Hydrostatic Force	Maximum Hydrodynamic Force	Debris Impact Load
		lb/ft	lb/ft	lb
Door D-2	Reactor Building	188.66	80.84	2,553.83
Door D-3		191.12	10.19	1,085.92
Door D-4	Diesel Generator Building	256.63	182.92	3,933.75
Door D-5		146.78	235.46	1,912.01
Door D-6		175.25	25.85	1,843.69
Door D-7		271.52	29.06	1,843.69
Door D-8		-	-	-
Door D-13	Service Water Intake Building	89	16.41	1,614.91
Door D-14		337.51	47.83	2,325.05

Table 4-21: Maximum Predicted Hydrostatic and Hydrodynamic Forces and Debris Impact Loads for Buildings Not Containing SSCs

Door ID	Building	Maximum Hydrostatic Force	Maximum Hydrodynamic Force	Debris Impact Load
		lb/ft	lb/ft	lb
Door D-1	Radwaste Building	376.33	194.1	3,675.98
Door D-9	AOG Building**	251.47	49.2	2,227.74
Door D-10		188.96	80.68	2,929.61
Door D-11		190.35	69.05	3,266.05
Door D-12		228.63	25.52	1,619.05
Door D-15	Flex Storage Building	-	-	-
Door D-16		-	-	-
Door D-17	Turbine Building	223.92	65.06	2,243.27
Door D-18		218.94	64.78	2,243.27
Door D-19		223.59	68.5	2,302.28
Door D-20		228.97	65.33	2,243.27
Door D-21		293.29	33.71	1,580.75
Door D-22		232.87	48.82	2,071.43
Door D-23		232.87	48.82	2,071.43
Door D-24		264.75	2.31	436.85
Door D-25		258.07	20.15	1,315.73
Door D-26		229.47	19.29	1,237.06

** AOG Building is a safety-related structure not containing equipment requiring flood protection

Based on a comparison of the maximum water depths around the safety-related structures during the governing combined-effect floods scenario (around 2-3 ft) and typical values of draft for barges in the Cape Fear River (minimum of 6 ft), barges are not expected to impact the safety-related structures.

Safety-related structures are located in asphalt/concrete areas. The surrounding areas are characterized by either gravel/riprap or grass. In the areas immediately surrounding the critical structures, the maximum flow velocities for the governing combined-effect flood scenario are generally below 5 ft/s. No erosion is expected for asphalt/concrete because the maximum values of flow velocity that can be sustained without significant erosion are at least an order of magnitude higher than those expected during the governing combined-effect flood scenario. Furthermore, the recommended maximum permissible mean channel velocity for fine gravel per EM 1110-2-1601 (Reference 42) is 6 ft/s. Gravel mobilization could potentially occur in shallow gravel areas further away from the safety-related structures; however, no entrainment and transport in suspension is expected.

According to a study of the sediment characteristics of North Carolina streams (Reference 43), the sediment material transported in suspension during high flows is expected to be fine, with a median size of 4 μm, corresponding to mud. Therefore, significant deposition of suspended sediments would not be expected to occur and change flow patterns during peak flows, because the flow turbulence would keep the fine material in suspension, preventing it from settling to the bottom. Sedimentation is more likely to occur when velocities decrease during the receding limb of the hydrograph.

5. COMPARISON WITH CURRENT DESIGN BASIS FLOOD HAZARD

The current design basis and reevaluated flood causing mechanisms at the site were compared to assess whether the reevaluated flood hazard is bounded by the current design basis flood elevation. The comparison is provided in Table 5-1.

Table 5-1: Summary Comparison with Current Design Basis Flood Hazard

Flood Causing Mechanism	Current Design Basis Flood Hazard Elevation	Flood Hazard Reevaluation Maximum Water Surface Elevation Results	Design Basis Bounds Reevaluation Flood Hazard?
Local Intense Precipitation	Not Applicable	LIP maximum flood depth above finished floor elevation adjacent to a safety-related structure is 1.07 ft corresponding to water surface elevation of 21.07 ft NGVD29. LIP maximum flood depth above finished floor elevation is 1.69 ft corresponding to water surface elevation of 21.69 ft NGVD29. LIP maximum water surface elevations vary on site and are provided in Table 4-2 4-2 and Table 4-3.	No
Flooding in Streams and Rivers	Not Applicable	Cape Fear River – Peak water surface elevation is 11.86 ft NGVD29, which is 8.14 ft below the average plant grade. Nancys Creek – Peak water surface elevation is 15.46 ft NGVD29, which is 4.54 ft below the average plant grade.	No
Dam Breaches and Failures – Overtopping Failure	Not Applicable	Cape Fear River – Peak water surface elevation i[redacted] ft NGVD29, which is [redacted] ft below the average plant grade. Nancys Creek – No dams in watershed and therefore not applicable.	No
Dam Breaches and Failures – Seismic Failure	Not Applicable	Cape Fear River – Peak water surface elevation is [redacted] ft NGVD29, which is [redacted] ft below the average plant grade. Nancys Creek – No dams in watershed and therefore not applicable.	No

(b)(3):16 U.S.C. 824o-1(d);(b)(4);(b)(7)(F)

(b)(3):16 U.S.C. 824o-1(d);(b)(4);(b)(7)(F)

Flood Causing Mechanism	Current Design Basis Flood Hazard Elevation	Flood Hazard Reevaluation Maximum Water Surface Elevation Results	Design Basis Bounds Reevaluation Flood Hazard?
Storm Surge & Seiche	PMH coinciding with peak local astronomical tide elevation of 22.0 ft NGVD29, resulting in a depth of flooding of 2 ft above the finished floor elevation on site.	PMH results in a maximum water surface elevation of 20.31 ft NGVD29, resulting in a depth of flooding of 0.44 ft and 0.31 ft above finished floor elevation. PMH water surface elevations vary on site and are provided in Table 4-15 and Table 4-16.	Yes
Tsunami	Not Applicable	Maximum Tsunami runup at the Intake Canal is 10.21 ft NGVD29, which is 9.79 ft below the average plant grade.	No
Ice Induced Flooding	Not Applicable	Not Applicable	Yes
Channel Migration or Diversion	Not Applicable	Not Applicable	Yes
Combined-Effect Floods	PMH coinciding with peak local astronomical tide elevation of 22.0 ft NGVD29, resulting in wave height of 1.6 ft and wave runup on vertical structures of 3.6 ft. The maximum instantaneous water surface elevation due to wave runup on top of the stillwater elevation is 25.6 ft NGVD29.	PMH with 25-year flood in the Cape Fear River and 10% exceedance high tide results in maximum stillwater elevation adjacent to a safety-related building of 22.50 ft NGVD29 and a depth of flooding of 2.87 ft above ground level. The corresponding maximum wave height is 1.76 ft and the wave runup on vertical structures is 4.4 ft. The maximum instantaneous water surface elevation due to wave height with the stillwater elevation is 23.38 ft NGVD29. The maximum instantaneous water surface elevation due to wave runup on top of the stillwater elevation is 26.9 ft NGVD29. The combined-effect floods elevations, depths, maximum wave heights, and runup vary on site and are reported in Table 4-18 and Table 4-19.	No

6. INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED TO ADDRESS ANY HIGHER FLOODING HAZARDS RELATIVE TO THE DESIGN BASIS

The NRC 10 CFR 50.54(f) Request for Information letter dated March 12, 2012 (Reference 2) provides that flood hazard reevaluations are performed using updated flooding hazard information and present-day regulatory guidance and methodologies. Vulnerabilities identified during the flood hazard reevaluations were entered into the corrective action process and will be dispositioned accordingly.

a. Interim Corrective Actions to mitigate the Beyond Design Basis LIP values are as follows:

Based on the LIP beyond design basis flooding levels being below current flooding protection, no interim actions for these buildings are being considered.

The LIP evaluation results in a maximum flood level of 1.69 ft above finished floor elevation corresponding to water surface elevation of 21.69 ft NGVD29. UFSAR Section 2.4.5.2, Surge and Seiche Water Levels states the following: "From the open coast, the surge water level propagation up the Cape Fear River into the intake canal was evaluated with a resultant level of 24 feet Mean Low Water (MLW) or 22 feet Mean Sea Water (MSL). The nominal plant grade of 20 feet MSL results in two feet of water depth surrounding the plant during maximum surge conditions. All of the safety-related structures are waterproofed to elevation 22 feet MSL. For example, personnel and equipment access doors are provided with sills above the 22-foot still water level, or alternatively are equipped with positive seals and closure devices when the sills are below 22 feet MSL."

b. Interim Corrective Actions to mitigate the Beyond Design Basis PMF values are as follows:

The PMF re-evaluation resulted in a maximum stillwater level adjacent to a safety-related building of 22.5 ft NGVD29. This represents an approximately 2.5% increase above the CLB levels of 22 ft. Based on available enhanced PMF flood protection as described below, BNP is not considering any further actions for buildings that contain safety related equipment at this time.

During the NTTF 2.3 Flooding Walkdown evaluation BNP considered the potential impact of a small increase in the design basis, as described in NEI 12-07, *GUIDELINES FOR PERFORMING VERIFICATION WALKDOWNS OF PLANT FLOOD PROTECTION FEATURES*, Section 3.12, Cliff Edge Effects. Consideration of a 5% increase above the design basis stillwater level was selected because it was a small increase that would result in a flood elevation of greater than 23 ft. This elevation was considered significant because the Control Building and the Diesel Generator Building entrances and operating floor elevations and the Service Water Building entrance elevation are at 23 ft. The UFSAR states that they are protected from PMF flooding since they are 1 ft above the CLB of 22 ft.

BNP elected to provide enhanced flood protection features for the Control Building, Diesel Generator Building and Service Water Building in the event of an approaching hurricane. Enhanced protection was also provided for the Unit 1 and 2 Reactor Building Railroad airlock doors to provide additional flooding protection margin above the CLB. The enhanced protection is provided in the form of metal "Cliff Edge Barriers" that protect the buildings to elevation 26 ft. When there is the potential for a hurricane to impact the site these barriers are installed at the 23 ft door elevations for the Control Building, Diesel Generator Building and at the 20 ft elevation of the Reactor Building Railroad airlock doors. The Service Water Building has permanent barriers installed in the open ventilation windows and provide protection to an elevation of 28.3 ft. The installation for these barriers is controlled by OAI-68, *Brunswick Nuclear Plant Response to Severe Weather*. Each barrier is uniquely designed for the door location where it will be installed and is staged adjacent to that area. Upon notification of an approaching hurricane (reasonably expected to be 1 to 3 days prior to landfall) the barriers will be installed and left in-place until the threat passes at which time they will be removed and staged for future use.

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