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10 CFR 50.54(f)

CNS-14-028

March 12, 2014

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

Duke Energy Carolina, LLC (Duke Energy)
Catawba Nuclear Station, Units 1 and 2
Docket Numbers 50-413 and 50-414
Renewed License Numbers NPF-35 and NPF-52

Subject: Flood Hazard Reevaluation Report, Response to NRC 10 CFR 50.54(f) Request for Additional 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 No. 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 No. ML12097A509)
3. NRC Letter, *Trigger Conditions for Performing an Integrated Assessment and Due Date Response*, Dated December 3, 2012, (ADAMS Accession No. ML12326A912)
4. Duke Energy Letter, *Catawba Nuclear Station Flooding Walkdown Information Requested by NRC Letter, Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.3, Flooding Walkdowns, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident*, dated March 12, 2012, dated November 27, 2012 (ADAMS Accession No. ML12334A444)

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 external flooding sources and report the results in accordance with the NRC's prioritization plan (Reference 2). The due date established for Catawba was March 12, 2014. Attachment 1 to this letter contains the required Flooding Hazard Reevaluation Report for Catawba.

The attached flood hazard reevaluation report describes the approach, methods and results from the reevaluation of flood hazards at Catawba. 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 Catawba.

The flood hazard reevaluation report shows that some flood levels exceed the Current Licensing Basis (CLB) levels. 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 CLB for existing plants. As such, these differences are not the results of errors within the CLB flood evaluations.

In accordance with Reference 3, an Integrated Assessment (IA) is required if flood levels determined during the flood hazard reevaluation are not bounded by the CLB flood levels. Enclosure 2 of reference 1 specifies that the IA be completed and a report submitted within two years of submitting the Flooding Hazard Reevaluation Report. An IA will be completed and a report submitted no later than March 12, 2016.

As discussed within Reference 1, NRC acknowledged that the current regulatory approach and the resultant plant capabilities, an accident with consequences similar to the Fukushima event is unlikely with nuclear power plants located in the United States. The NRC concluded that the continued plant operation do not pose an imminent risk to the public health and safety. The flooding walkdowns of Catawba CLB 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 Catawba are available, functional and implementable and if necessary, any degraded, nonconforming flood protection features were entered in Catawba Corrective Action Program.

Enclosure 1 of attachment 1 provides a discussion of the interim evaluation and actions taken to address the higher flooding levels relative to the CLB flood levels. These actions will enhance the current capability to maintain the plant in a safe condition during the beyond-design-basis external flooding events that exceed the CLB flood levels and as a result, continued plant operation does not impose an imminent risk to the public health and safety while completing the Integrated Assessment.

This letter contains regulatory commitments as listed in the table provided by Enclosure 1 of attachment 1. The table provides a list of the interim actions taken along with the dates these actions were implemented. Duke Energy commits to not modify any of these interim actions or completion dates without notifying the NRC Project Manager, Jason Paige, in advance.

Should you have any questions concerning this letter or require additional information, please contact Phil Barrett at (803) 701-4138.

I declare under penalty of perjury that the foregoing is true and correct. Executed on March 12, 2014.

Sincerely,

Kelvin Henderson
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Attachment 1- Catawba Flooding Hazard Reevaluation Report
Enclosure 1 to Attachment 1 – Interim Actions Taken

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RGC Date File (CN01RC)

NCMPA-1
PMPA
NCEMC
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Attachment 1

FLOODING HAZARD REEVALUATION REPORT FOR

Catawba Nuclear Station, Units 1 and 2

Docket Numbers 50-413 and 50-414

Renewed License Numbers NPF-35 and NPF-52

CATAWBA NUCLEAR STATION
FLOODING HAZARD REEVALUATION REPORT

RESPONSE TO REQUEST FOR INFORMATION PURSUANT TO TITLE 10 OF THE CODE OF FEDERAL REGULATIONS 50.54 (F) REGARDING RECOMMENDATION 2.1: FLOODING OF THE NEAR-TERM TASK FORCE REVIEW OF INSIGHTS FROM THE FUKUSHIMA DAI-ICHI ACCIDENT

Prepared for:
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Prepared by:
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February 17, 2014

HDR

REPORT VERIFICATION

PROJECT: **CNS FLOODING HAZARD REEVALUATION REPORT**

TITLE: **RESPONSE TO REQUEST FOR INFORMATION PURSUANT TO TITLE 10 OF THE CODE OF FEDERAL REGULATIONS 50.54 (F) REGARDING RECOMMENDATION 2.1: FLOODING OF THE NEAR-TERM TASK FORCE REVIEW OF INSIGHTS FROM THE FUKUSHIMA DAI-ICHI ACCIDENT**

This document has been reviewed for accuracy and quality commensurate with the intended application.

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CNS FLOODING HAZARD REEVALUATION REPORT
RESPONSE TO REQUEST FOR INFORMATION PURSUANT TO TITLE 10 OF THE CODE OF FEDERAL REGULATIONS 50.54 (F) REGARDING RECOMMENDATION 2.1: FLOODING OF THE NEAR-TERM TASK FORCE REVIEW OF INSIGHTS FROM THE FUKUSHIMA DAI-ICHI ACCIDENT

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(b)(3) 16 U.S.C. § 824o-1(d), (b)

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List of Acronyms

1-D, 2-D	one dimensional, two dimensional, etc.
CAP	Corrective Action Program
CFR	Code of Federal Regulations
cfs	cubic feet per second
CMH	Conduit Manholes
CN	curve number
CNS	Catawba Nuclear Station
DTM	Digital Terrain Model
ECR	Engineering Change Request
ft	foot/feet
FERC	Federal Energy Regulatory Commission
GIS	Geographic Information Systems
HEC-RAS	Hydrologic Engineering Center's – River Analysis System
HHA	Hierarchical Hazard Assessment
HMR	Hydrometeorological Report
I and E	Instrumentation and Electrical
ICM	Integrated Catchment Model
in/hr	inch(es) per hour
L	lag time
LIP	local intense precipitation
LPSW	Low Pressure Service Water
mph	miles per hour
mi	miles
msl	mean sea level
NEI	Nuclear Energy Institute
NGVD	National Geodetic Vertical Datum
NRC	Nuclear Regulatory Commission
NSW	Nuclear Service Water
NTTF	Near-Term Task Force

NUREG	Nuclear Regulation
PMF	probable maximum flood
PMP	probable maximum precipitation
PMS	probable maximum storm
SCS	Soil Conservation Service
SNSW	Standby Nuclear Service Water
SNSWP	Standby Nuclear Service Water Pond
SOF	Statement of Fact
SPF	Standard Project Flood
sq ft	square feet
sq mi	square miles
SRM	Staff Requirements Memorandum
SSC	systems, structures, and components
UFSAR	Updated Final Safety Analysis Report
U.S.	United States
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
Yard	CNS Yard

Executive Summary

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

The resulting NTTF report concludes that continued United States (U.S.) nuclear plant operation does not pose an imminent risk to public health and safety and provides a set of recommendations to the NRC. The NRC directed its staff to determine which recommendations should be implemented without unnecessary delay (Staff Requirements Memorandum [SRM] on SECY-11-0093).

The NRC issued its request for information pursuant to Title 10 of the Code of Federal Regulations, Section 50.54(f) (10 CFR 50.54[f]) on March 12, 2012, based on the following NTTF flood-related recommendations:

- Recommendation 2.1: Flooding
- Recommendation 2.3: Flooding

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

1. Gather information with respect to NTTF Recommendation 2.1, as amended by SRM on SECY-11-0124 and SECY-11-0137, and the Consolidated Appropriations Act for 2012, Section 402, to reevaluate seismic and flooding hazards at operating reactor sites.
2. Collect information to facilitate NRC's determination of the need to update the safety-related design basis and systems, structures, and components (SSCs) that are important to protect the updated hazards at operating reactor sites.
3. To collect information to address Generic Issue 204 regarding the flooding of nuclear power plant sites following upstream dam failures.

This report was prepared for the Catawba Nuclear Station (CNS) Generating Plant Units 1 and 2 in response to NTTF Recommendation 2.1 only.

Section 1

Site Information Related to the Flood Hazard

1.1 Detailed Site Information

Catawba Nuclear Station (CNS) is located in northeastern York County, South Carolina, approximately 6 miles north of Rock Hill, South Carolina. The site is located on a peninsula bounded by Beaver Dam Creek to the north, Big Allison Creek to the south, Duke Energy Carolinas, LLC's (Duke Energy) Lake Wylie to the east, and private property to the west. Duke Energy's Wylie Dam and Hydroelectric Station are located approximately 4.8 river miles southeast of the site. Rock Hill, South Carolina; Gastonia, North Carolina; and Charlotte, North Carolina; are the nearest large cities.

The location and description of CNS presented in the Updated Final Safety Analysis Report (UFSAR) Chapter 2 include reference to figures showing the general arrangement, layout, and relevant elevations of the station. The original design of the CNS Yard (Yard) grade was nominally 593.50 feet (ft) (SOF 1.1.1-01) mean sea level (msl) (all elevations referenced in this report are based on National Geodetic Vertical Datum [NGVD] 1929). The mezzanine floor elevation in the Turbine, Auxiliary, and Service Buildings is 594.00 ft msl. Exterior accesses to these buildings are at an elevation of 594.00 ft msl (SOF 1.1.1-02). Section 1.4 provides further discussion of modifications to the Yard grade.

The (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft msl, and the Nuclear Service Water (NSW) and Standby Nuclear Service Water (SNSW) pumps are located in a concrete structure on the south abutment to the dam at elevation 599.50 ft msl and are not subject to flooding or wave effects based on the existing licensing basis (SOF 1.1.1-03).

1.2 Current Design Basis Flood Elevations

1.2.1 Local Intense Precipitation

The flood water elevation due to a maximum local intense precipitation (LIP) (also known as the point probable maximum precipitation [PMP]) has been evaluated for producing local inundation throughout the Cooling Tower Yard, Switchyard, Construction Yard, and the Powerhouse Yard over the life of the generating facility. The inundation analysis is documented in calculation CNC-1114.00-00-0040, Rev 29. The analyses have been performed using a variety of hydraulic routing tools, including the United States Army Corps of Engineers (USACE), HEC-1 model, traditional hand calculation methods combined with computer automated Microsoft Excel spreadsheets to facilitate the performance of the routing calculations. The routing analysis is based on the "Modified Puls" method available in the HEC-1 model. CNC-1114.00-00-0040 has been revised 29 times to incorporate the review of hydrology and hydraulic assumptions and physical changes in Yard areas that have occurred since the calculations were originated in 1978. The latest calculation, Revision 29, addresses the Vehicle Barrier System enhancement project, subterranean barrier modifications, and items noted in the 2.3 Fukushima Near-Term Task Force (NTTF) Flood Walkdown Inspection report (October 2012).

The licensing basis PMP for the CNS site is summarized in Section 2.4.2.3 of the UFSAR and in Section 5 of the 2.3 Fukushima NTTF Flood Walkdown Inspection report. The CNS licensing basis PMP is based on U.S. Weather Bureau Hydrometeorological Report (HMR) 33. A 6-hour (hr) duration PMP, producing 24.1 inches (adjusted period total) and a peak 1-hr rainfall of 9.2 inches, was used to analyze potential inundation in the Yard areas (SOF 1.2.1-01). The HEC-1 model, using the Soil Conservation Service (SCS) dimensionless unit hydrograph method, was used to estimate the precipitation/runoff relationship for each sub-area in the model and routed using the "Modified Puls" method. Revision 22 of the calculation documents that an Excel spreadsheet was developed to facilitate the routing for updating the analysis (SOF 1.2.1-02). The resulting maximum inundation levels in the Powerhouse Yard using the HMR33 PMP are shown in Table 1.2.1-1.

TABLE 1.2.1-1
MAXIMUM POWERHOUSE YARD INUNDATION LEVELS

Analysis Basis	PMP Input to Model HMR 33 (with 0.80 Reduction Factor)
Previous Maximum Yard Inundation Level CNC 1114.00-00-0040 Rev 22 (SOF 1.2.1-03)	593.94 ft msl
CNC 1114.00-00-0040 Rev 28-29 using 2010 Survey and catch basin network Unit 1 Yard Inundation Level (SOF 1.2.1-04)	594.00 ft msl
CNC 1114.00-00-0040 Rev 28-29 using 2010 Survey and catch basin network Unit 2 Yard Inundation Level (SOF 1.2.1-05)	594.80 ft msl

Note: The elevations shown in Table 1.2.1-1 include the effectiveness of the yard drainage system as documented in the UFSAR Section 2.4.2.3.3.

The description of the routing assumptions used in the existing PMP calculations is found in Section 2.4.2.3.1 of the UFSAR and repeated below.

The plant site is provided with a surface water drainage system that is designed and constructed to protect all safety-related facilities from flooding during a local PMP. The drainage system consists of (1) catch basin inlets which are connected by corrugated metal pipes to form several networks and (2) graded areas which permit free surface outflow to Lake Wylie when ponding in the powerhouse yard reaches Elevation 593.50 ft msl. All pipe networks and graded areas discharge at elevations which are higher than Lake Wylie full surface elevation (Elevation 569.40).

To obtain time dependent inflow for the PMP, the site is divided into sub-areas. Inflow to the power block area is due to precipitation which falls on 129 acres, which include the powerhouse yard, buildings, and the construction yard. The switchyard and cooling tower yard sub-areas have been bermed/curbed to route water away from the power block area. Elevations of these berms/curbs are conservatively set by neglecting outflow from the switchyard and cooling tower yard storm drainage systems. Inflow to the switchyard and cooling tower yard sub-areas is due

to precipitation which falls directly on the yards and structures, excluding the cooling towers. The inflow hydrograph for the power block area is determined by combining the inflow hydrograph for the construction yard and powerhouse yard sub-areas. Each hydrograph is based on a SCS dimensionless unit hydrograph. The HEC-1 computer program was used to develop a total hydrograph for a given lag time with lag time (L) being defined as the time in hours from the center of mass of rainfall excess to the peak discharge.

For the construction yard, a hydraulic length of 1,815 ft and an average slope of 1.18 percent was obtained from site topography. The maximum retention is based on the SCS Curve Number 98. Lag time for the construction yard sub-area is 13.4 minutes. The time of concentration for the construction yard is 22.4 minutes. Lag time for the powerhouse yard sub-area was assumed to be zero to approximate an instantaneous time of concentration. Since the time of concentration used to develop the inflow hydrograph is small, the peak 1-hour precipitation (9.2 in.) was distributed according to the USACE procedure for 5-minute durations as shown in Table 1.2.1-2.

TABLE 1.2.1-2
5-MINUTE TIME SERIES FOR PEAK 1-HOUR RAINFALL FOR HYDROMETEOROLOGICAL REPORT 33 (HMR33)

Time (minutes)	Incremental PMP (inches)	Cumulative PMP (inches)
5	0.28	0.28
10	0.37	0.65
15	0.46	1.11
20	0.55	1.66
25	0.83	2.49
30	1.56	4.05
35	2.3	6.35
40	10.1	7.36
45	0.74	8.1
50	0.46	8.56
55	0.37	8.93
60	0.27	9.2

During a local intense PMP, water will pond in the power block area and on the roof of the service building. With the exception of the reactor building, the roofs of safety-related structures are designed so that water flows directly off roofs with no accumulation. A gutter drain system

catches the water and routes it to collection points which discharge directly into the yard drainage system. The reactor building roof drainage system is designed for a rainfall intensity of 5.0 inches per hour (in/hr). Intensities in excess of 5.0 in/hr result in ponding; however, once the water level reaches Elevation 711.34 ft msl, the water flows directly off the roof. The reactor building roof is designed to safely carry live loading due to ponding. In determining the effect of a local intense PMP on the powerhouse yard, it is assumed that water flows directly off the reactor building and service building without ponding or discharging through the roof drainage system. Water which ponds in the power house yard will discharge into catch basins and over the northeast and south ends of the yard that discharge into Lake Wylie. Two types of catch basin inlets are credited for the site. Type II inlets consist of slotted catch basin covers with an effective opening of 0.69 square feet (sq ft). Type I inlets have no slotted cover, but are protected by steel grating on four sides and top. The total effective opening in the grating on any one side is at least equal to the effective opening of the pipe inlet (1.48 sq ft), but may be as much as 3.9 times the effective pipe opening. The open area provided by all four sides of the Type I inlet varies from 4 to 15.6 times the pipe opening as margin for not considering complete blockage by debris accumulation. There are 89 Type I inlets provided in the power block area, but only 80 are required to be operable at any given time. All inlets are connected to corrugated metal pipes which are fully coated with a paved invert. The yard drainage pipes, individually and within a network, are designed using Manning's equation for pipe flowing full. Accumulative totals are used throughout the networks to determine pipe sizes. All pipe gradients are 0.5 percent or greater. Water in the power block area is assumed to rise and fall as a "level pool."

The PMP flood event is routed through the CNS switchyard using the Modified Puls method for an instantaneous time of concentration. The switchyard catch-basin drainage system was considered blocked with no outflow. A curb (Elevation 632.67 ft msl) is provided on the north, south, and east ends of the switchyard to block water from flowing onto the powerhouse yard. In addition, trench plugs are provided at the curb location in all trenches that connect between the switchyard and the powerhouse yard. The flood routing was performed with the HEC-1 computer program using storage and discharge data which are based on the switchyard topography. Storage in the switchyard was estimated by characterizing the yard as an inverted

pyramid with a top area of 16.5 acres and a corresponding apex depth of 1.00 ft below Elevation 632.00 ft msl. When ponding in the switchyard reaches elevation 632.00 ft, sheet outflow over the west side of the yard is routed using a standard weir equation. The natural topography below the overflow area conveys the discharge away from the site.

Similarly to the switchyard runoff calculation, the PMP flood event is routed through the cooling tower yard using the Modified Puls method for an instantaneous time of concentration. The cooling tower yard catch-basin drainage system was considered blocked with no outflow. The area occupied by the cooling towers was neglected for the inflow and storage calculations since all precipitation which falls on the towers was assumed to be contained within the system. An earth berm (Elevation 620.50 ft msl) is provided on the northwest end of the cooling tower yard to route water away from the powerhouse yard. In addition, trench plugs are provided at the berm location in all trenches that connect between the cooling tower yard and the powerhouse yard. The flood routing was performed with the HEC-1 computer program using storage and discharge data which are based on the cooling tower yard topography. Storage in the cooling tower yard is determined by characterizing the yard as an inverted pyramid with a top area of 32.4 acres (ac) and a corresponding apex depth of 1.50 ft below elevation 620.00 ft msl. When ponding in the cooling tower yard reaches elevation 620.35 ft msl, sheet outflow over the southwest side of the yard is routed using a standard weir equation. The natural topography below the overflow area conveys the discharge away from the site.

In 1984, the NRC requested that CNS perform PMP calculations using HMR51/52 (SOF 1.2.1-06). A 6-hr duration PMP producing 30.1 inches (non-adjusted period total) (SOF 1.2.1-07) and a peak 1-hr rainfall of 19.0 inches was used to analyze potential inundation in the Yard areas using the same hydraulic model incorporating the HMR33 PMP. The resulting maximum inundation levels in the Powerhouse Yard using the HMR51/52 PMP are shown in Table 1.2.1-3.

TABLE 1.2.1-3
MAXIMUM POWERHOUSE YARD INUNDATION LEVELS COMPARISON

Analysis Basis	PMP Input to Model HMR 33 (with 0.80 Reduction Factor)	PMP Input to Model HMR 51/52
Previous Maximum Yard Inundation Level CNC 1114.00-00-0040 Rev 22 (SOF 1.2.1-03)	593.94 ft msl	594.74 ft msl
CNC 1114.00-00-0040 Rev 28-29 using 2010 Survey and catch basin network Unit 1 Yard Inundation Level (SOF 1.2.1-04)	594.0 ft msl	594.9 ft msl
CNC 1114.00-00-0040 Rev 28-29 using 2010 Survey and catch basin network Unit 2 Yard Inundation Level (SOF 1.2.1-05)	594.8 ft msl	595.9 ft msl

Note: The elevations shown in Table 1.2.1-1 include the effectiveness of the yard drainage system as documented in the UFSAR Section 2.4.2.3.3.

As a difference in assumption for the application of the PMP using HMR51/52, no reduction in PMP values was applied to account for the imperfect fit of isohyetal patterns to the basin shape (SOF 1.2.1-08). Rainfall percentages given in HMR No. 52 for the 5-, 15-, 30-, and 60-minute intervals were used to estimate rainfall for each 5-minute interval during the peak one hour with the resulting rainfall distributed according to the USACE procedure as shown in Table 1.2.1-4.

TABLE 1.2.1-4
5-MINUTE TIME SERIES FOR PEAK 1-HOUR RAINFALL FOR HMR51/52

Time (minutes)	Incremental PMP (inches)	Cumulative PMP (inches)
5	0.35	0.35
10	0.80	1.15
15	1.05	2.20
20	1.16	3.36
25	1.61	4.97
30	2.02	6.99
35	6.18	13.77
40	1.49	14.66
45	1.60	16.26
50	1.14	17.40
55	0.95	18.35
60	0.65	19.0

Inflow hydrographs for the powerhouse yard, switchyard, and cooling tower yard were developed in the same manner as the HMR33 calculations using the SCS method and routed using the Modified Puls method. Berms around the switchyard and cooling tower yard are high enough to prevent water from flowing from these yards onto the powerhouse yard (SOF 1.2.1-09). Calculations (CNC-1114.00-00-0040 Rev 29) (Reference 8) based on the 1-hour PMP distribution in Table 1.2.1-4 indicate that water will pond onsite to a maximum elevation of 594.90 ft msl for Unit 1 and 595.90 ft msl for Unit 2 (SOF 1.2.1-10). These values are 0.16 ft and 1.16 ft higher than values shown in the UFSAR for the HMR51/52 PMP.

Section 5 of the 2.3 Fukushima NTTF Flood Walkdown report notes for the HMR51/52 PMP crediting 80 Type I catch basins, associated piping network, and sheet outflow areas the resultant pending water level is 594.59 ft msl (elevation determined prior to CNC-1114.00-00-0040 Rev 22), which is higher than the entrance elevation to safety-related structures. The higher yard inundation elevation was evaluated by Duke Energy. Ponding of water remains above 594.0 ft msl elevation for approximately 35 minutes. Water was routed through doors into the Auxiliary Building, Auxiliary Service Building, Exterior Doghouses, former UHI Buildings, and Turbine Buildings. Water entering the Turbine Buildings, would eventually be routed down to the Turbine and Service building basement (568.0 ft msl) which is separated from any structures,

systems, and components important to safety by a 12-ft-high concrete flood wall. In other buildings, the water would spread across the floor areas and be intercepted by the floor drain system (WL system). The floor drain system routes the volume of water to four floor drain sumps and a floor drain tank, all located at elevation 543.00 ft msl in the Auxiliary Building. The evaluation concluded that no safety-related equipment is affected by this water inflow (SOF 1.2.1-11).

The flood levels presented in the 2.3 Fukushima NTTF Flood Walkdown report evaluating the site's flood protection features and procedures were 594.90 ft msl for Unit 1and 595.90 ft for Unit 2 (Reference 6). The current revision to the Yard Flooding Analysis is based on an aerial topographic survey performed in March 2010. This survey allowed a more accurate model of the storage volumes and location and elevation of outflow weirs. The analysis showed that water will overtop the switchyard curb. The calculation concluded that it was appropriate to include the Switchyard drainage area in the Powerhouse drainage area. The west Switchyard outflow area is neglected in the current analysis (SOF 1.2.1-12).

Current design basis recognizes that external flood water will flow through gaps under exterior doors. This flow is evaluated in calculation CNC-1206.03-00-0142 (Flooding of Safety-Related Structures Due to Excessive Rainfall) (SOF 1.2.1-13).

1.2.2 Flooding in Reservoirs

1.2.2.1 Catawba River Reservoirs

The main hydrologic/hydraulic features influencing the CNS plant site are the Catawba River and the series of seven reservoirs that regulate the river upstream and along the shores where the CNS site is located (Lake Wylie). The headwaters of the Catawba are at the Blue Ridge Divide (Eastern Continental Divide) near Old Fort, North Carolina. The river flows generally east and then south where it joins the Wateree River at Lake Wateree near Camden, South Carolina. The Catawba River is approximately 240 miles (mi) long and has a drainage area of approximately 4,750 square miles (sq mi) above Wateree Dam.

Lake Wylie, originally created in 1904 with the construction of a small low head dam on the Catawba River for hydroelectric power production, was rebuilt to its current size in 1925 with the construction of Wylie Dam at the same site near Rock Hill, South Carolina. The lake extends north from Wylie Dam up the Catawba River 28 mi to Mountain Island Dam. The impoundment also extends approximately 5 miles up the South Fork, a western tributary that runs parallel to the Catawba River. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) There are 11 hydroelectric reservoirs and 13 Federal Energy Regulatory Commission (FERC)-regulated hydroelectric powerhouses along the Catawba River.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) regulate flows on the Catawba River upstream from CNS and Lake Wylie. They have a combined usable storage of approximately (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) reservoirs are regulated by the FERC and are operated by Duke Energy under FERC License No. 2232. The reservoirs are managed to maintain seasonal reservoir levels using available hydro turbines and spillway structures, and all dam structures have been remediated to meet current FERC engineering guidelines for stability and discharge of the probable maximum flood (PMF).

The maximum flow recorded for the Catawba River at United States Geological Survey (USGS) gage number 1460 near Rock Hill, South Carolina (just downstream of the Wylie Dam) is 151,000 cubic feet per second (cfs) on May 23, 1901. The period of record for this gage is 1895 to 1903 and 1942 to the present. Two major floods not recorded by the USGS Rock Hill gage are the flood of July 1916, with an estimated flow at Wylie Dam of 299,400 cfs, and the flood of August 1940, with an estimated flow of 169,160 cfs. The July 1916 flood is considered the flood of record for the Catawba River upstream of Lake Wylie. Note that USGS gage records for the 1942 to present period are highly regulated through the (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) hydropower reservoirs upstream of the gage.

Original Project studies were conducted prior to the release of HMR51/52 using a regional hydrologic study to evaluate effects on reservoirs and spillways of PMP occurring over the entire

Catawba River basin. The following paragraphs summarize the hydrologic and hydraulic methodology employed to determine the PMF. Additional details including figures are found in Section 2.4 of the CNS UFSAR.

The greatest storm recorded over the Wylie drainage area occurred from July 13 to 17, 1916. However, greater amounts of precipitation occurred regionally in Elba, Alabama; and Bonitoy and Yankeetown, Florida. It is of note that the later storms all occurred immediately along the coastal area and are expected to produce diminishing amounts of precipitation by transposing these storms inland some 200 mi to the Wylie watershed. To arrive at the PMP over the Catawba River basin, the July 13 to 17, 1916, storm was selected based upon meteorological and physiographic considerations as a guide to the determination of time and area rainfall distribution pattern. The following adjustments were made to this storm to increase its magnitude and intensity to such values considered to be equal to the PMP over the Catawba River basin.

1. Rainfall depth-duration values were distributed in accordance with that of the 1916 storm.
2. Storm position was transposed over a limited distance within the Catawba River basin to produce a maximum concentration of precipitation over a selected area.
3. Precipitation amounts are increased 40 percent.

In the area of the Wylie watershed, snow melt is not a consideration because of its southern location. Hourly incremental rainfall and rainfall excess for the 54-hour period of the PMP are available for inspection in the Regulatory Compliance Licensing Files as former FSAR Figures 2-35 and 2-36.

The topography of the Catawba River basin is gentle to moderate, sloping toward the river in a southeasterly direction. The soil designated according to the National Cooperative Soil Survey Classification of 1967 is Ultisoil U5-3. Initial loss for conditions, usually preceding major floods in humid regions, normally range from about 0.2 to 0.5 inch and is relatively small in comparison with the flood runoff volume. A value of 0.5 inch was used for initial loss in the study. Infiltration rates vary throughout the storm period from a high rate at the beginning to a relatively low and uniform rate as the precipitation continues. Model infiltration rates were

estimated based on comparison of regional studies which were judged to be comparative to the Catawba River. The topography, soil groups, and climate of the regional basins were judged to be very similar. For the design basis study, an infiltration rate of 0.10 inch per hour was selected (SOF 1.2.2.1-03).

To obtain time dependent inflow to the Catawba River from the PMP, each reservoir's drainage area was divided into subareas depending on the number of larger tributary streams flowing into each reservoir. Synthetic unit hydrographs were used. Synthetic unit hydrograph coefficients were derived from the historic storm of September 29 to 30, 1958 (Hurricane Gracie) for nine tributary streams from which gaging records were available. The lag time was reduced so that it more closely represented assumed conditions during a large flood runoff. The subdivision of any reservoir into principle subareas was necessary for the purpose of reflecting more accurately the non-uniformity and varying-intensity of rainfall over large reservoir drainage areas. Hourly precipitation amounts were distributed to existing precipitation stations by the Thiessen polygon method. The area of each polygon falling within a subarea was expressed as a percentage of the total subarea. These percentages were the rainfall-subarea coefficients assigned to each precipitation station rainfall.

The steps used to synthesize flood flow into reservoirs are summarized below:

1. The applicable portion of each rainfall station's precipitation was converted directly into inflow for that portion of the polygon covered by reservoir water surface.
2. The runoff (rainfall less losses) from each rainfall station was applied to the percentage which the precipitation station Thiessen polygon bounded. These values for each precipitation station are summed for each subarea, resulting in the average hourly rainfall excess for the subareas. The average hourly rainfall excess in inches is then applied to each subarea unit hydrograph, resulting in a storm hydrograph of local inflow for each subarea for each hour of runoff.
3. The total inflow to each reservoir consists of local inflow from each subarea of the reservoir local drainage area, plus local inflow due to reservoir surface rainfall, plus upstream flow, plus base flow.

The flood resulting from the PMP is routed through the Catawba River system to Wylie Dam by means of a flood routing program developed by C. T. Main (November 1968) (Reference 42). Local reservoir inflow for each hour of the storm and for each reservoir was calculated. Reservoir elevations were computed from reservoir inflows and with the discharges corresponding to these elevations. The method of computing reservoir elevations and discharges used successive approximations to satisfy the relation that inflow is equal to outflow plus storage.

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

The discharge from the first reservoir allowing for lag time, where applicable, was added to the local inflow for the next reservoir, and the routing procedure was repeated for all reservoirs for each hour of the storm plus any additional hours needed to cover the complete runoff.

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

These initial reservoir levels were based on historical water level records for late summer and early fall. Discharges were then limited to gate capacities at given elevations and the reservoirs begin to rise if inflow continues to exceed gate capabilities. When reservoir elevations start to fall, which occurs when inflows become less than outflows, discharges are continued at the capacity of the gates until such time as the reservoirs reach their predetermined control elevations. Subsequently, the reservoir levels are held constant for the remaining period of the flood; outflow becomes equal to inflow during this period. Discharges from generation of power were assumed to continue throughout all gate operation procedures unless reservoir levels overtop bulkheads protecting powerhouses or switchyards, at which time the discharge through the powerhouse was stopped for the remainder of the storm period. The effect of not having any power generation releases was also calculated.

Operation of all floodgates is performed at the direction of personnel from the Duke Energy (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The decision to operate the floodgates is based on water elevations, rainfall, power plant operations, and weather forecasts over the entire eleven-reservoir, Duke Energy-operated Catawba River system. The spillway gates are opened as necessary to maintain the reservoirs at or close to the normal operating level. The reservoir levels are monitored

hourly. The operation of the Catawba River system modeled in the C.T. Main computer program is consistent with Duke Energy hydro operations practice (Reference 42).

(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)

Using the methodology summarized in this section, routing the PMF through the Catawba River system results in a maximum inflow of cfs with a maximum discharge of cfs. Several trial positions of the Storm Center were made to determine the most critical position for producing the maximum flood over Lake Wylie. The storm center was positioned over each of the reservoir drainage areas in turn and then routed through the Catawba River system into Lake Wylie. This methodology is standard practice to maximize the PMP rainfall isohyetals over the drainage to produce the most runoff impacts. The location which produced the highest reservoir elevation at Wylie Dam of ft msl results from (Table 1.2.2-1.

(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)

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

TABLE 1.2.2-1
LAKE WYLIE PROBABLE MAXIMUM FLOOD FLOODING RESULTS

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

The maximum reservoir elevation calculated for Lake Wylie assumes that all gates are available for operation. All of the flood gates at Wylie Dam are inspected and physically operated yearly in accordance with FERC regulation. In addition, each gate is fully opened every 5 years. These gates are operated by

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

Based on the above discussion,

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

1.2.2.2 Standby Nuclear Service Water Pond

The Standby Nuclear Service Water Pond (SNSWP) is a nuclear safety-related impoundment constructed by placing a dam across a small tributary of Lake Wylie to the north of the CNS Yard. Table 1.2.2.2-1 (SOF 1.2.2.2-01) provides pertinent information about the pond and dam.

TABLE 1.2.2.2-1
SNSWP DESIGN INFORMATION

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

The SNSWP was analyzed for a PMP centered critically over the SNSWP drainage basin using the procedure outlined in the Bureau of Reclamation publication titled, Design of Small Dams (Reference 56). Due to the small drainage area, the PMP (30.1 inches) (SOF 1.2.2.2-02) for a

10-sq-mi area and a 6-hr duration was used. The 6-hr PMP was divided into an hourly temporal sequence which produced the greatest PMF for the basin. The incremental runoff values are presented in Table 1.2.2.2-2 (SOF 1.2.2.2-03).

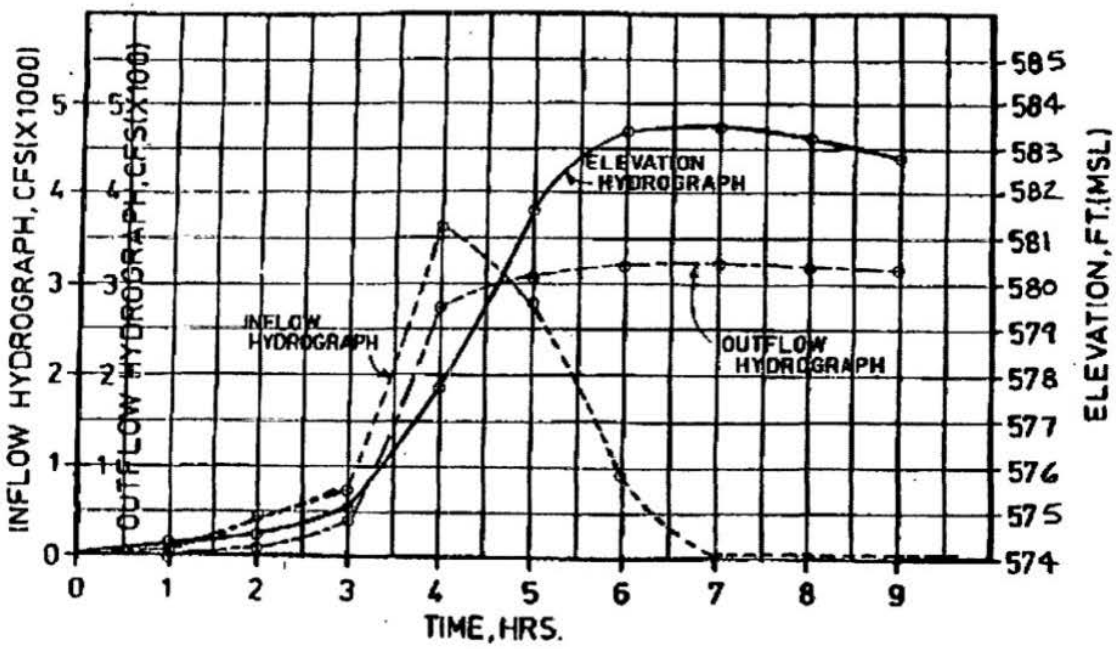
TABLE 1.2.2.2-2
SNSWP DRAINAGE BASIN HOURLY INCREMENTAL RUNOFF

Hour	Runoff (inches)	Inflows (cfs)
1	2.4	70
2	2.7	450
3	3.3	750
4	14.9	3650
5	4.6	2800
6	2.5	900

Determination of the runoff for a period of longer duration was not considered due to the size of the basin. The time of concentration, 40 minutes (SOF 1.2.2.2-04), was calculated using the hydraulic length and slope of the basin and formula $T_c = ((11.9*L^3)/H)^{0.38}$.

Figure 1.2.2.2-1 gives the PMF inflow, outflow, and water surface elevation hydrographs for the SNSWP (SOF 1.2.2.2-05).

FIGURE 1.2.2.2-1
SNSWP PMF HYDROGRAPHS



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

The wave run-up was calculated using a maximum wind velocity of 40 miles per hour (mph) and a fetch length of 0.42 mi. The maximum height of the wave on the slope including the run-up is 1.0 ft.

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

The SNSWP dam is protected from wave action by riprap.

1.2.3 Dam Failures

(b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Other small farm ponds and dams are scattered throughout the drainage basin but none were reported to have significant storage to be considered in an evaluation of impacts due to dam failures. This section addresses the consideration of a dam failure at one of the (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) All the dams above Wylie are composed of part earth embankment and part concrete gravity-type structures. Table 1.2.3-1 provides a list of the upstream dams and drainage areas in sq-mi for each reservoir (SOF 1.2.3-01).

TABLE 1.2.3-1
DUKE ENERGY FERC LICENSED DEVELOPMENTS ON THE CATAWBA RIVER UPSTREAM OF CNS

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

Flood design considerations for the licensing basis at the CNS site included two dam failure scenarios.

1. A 25-year frequency flood passing through Lake Wylie combined with a seismic failure of an upstream dam.

2. A Standard Project Flood (SPF) passing through Lake Wylie combined with the failure of an upstream dam due to an Operating Basis Earthquake (OBE). The SPF is considered equal to one-half of the PMF.

The seismic failure for each upstream dam was timed to coincide with the Standard Project Storm centered over its drainage area. At the hour which the reservoir reaches its maximum level, it was assumed that seismic failure of the dam occurs. The flood routing was computed for hourly intervals by means of a flood routing program with the procedure described in UFSAR Section 2.4.3. The results of the test scenarios showing maximum reservoir elevations at each upstream reservoir are shown in Table 1.2.3-2 (SOF 1.2.3-02).

TABLE 1.2.3-2
COMBINED EFFECTS SPF PLUS DAM FAILURE SIMULATION SCENARIO RESULTS

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

The bounding dam failure scenario for the CNS site was concluded to result from the

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

This combined effects failure event resulted in a

maximum static water elevation of (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft msl which is (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft lower than the Yard elevation of 593.50 ft msl.

TABLE 1.2.3-3

COMBINED EFFECTS SPF PLUS (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) SIMULATION SCENARIO RESULTS

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

Assuming a (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam, the normal source of cooling water for CNS is lost between elevation 569.40 and 550.00 ft msl. Elevation 550.00 ft msl is the approximate top of rock for that section of the Wylie Dam (SOF 1.2.3-04). If this occurs, the plant would be aligned to the SNSWP as a cooling water source. The SNSWP dam was designed to withstand rapid drawdown on the Lake Wylie slope of the dam.

Local flooding effects from a PMP centered over the SNSWP were considered in the licensing basis. However, the south rim of the SNSWP is higher than the top of the SNSWP dam (crest elevation [redacted] ft msl) and a failure of the dam would result in flood-breach discharge entering Lake Wylie in lieu of the CNS Yard.

(b)(3) 16 U.S.C. § 824o-1(d), (b)

1.2.4 Storm Surge and Seiche

Storm surge and seiche events were postulated to affect the site using the Probable Maximum Hurricane winds, as defined by the Weather Bureau report HUR 7-97. The description of analyses performed to address surge and seiche impacts on Yard flooding is substantially

repeated from the CNS UFSAR Section 2.4.5. Surge and seiche water levels generated by various wind mechanisms (as described in Weather Bureau Report HUR 7-97 and "Shore Protection Manual", Department of the Army Corps of Engineers, 1973) are described in Table 1.2.4-1 (Reference 52.)

Two hurricane tracks were considered possible occurrences and were included in studies performed by Duke Energy to maximize the surge and wave effects at the plant site. Case A assumes a hurricane track wherein the storm center moves shoreward in a northwesterly direction, crosses the coastline near Savannah, Georgia, and then gradually re-curves toward the northeast to pass to the east of Lake Wylie. This track was hypothesized to place the radius of maximum wind coincident with the effective fetch at the plant site. Case B is patterned after the track of Hurricane "Ginger" of the 1971 season. This storm follows a generally northwesterly course from the Atlantic Ocean, and goes ashore on the North Carolina coast between Wilmington and Cape Hatteras. It then follows a westerly course to a point between Charlotte and Raleigh where it curves northeast into Virginia. This track is also modified to put the radius of maximum wind coincident with the effective fetch at the plant. The maximum wind speeds calculated were 101.5 mph for Hurricane A and 116.0 mph for Hurricane B. (SOF 1.2.4-01.)

TABLE 1.2.4-1
SURGE AND SEICHE WAVE CALCULATION RESULTS

Assumed Hurricane Path	Wind Velocity (mph)	Wind Tide (ft)	Wave Height (ft)	Run-up (ft)	Total Height[1] (ft)
A	101.50	0.33	7.40	4.00	6.60[a]
		0.33	12.40	5.50	8.10[b]
B	116.00	0.44	8.60	5.10	7.70[a]
		0.44	14.40	5.80	8.40[b]

Note: Data above from UFSAR Section 2.4.5.1
[1] Total Heights for Hurricanes A and B include the component due to differential pressures.
[a] = significant wave: the average height of the one third highest waves of a given wave group.
[b] = one percent wave: the average of the highest one percent of all waves in the group 1.67 times the height of the significant wave.

It should be noted that the hypothetical 1-in-100-year flood could be passed by the Wylie Dam with no significant increase in water surface elevation (SOF 1.2.4-02). Therefore, combining surge or seiche wave height with 100-year flood elevations is not the bounding site flood causing event (combined effect flood elevation 592.2 - 569.4 = 22.8 ft >> 8.4 ft).

1.2.5 Tsunami

Tsunamis were never postulated to affect the site, and no flood elevation is given in the original licensing/design basis case basis of the plant. CNS is located inland (more than 150 miles from the Atlantic coast) (SOF 1.2.5-01) and not on a waterway that would be subject to effects of a Tsunami.

1.2.6 Ice-Induced Flooding

Ice-induced flooding was never postulated to affect the site, and no flood elevation is given in the original licensing/design basis case basis of the plant. The climate in the Catawba River basin is moderate (minimum monthly mean water temperature for Lake Wylie is in the low 40's) (SOF 1.2.6-01) and there has not been any recorded ice formation on reservoirs in the river system.

1.2.7 Channel Diversion

Channel diversions were never postulated to affect the site and no flood elevation is given in the original licensing/design basis case basis of the plant. The Catawba River is highly regulated by a series of dams. Reservoirs are back-to-back and backwater effects of each dam mitigate reservoir velocities that would be necessary to produce channel diversion. In the event of the loss of Lake Wylie, the Catawba Nuclear Station could be safely shut down using the SNSWP. The SNSWP was constructed in a small tributary to the main channel of Lake Wylie and is protected from scour by topographic features and rip rap placement on dam slopes.

1.2.8 Combined Effects

Combined flooding effects (PMP, PMF, dam failure and/or wind-driven waves) were reviewed for impacts at the CNS site. Maximum water level elevation at the station occurs with the simultaneous failure of [redacted] (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Several storm locations were used to find the highest flood level. The maximum water surface elevations when combined with a 40-mph wind are shown in Table 1.2.8-1 (SOF 1.2.8-01).

Procedures used in the licensing basis evaluation for calculating wind set-up and waves are described in the USACE Shore Protection Manual (1973) (Reference 52). The calculation was performed assuming deep water waves, a lake surface elevation of [redacted] ft msl, and a 40-mph overland wind (SOF 1.2.8-02). The run-up was calculated for three locations, the plant yard at the end of the intake canal, the discharge structure, and the SNSWP dam. The results presented in Table 1.2.8-1 assume a static water elevation of [redacted] ft msl, corresponding to the SPF and failure of [redacted] (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam, indicate there would be minor wave run-up of [redacted] ft at the north end plant yard, which is below openings for Class I structures (SOF 1.2.8-03).

(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)

TABLE 1.2.8-1
COMBINED EFFECTS MAXIMUM RESERVOIR ELEVATION WITH WIND-DRIVEN WAVES

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

1.3 Licensing Basis Flood-Related and Flood Protection Changes

CNS staff conducted a station flood walkdown in conjunction with 10CFR50.54 (f) (Reference 6.) in late 2012 as documented in CNC-1114.00-00-0065. (Reference 9.) A total of 18 flood protection features were evaluated per the guidelines in Nuclear Energy Institute (NEI) 12-07.

The report noted the following licensing basis flood-related and flood protection changes from the UFSAR as described below.

1. The modification (EC 108015) to increase the height of the Unit 1 ZD vents at the Diesel Generator Building roof (Elevation [redacted] ft msl) was completed after the flood feature walkdown. The modified height of the ZD vents opens at Elevation 598.00 ft msl (1A) and 598.04 ft msl (1B). The openings are at least [redacted] ft above the flood level of [redacted] ft msl.

(b)(5) 16 U.S.C. § 824o-1(d), (b)

(b)(3) 16 U.S.C. § 824o-1(d), (b)

(b)(3) 16 U.S.C. § 824o-1(d), (b)

2. Flood barriers for the Unit 1 and Unit 2 - [redacted] ft msl Electrical Penetration Room doors have been identified and placed in the Corrective Action Program (CAP). Other exterior designated doors with low margin are also identified in CAP and are being evaluated.

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

3. The site topographic survey is an input into the CNS PMP analysis to determine site flood levels. Any major changes to the topography are controlled by the modification process. The last site topography survey was in March 2010, and there have been no major activities implemented since the survey was performed.

4. A potential deficiency with the Unit 2 Auxiliary Building [redacted] ft msl Electrical Penetration Room door was identified. Due to the recently revised ponding levels described in the design basis section of this report, the water flow rate through the gap under the door will increase. The water entering through the gap under this door routes to the Unit 2 CA Pump Room. A review of the calculation for flooding of safety-related structures due to external flooding identified that this may result in the water level in the CA Pump Room rising and then overflowing into the Turbine Driven CA Pump pit. Preliminary results show that the water level in the Turbine Driven CA Pump pit may rise to a level that would be above the bottom of the turbine driven pump shaft.

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

5. As a conservative measure, an interim action has been incorporated to reduce the water input to the CA Pump Room from the Waste Solidification Building sump in the event of an external flooding event. The Waste Solidification Building Sump Pump discharge valve is normally closed. Operation of this sump pump is directed per the annunciator response procedure for a high level in this sump. This procedure has been revised to include a step to contact Operations to ensure an external flooding event is not occurring or expected prior to placing the Waste Solidification Building sump pump in service. An additional interim action has been incorporated into AP/0/A/5500/030 (Plant Flooding) to provide a barrier to reduce the water flow under the Unit 2 Auxiliary Building [redacted] ft msl Electrical Penetration Room door. These actions will prevent the water level in the

(b)(3) 16 U.S.C. § 824o-1(d), (b)

Turbine Driven CA Pump pit from reaching a level which would threaten the Turbine Driven CA Pump.

6. An ECR (engineering change request) has been created to install a flood gate (barrier) to block the Unit 2 Electrical Penetration Room door during an external flooding event.
7. During the procedure walkthrough for AP/0/A/5500/030 and RP/0/A/5000/007, it was observed that for the two Auxiliary Service Building doors (AR5 and ARG) required to be closed or have the opening blocked, there is no guidance on how to manually close the doors. Also, no equipment or supplies are staged to block the door opening if the door cannot be closed. An action was entered into the CAP to add steps to the procedure for closing the doors manually. Another action was entered in CAP to have an appropriate temporary barrier available to block the door openings, if the doors cannot be closed.

1.4 Watershed and Local Area Changes

Changes to the local site topography and support buildings have taken place since original construction. Changes in local area conditions have been captured in the modeling performed to support the flooding assessment due to PMP and dam failure inundation using recent aerial and ground survey data (2010) along with updated drainage, utility trench location, and building geometry. Calculation CNC-0114.00-00-0040 Rev 29 (Reference 8) documents local area changes and the impact on PMP inundation modeling. Changes to local area runoff/drainage conditions can generally be grouped by modifications within the secure area and outside of the secured area. Figure 1.4-1 (Reference #26) identifies areas where site changes have been made that could impact site drainage. The changes within the secure area include:

- The four domed storage tanks, two of which are to the south of Reactor No. 1 and two of which are to the north of Reactor No. 2.
- All surface types (asphalt, concrete, and crushed stone).
- Installed (or being installed) security shield/walls attached along the top of the buildings allow water from the roofs to drain between the security wall and face of the building.
- Diesel generator vents were raised following the completion of the 2011 site flood analysis.

- Security barrier and fencing:

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

- Doublewide trailers secured with hurricane strap tie downs (e.g., Craft Offices Buildings 7757, 7768, and 7769).
- The concrete pad containing portable storage container boxes and the Paint Mix Storage Building (Building 7722) east of the Craft Offices.
- Building 7767.
- The permanent arched tent structure to the west of Building 7767 with both the east and west ends open to the Yard.
- Building 7772 (Scaffold Storage Building with no siding on the lower 25 inches of the walls).
- A potential for flow obstruction during the LIP may form during debris buildup on grating placed over the cable trench located at the southeast corner of Turbine Building No. 1 and the Standby Shutdown Facility.

Exterior Secure Area

- An 8-inch curb in Transformer Yard.

- There are two sections of paved parking lots located generally west of the Transformer Yard. The eastern lot generally drains to the east/southeast (toward the Powerhouse Yard) while the western parking lot generally drains to the southwest (away from the plant site).
- Construction Yard plateau to the west/northwest of the vendor parking lot. Repairs included stone and riprap placement in the gully and maintenance work of the Construction Yard plateau edge to form an eastern-edge berm and adjoining drainage swale with spaced stone berms within the swale and perpendicular to the plateau berm for energy dissipation.
- Fukushima Flex Construction:
  - Proposed diesel pump ramp modifications to the land adjoining the SNSWP.
  - Proposed building and laydown area located on the Cooling Tower Plateau and located north of the cooling towers does not appear to pose any flow path disruption to the Powerhouse Yard.
- The chlorine pre-treatment building (Building 7773a) adjoining the Low Pressure Service Water (LPSW) Intake Structure (Building 7773) and the temporary trailer adjoining the intake. Both buildings show visible evidence of flow passage through/under the buildings.
- The Cooling Tower Yard berm running along the western edge of the plateau.
- Additional roof structure is being added to Building 7714 (Water Chemistry Building) that adjoins the Allison Creek-CNS Discharge.
- New Building 7752.
- Buildings 7714 and 7752.

FIGURE 1.4-1
AERIAL PHOTOGRAPH OF CNS SITE IDENTIFYING AREAS OF SITE SECURITY CHANGES OBSERVED DURING SITE WALKDOWN FOR THE FUKUSHIMA 2.1 FLOOD HAZARD REVIEW (REFERENCE 26)

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

Changes in the approximately 3,020-sq-mi drainage basin have occurred since initial licensing basis analyses were performed. The largest change would be in the Charlotte, North Carolina,

area and in the areas immediately surrounding the reservoirs where populations have increased. The impacts of these changes while looking at the entire 3,020-sq-mi drainage basin are considered small percentages of the total drainage area. Section 2 hydrology summaries are based on studies performed in the 1990's and consider changes in the watershed since the initial licensing basis hydrology was performed in the 1970's.

1.5 Current Licensing Basis Flood Protection and Mitigation Features

Based on the CNS licensing/design basis case, a list of flood protection, mitigation, and early warning indicator features has been compiled in specification CNS 1465.00-00-0011. Appendix A of 10CFR50, General Design Criterion (GDC) 2 requires that nuclear power plant structures, systems, and components important to safety be designed to withstand the effects of natural phenomena such as flood, tsunami, and seiche without loss of capability to perform their safety function. In accordance with Regulatory Guide 1.59, Design Basis Floods for Nuclear Power Plants, Rev. 2, 8/77 (Reference 61), Nuclear Power Plants are required to be designed to prevent the loss of capability for cold shutdown and maintenance thereof resulting from the most severe flood conditions that can reasonably be predicted to occur at the site as a result of severe hydro-meteorological conditions, seismic activity, or both.

CNS has developed a list of features created to fulfill the NRC-issued information request on March 12, 2012, in accordance with 10 CFR 50.54(f). Enclosure 4 of the 50.54(f) letter was directed toward addressing the NTTF Recommendation 2.3 for Flooding and requested the results of a flooding design basis walkdown. Below is a description of the different features presented in the 2.3 Flooding Walkdown report (CNC-1114.00-00-0065 Rev 1).

The following are protection features that are credited in the licensing basis to protect safety-related systems, structures, and components against external sources of flooding.

Site Flood Protection and Mitigation Features:

1. Yard drainage and catch basins

   Surface water drainage is accomplished through the yard drainage catch basin inlets and piping networks. Type I catch basins do not have a slotted cover, but are protected by steel grating on four sides and top to prevent blockage. The total effective opening in the grating on any one side is at least equal to the effective opening of the pipe inlet. There are 89 Type I catch basins provided in the power block area, and only 80 are required to be operable.

2. Sheet outflow areas

   The Powerhouse yard has two sheet outflow areas, one on the south side of the Powerhouse yard and one on the North east side of the powerhouse yard near the LPSW intake area. Additionally, a sheet outflow area is provided on the west side of the Switchyard.

3. Cooling tower protective berm

   The Cooling Tower Yard is at a higher elevation than the Powerhouse Yard. An earth berm on the north and west perimeter with minimum top elevation 620.50 ft msl is provided to divert water away from the Powerhouse Yard area.

4. Switchyard protective concrete and asphalt curbs

   The Switchyard is at a higher elevation than the Powerhouse Yard. A concrete and asphalt curb on the north, south, and east perimeter with minimum top elevation 632.67 ft msl diverts water from flowing onto the Powerhouse Yard area.

5. Flood barriers in cable trenches between cooling tower and switchyard

   Flood Barriers are installed in below-grade trenches at berm and curb locations. The flood barriers stop water flow between the Cooling Tower Yard and Switchyard, and the Powerhouse Yard.

6. Designated pressure rated (3.00 psi) flood barrier doors in the auxiliary building, auxiliary service building, exterior doghouses, UHI buildings, fuel receiving buildings, and diesel generator buildings.

   For the time period during the local intense PMP when the yard inundation level is estimated to be above elevation 594.00 ft msl, designated doors are credited to remain closed to minimize water inflow. The designated doors are:

   - Auxiliary Service Building: AX664D, AX605D, AX617, AX671, AR 5, AR 6, AX666A
   - Auxiliary Building (Electrical Pen Rooms): AX658A, AX656B
   - Exterior Doghouses: AX661, AX660
   - UHI Buildings: AX301A, AX300A
   - Fuel Receiving Building: AX600, AX600B, AX627, AX629, AX629B
   - Diesel Generator Buildings: AX302B, AX304B, AX306B, AX308B

   Entrances for these doors have flood barriers with top elevation 597.00 ft msl (AX302B and AX304B) and 597.13 ft msl (AX306B and AX308B) that are credited to minimize inflow.

   All doors entering safety-related buildings are pressure doors (designed for 3.00 psi). All regularly used exterior doors are equipped with automatic closures, except for equipment access doors, AR6 and AR5. These doors are controlled from inside the Auxiliary Service Building, and cannot be opened from the outside. Station Security, by procedure, ensures that the Auxiliary Building and Auxiliary Service Building exterior doors are closed in the event of severe weather.

7. Designated Below Grade Conduit Seals

   Electrical conduits in Conduit Manholes (CMH) 2, 3, 18A, 18B, and 21 are sealed to prevent water intrusion into safety-related buildings. Electrical conduits which communicate between safety-related structures and miscellaneous yard areas are sealed or enclosed to prevent water intrusion.

8. Trench Cover Seals

   Trench covers in the Refueling Water Storage Tank Pipe trenches and the Monitor Tank Building Pipe trench, which communicate with the Auxiliary Building area, are sealed to prevent water intrusion.

9. Access Hatch Seals in Diesel Generator Building Roofs

   Roof hatches at grade (Elevation 594.00 ft msl) and penetration sleeves in the Diesel Generator building roofs are sealed to prevent water intrusion.

10. Turbine/Service Building Flood Wall

    A 12-ft-high concrete wall (top Elevation 577.50 ft msl) along column line 34 in the Turbine/Service Building retains water entering the Turbine/Service Building preventing water from entering safety-related areas. Estimates of water infiltration depth behind the flood wall yield a maximum elevation of 568.50 ft msl in the Turbine Building basement (Elevation 568.00 ft msl).

11. Groundwater Drainage

    A permanent Category I groundwater drainage system is installed to maintain a normal groundwater level at or near the base of the foundation mat and basement walls, eliminating the uplift and hydrostatic forces on the Auxiliary and Reactor Buildings. The groundwater drainage system consists of foundation underdrains and continuous exterior wall drains. Testing and inspection of the ground drainage system are performed in accordance with SLC 16.7-8.

12. Auxiliary building roof – sealing of penetrations

(b)(3): 16 U.S.C. § 824o-1(d); (b)

Roofs of safety-related structures are designed with no obstructions so that water will flow directly off roofs with minimal accumulation. The Auxiliary Building Roof at Elevation [redacted] ft msl is at ground elevation. PMP flood levels will accumulate on roof. All roof penetrations should be sealed to prevent water intrusion.

Section 2

Flooding Hazard Reevaluation

The reevaluations in Section 2.0 are not part of the CNS licensing basis as documented in the UFSAR. The following sections describe additional reevaluation analysis for assessing appropriate external potential flooding hazard events including the effects from PMP on the site, PMF on reservoirs, and dam failures that have been performed post-licensing basis to meet the Hierarchical Hazard Assessment (HHA) procedure described in NUREG/CR-7046 for assessment of flooding hazard at safety-related SSCs. (Reference 60)

CNS is located on the shoreline of Lake Wylie which is a FERC-regulated hydroelectric development that is impounded by a concrete gravity dam and powerhouse, gated and ogee spillway, and an embankment on the west abutment approximately 4.8 river miles downstream of CNS. As a FERC-regulated dam, the structures have been designed and verified with hydrologic and hydraulic analysis and dam stability analyses in accordance with "FERC Engineering Guidelines for the Evaluation of Hydropower Projects." (Reference 13) These structures are monitored and inspected on regular intervals by Duke Energy, FERC, and independent dam safety consulting engineers.

2.1 Local Intense Precipitation

The flood water elevation due to a maximum PMP was reevaluated using current practice Hydrometeorological Report (HMR) PMP and state-of-the-practice engineering software. The analysis evaluated the maximum water surface elevation within the CNS power block area resulting from the occurrence of the PMP and updated site topography and structure layout. This reevaluation utilizes HMR51 PMP and rainfall distribution patterns in accordance with guidance in NUREG/CR-7046. The modeling software selected for this evaluation generally exceeds guidance outlined in NUREG/CR-7046, 1-D channelized flow using the USACE's Hydrologic Engineering Center's-River Analysis System HEC-RAS software. The selected model is capable of combining both 1-D analysis methods for roof drainage simulations and more appropriate 2-D flow equations for surface flow on relatively flat CNS Yard surfaces

2.1.1 Probable Maximum Precipitation

In accordance with guidelines from Section 3.2 of NRC NUREG/CR-7046, the LIP was estimated for the CNS site using point PMP. Point rainfall (1-mi^2) PMP values for durations of 1 hour and less are determined using the procedures as described in HMR No. 52 (Reference 45). Since the CNS site is less than 1-mi^2, a 1-hour PMP was used to evaluate the effects of local intense precipitation in the immediate vicinity of the site. This methodology is current industry practice.

Generally, for smaller drainage areas like CNS, shorter durations are critical. HMR No. 52 contains guidance to determine PMP estimates for durations less than 6 hours.

PMP charts (HMR No. 52 Figures 24, 36, 37, and 38) were used to determine PMP estimates for durations of 1 hour and less based on the location and size of the drainage basin. The site location was approximated on each HMR52 figure as shown in Figure 2.1.1-1. Using the PMP chart and the site location, the 1-hour, 1-mi^2 PMP estimate was determined to be 18.9 inches per hour (in/hr) as illustrated in Figure 2.1.1-1 (SOF 2.1.1-01).

For areas less than 200 mi^2, ratios were used to determine the 5-, 15-, and 30-min duration PMP estimates. The ratios were found using PMP charts (HMR No. 52 Figures 36, 37, and 38). Using the PMP charts and the site location, the ratios and PMP estimates for durations less than 1 hour were determined as shown in Table 2.1.1-1. The ratios were applied to the 1-hour, 1-mi^2 PMP estimate of 18.9 in/hr (SOF 2.1.1-02).

TABLE 2.1.1-1
PMP RATIOS AND ESTIMATES (INCHES) FOR DURATIONS LESS THAN 1 HOUR

	1-mi^2 Point Rainfall			
	5-min	**15-min**	**30-min**	**1-hr**
PMP (in.)	6.2	9.7	14.0	18.9
Ratio to 1-hr PMP	*0.326*	*0.512*	*0.740*	-

FIGURE 2.1.1-1
TYPICAL EXAMPLE OF HMR NO. 52 FIGURE USED IN THE PMP ANALYSIS

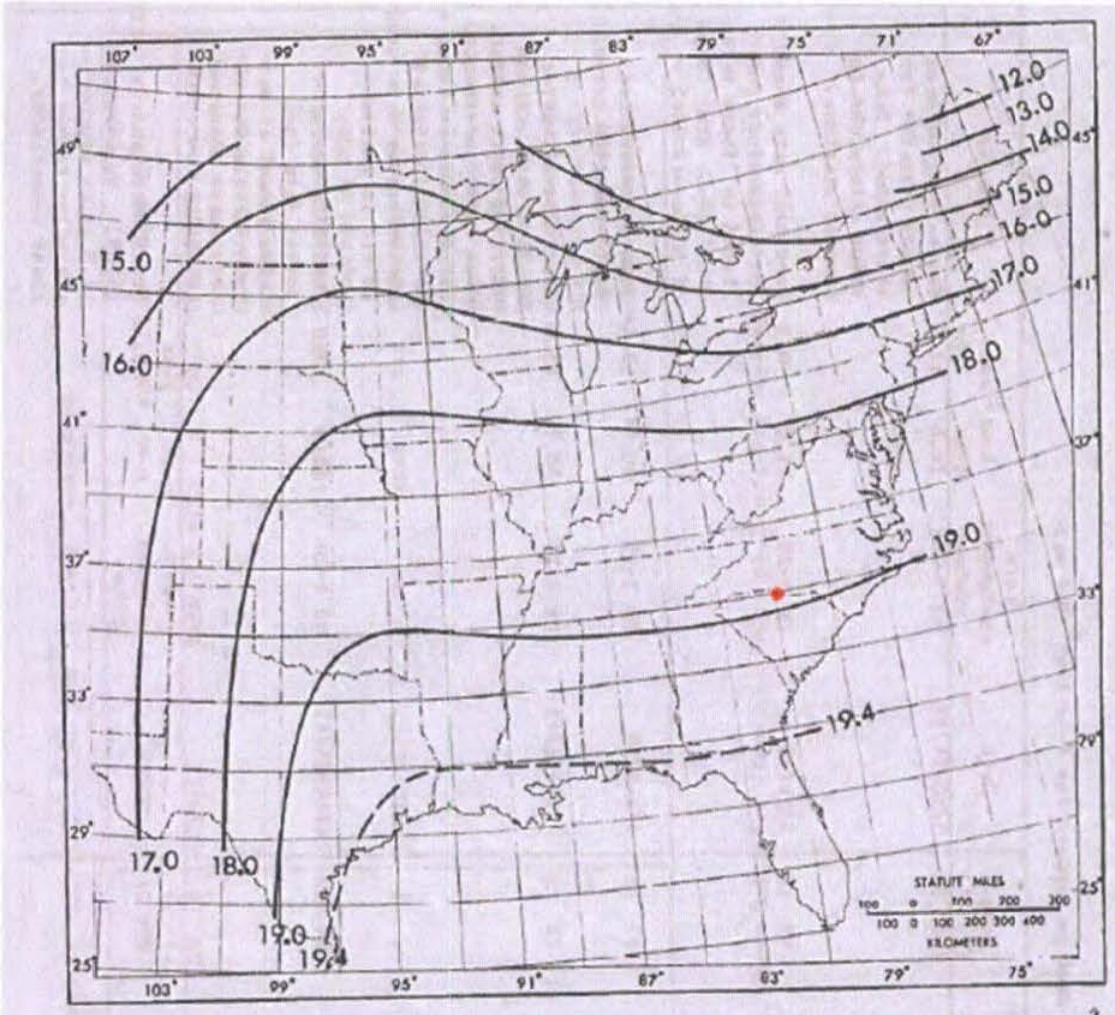


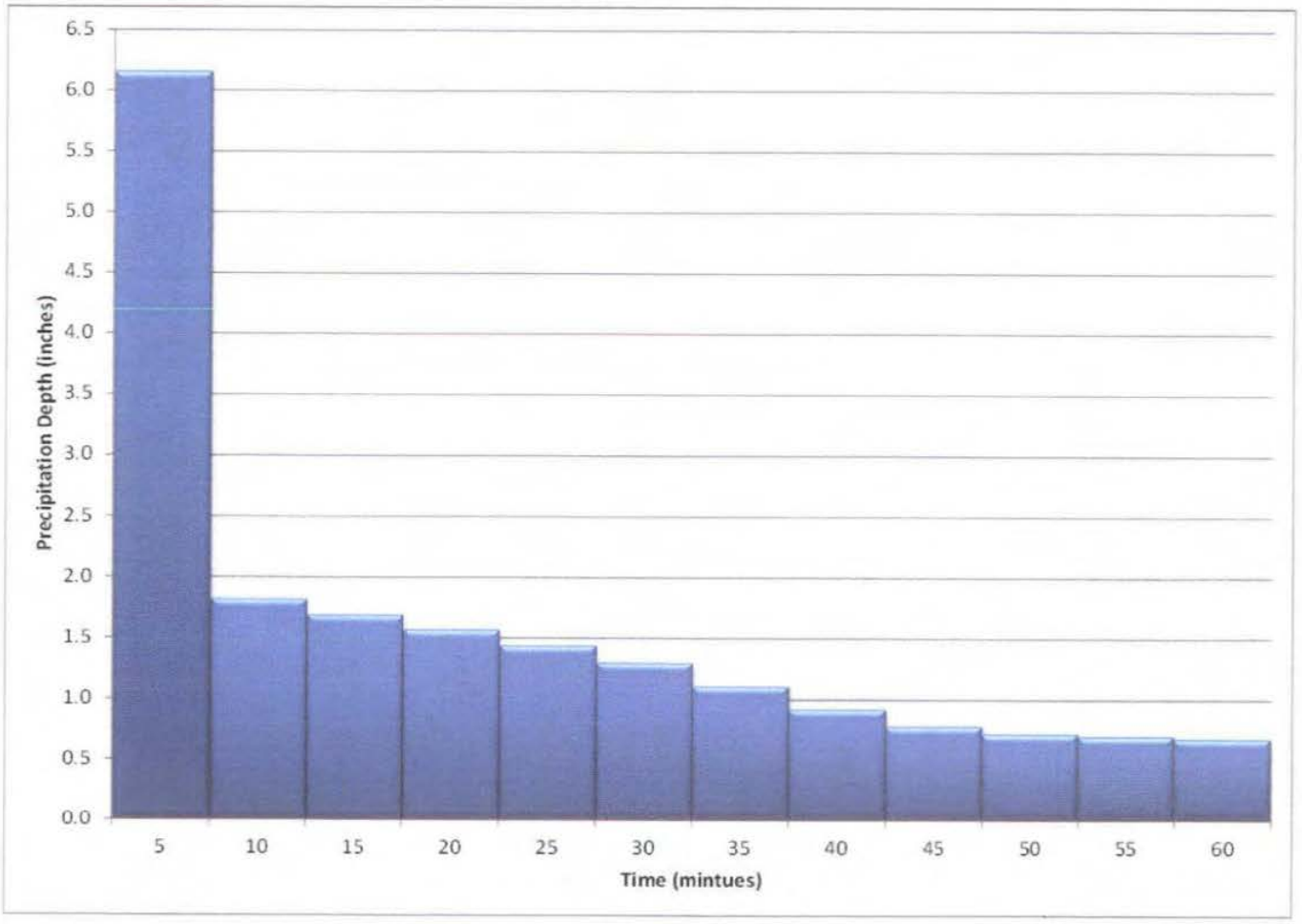
Figure 24.—1-hr 1-mi^2 PMP analysis based on figure 23 and 6-hr 10-mi^2 precipitation from HMR No. 51.

The PMP was evaluated based on a front-end loaded, 1-hour maximum temporal distribution. The PMP duration was chosen based on guidance provided in NRC NUREG/CR-7046, Section 2.4.3 (Reference 60). The front-end loading temporal distribution applies the most intense rainfall at the beginning of the storm and decreases in intensity over time as shown in Table 2.1.1-2 and Figure 2.1.1-2 (SOF 2.1.1-03).

TABLE 2.1.1-2
5-MINUTE INCREMENTAL CNS PMP

Time (minutes)	Incremental Precipitation (inches)
0	0
5	6.16
10	1.82
15	1.69
20	1.57
25	1.44
30	1.30
35	1.11
40	0.92
45	0.78
50	0.72
55	0.70
60	0.68
Total	18.9

FIGURE 2.1.1-2
CNS PMP HYETOGRAPH



2.1.2 Site PMP Model Setup

Innovyze Infoworks ICM (Integrated Catchment Model), Version 3.0 software 2012 (ICM) (Reference 35) was used to evaluate the effects of the point PMP at CNS. ICM is a fully integrated 1-Dimensional (1-D) and 2-Dimensional (2-D) hydrodynamic model which allows for a more appropriate hydraulic simulation (vs. a channelized 1-D approximation model) of the relatively flat topography found on the CNS site. 1-D model simulation options are used to model runoff from building roofs, while 2-D simulation is used to model overland site hydraulics enabling the hydraulics and hydrology to be incorporated into a single model.

Overland flow for the entire CNS site including the SNSWP and Yard is modeled with ICM's 2-D surface flooding module. This portion of the modeling extent is known as the 2-D Zone. The buildings within the 2-D Zone and their associated hydraulic features are modeled as 1-D sub-catchments that connect and link to the 2-D Zone using the weir and sluice gate model options to simulate roof drainage from edges/parapet walls and scuppers, respectively. Roof drains (flat roofs) were conservatively assumed 100 percent blocked during the simulation to route additional water to the yard ground surfaces. Blocking roof drains allow more water to spill over the roof edge or scuppers adding more water to the ground surface.

The required 2-D model mesh Digital Terrain Model (DTM), Figure 2.1.2-1, was developed using CNS site-specific aerial photography and LiDAR survey data dated 2010. (References 46 and 47) This survey was also used to determine the location and footprint dimensions of buildings and permanent features (i.e., security barriers) in the yard. Building drainage (e.g., parapet elevations, scuppers, etc.) for all buildings within the security perimeter of CNS was developed using engineering drawings provided by Duke Energy. Site conditions were observed by the modeling team through conducting a CNS Yard walkdown which was documented by report dated November 2013 (Reference 26).

Model boundary conditions were assigned to locations that would not impact flow calculations in areas of the CNS Yard near critical equipment and buildings. This was accomplished by reviewing the site topography and the 2-D Zone extents which were chosen by the modeler to

end on or near the watershed delineation features surrounding the site, Figures 2.1.2-1 and 2.1.2-2.

All buildings within the 2-D Zone, including but not limited to the reactor, auxiliary, and turbine buildings, were created as 1-D sub-catchments and modeled using the Storm Water Management Model (SWMM) rainfall-runoff and routing simulation options within ICM. Each roof section was modeled as a conceptual volume defined by an elevation-area relationship. The roof surfaces use a Curve Number (CN) of 98. SWMM runoff routing values (Manning's *n* values) were selected based on roof material types. Roughness values for each material type are provided in Table 2.1.2-1 (SOF 2.1.2-01).

FIGURE 2.1.2-1
CNS DIGITAL TERRAIN MODEL 3-DIMENSIONAL PERSPECTIVE LOOKING NORTH NO BUILDINGS SHOWN

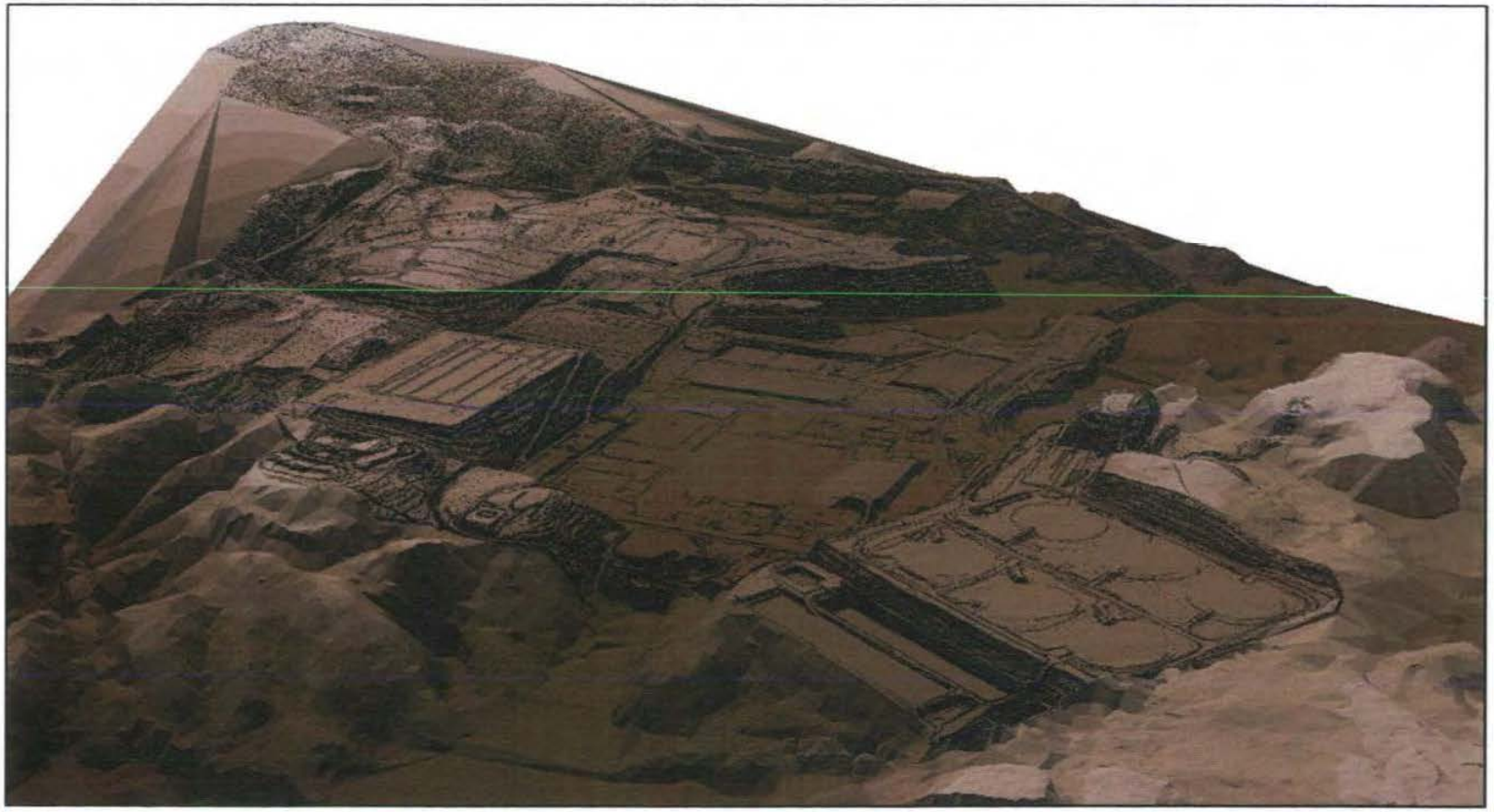


FIGURE 2.1.2-2
CNS DIGITAL TERRAIN MODEL EXTENTS SHOWN BY RED LINE

HDR

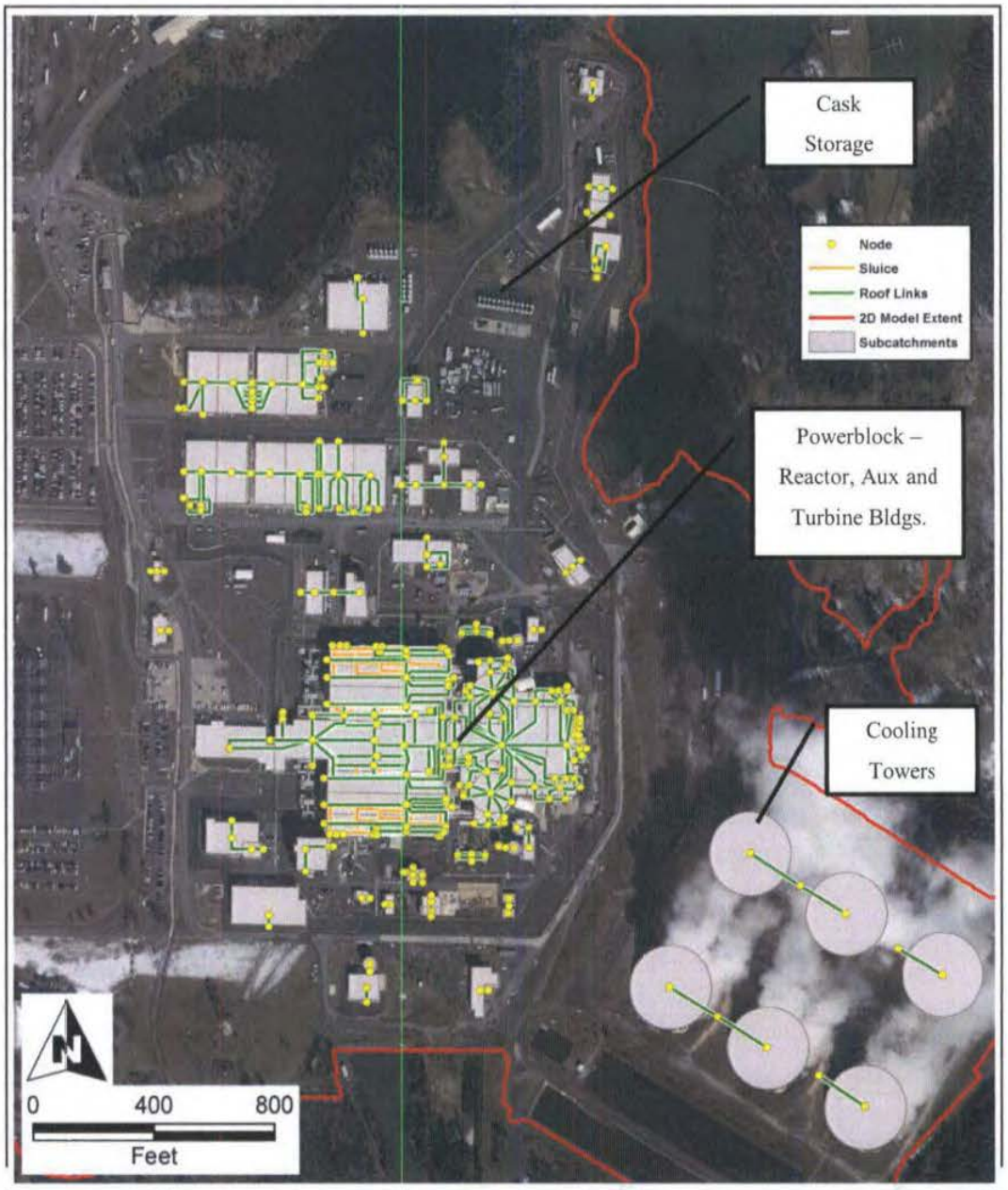
Catawba Nuclear Station: 2D Model Extent
Calculation CNS-194292-009 (LIP and LIP 2-D Model Results)
Infoworks ICM: Base Case
Hydrology: LIP

TABLE 2.1.2-1
ROOF RUNOFF ROUTING VALUES

Surface Material	Manning's *n* value
Steel	0.011
Asphalt	0.030
Concrete	0.016
Tent	0.010

Overflow of the roof gutter or parapet system was accounted for by use of a conceptual weir that discharges to a location based on assigned flow paths (e.g., discharges onto ground or another roof). All conceptual weirs were modeled with a discharge coefficient of approximately 2.6 (ICM model input of 1.43 in metric), a typical minimum discharge coefficient for broad crested weirs per the United States Department of The Interiors Geological Survey Circular 397, Discharge Characteristics of Broad-Crested Weirs (Reference 57), and a modular limit of 0.9. These locations are directly applied to the 2-D Zone, at downspout locations where appropriate, or other roof sections depending on roof geometry. Figure 2.1.2-3 shows the ICM model building roof connectivity, including 1-D sub-catchment connections (i.e., weirs) and sluice gates (i.e., scuppers) (SOF 2.1.2-02).

FIGURE 2.1.2-3
CNS DIGITAL TERRAIN ROOF DRAINAGE CONNECTIVITY



HDR

Catawba Nuclear Station: Complete ICM Model Connectivity
Calculation CNS-194292-009 (LIP and LIP 2-D Model Results)
Infoworks ICM: Base Case
Hydrology: LIP

Note: Building IDs were not added to this figure to preserve clarity of the roof connectivity links.

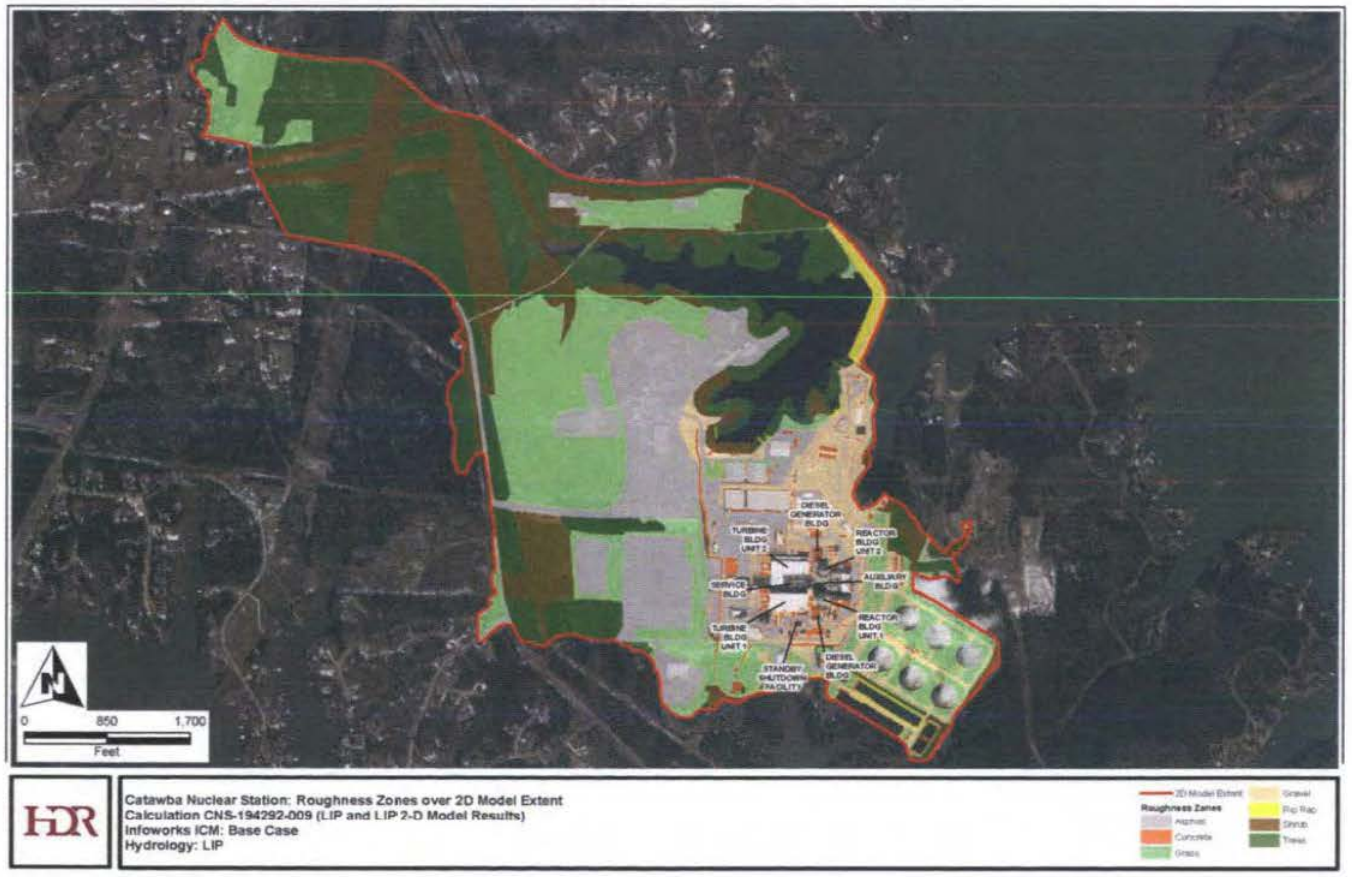
Overland flow surface roughnesses for the 2-D Zone were classified as grass, gravel, riprap, or concrete/asphalt. Table 2.1.2-2 provides the associated Manning's *n* values used for each roughness zone (SOF 2.1.2-03). Figure 2.1.2-4 shows the model boundaries of each roughness zone (SOF 2.1.2-04).

TABLE 2.1.2-2
ROUGHNESS ZONE MANNING'S *n* VALUES

Surface Material	Manning's *n* value
Grass	0.030
Gravel	0.023
Rip Rap	0.036
Concrete/Asphalt	0.013
Trees	0.100
Shrubs	0.070

All areas in the 2-D model mesh, including contributing watersheds outside of the main CNS Yard, assume 100% runoff maximizing the inundation of the site for the LIP reevaluation of flood hazard potential following guidance in NUREG/CR 7046.

FIGURE 2.1.2-4
CNS ICM MODEL YARD ROUGHNESS ZONE BOUNDARIES



2.1.3 Security Barrier Model Features

Changes to the local site topography and support buildings have taken place since original construction and have been captured in calculation CNC-1114.00-00-0040 Rev 29 (Reference 8). Changes to local area runoff/drainage conditions can generally be grouped by modifications within the secure area and outside of the secured area. Figure 1.4-1 (Section 1.4) identifies site landscape/structure change areas that have been added to the ICM-site model.

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

During the PMP event, these security barriers act to redirect surface flow patterns. They are modeled as permanent porous polygons with a porosity of 0 percent (i.e., fully obstructed) that raise the ground elevation 2.66 feet (32 inches).

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

provided by Duke Energy and visual observations made during HDR's October 8, 2013, *Fukushima NTTF 2.1: Catawba Nuclear Station Flood Hazard Reevaluation Site Walkdown in Support of 2-D Terrain Model* (Reference 26). Figure 2.1.3-1 shows the location of the modeled security barriers.

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

This security barrier was observed to be comprised

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

The sections comprised of the fencing were modeled at a height of 7 feet and a calculated porosity of 82 percent (i.e., 18% of the cross-sectional area blocked by a 17-gauge cyclone fence and steel grating). The gate sections were modeled at a height of 3.66-feet (44 inches) and a porosity of 0 percent (i.e., fully obstructed). Due to the difficulty in estimating the percent blockage of the fencing security barrier, a sensitivity analysis was performed on the porosity of the fenced security barrier during

the LIP event. This analysis assumed twice the cross-sectional area was blocked by the cyclone fencing and steel grate material resulting in a porosity of 64 percent (i.e., 36 percent blocked). This sensitivity analysis indicated that the modeled peak water surface elevation and general flow paths are not significantly affected by realistic but conservative changes to the porosity. For this reason, the calculated porosity of 82 percent was utilized in the final model setup. (SOF 2.1.3-01) Figure 2.1.3-1 shows the location of the modeled northern security barrier and is identified as "Vehicle barrier-fence" in the figure legend.

In addition to the user inputs outlined above, the breaklines utilized in the DTM development were applied and the building polygons within the 2-D Zone were modeled as voids when creating the mesh. The resulting mesh contains 297,977 triangles and 136,634 elements. The approximate minimum, maximum, and average element areas are 1 ft^2, 1,371.8 ft^2, and 194.9 ft^2, respectively.

FIGURE 2.1.3-1
CNS ICM MODELED SECURITY BARRIERS

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

2.1.4 CNS ICM Model Results

The evaluation of potential flooding on the CNS site was performed by applying the point PMP to the ICM model as 5-minute interval rainfall intensities (in/hr) as shown in Section 2.1.1, Figure 2.1.1-4. This rainfall profile was applied to both the 2-D Zone and the 1-D sub-catchments (roofs). The ICM model was used to create a 6-hour simulation to evaluate the PMP maximum flooding effects within the CNS Yard and SNSWP under existing modeled site characteristics. PMP runoff to the SNSWP and responding storage in the pond was conservatively modeled assuming no outflow from the existing pipe outlet spillway.

Results of the LIP modeling for inundation locations and levels are shown in figures included in Appendix B of calculation CNS-294292-009, Rev. 0. (Reference 16). The model output included in Reference 16 includes figures related to the location of the site, development of the PMP, flow patterns and the LIP maximum flood depths and elevations in the CNS Yard with no Yard catch basin and roof drains active. Runoff from the roofs of building was directly added to the 2-D mesh as noted in Section 2.1.2. Flood inundation level versus time hydrographs in Reference 16 show residual water remaining at the model output node points beyond the initial falling limb of the hydrograph and extending to hour 6 of the LIP simulation. The remaining water shown at these nodes is the result of the relatively flat CNS Yard surface and the non-active catch basins. This is expected as CNS Yard catch basins are typically the lowest elevation points in the CNS Yard by design and surrounding areas are sloped to drain to these points. The catch basin blockage is a conservative modeling assumption recommended by guidance provided in NRC NUREG/CR 7046 (Reference 60). It should be noted that higher maximum water surface elevations characteristically correspond to higher ground surface elevations and not necessarily a greater depth of water.

The effects of the LIP result in variable water surface elevations modeled across the entire CNS Yard. Three primary locations have been identified for reporting typical summary results:

1. the perimeter around the main complex buildings and the diesel generator area,

   A representative maximum water surface elevation level in the CNS Yard around the main complex (i.e., Auxiliary, Reactor, and Turbine Buildings) ranges from approximately 594.9 ft msl to 595.6 ft msl. (SOF 2.1.4-01) Previously, the maximum water surface elevation levels in the North (Unit #2) and South Yard (Unit #1) portions of the CNS Yard were 595.9 ft msl and 594.9 ft msl (Reference 8), respectively. A representative maximum water surface elevation level in the CNS Yard around Diesel Generator Building Unit #1 and Unit #2 are approximately 594.9 ft msl and 595.5 ft msl, respectively (SOF 2.1.4-02). Previously, the maximum water surface elevation levels for Diesel Generator Building Unit #1 and Unit #2 were 594.9 ft msl and 595.9 ft msl (Reference 8), respectively.

2. the Cask Storage area in the northern portion of the CNS Yard

   A representative average maximum water surface elevation level of flow in the CNS Yard around the Cask Storage area is approximately 602.1 ft msl. (SOF 2.1.4-03)

3. the SNSWP

   A representative maximum water surface elevation level for the SNWSP is approximately 583.0 ft msl (SOF 2.1.4-04). Previously, the maximum SNSWP elevation was determined to be 583.5 ft msl (Reference 8).

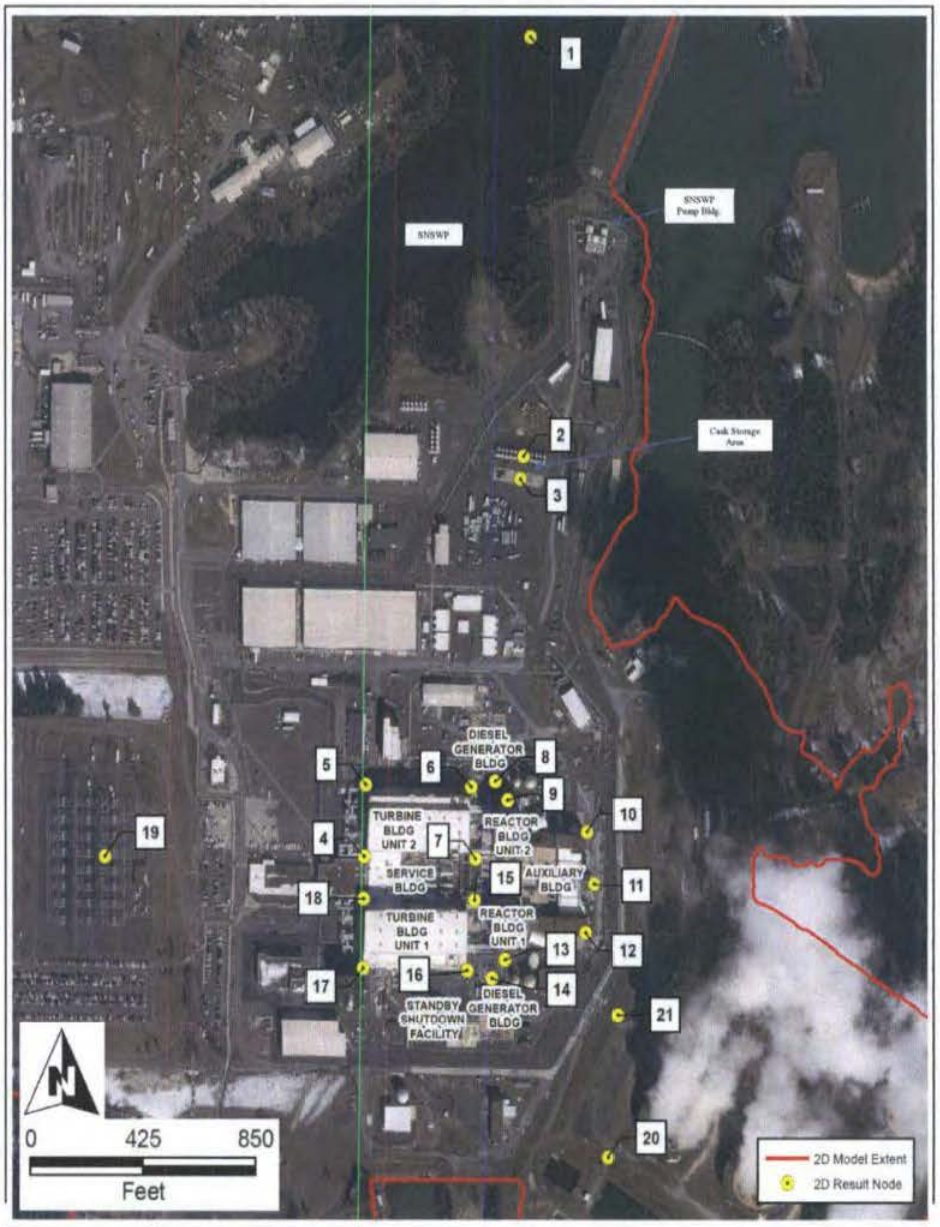
ICM model results for the LIP indicate that the protective berms/curbs around the switchyard and cooling tower sub-areas are overtopped and a portion of the sub-basin runoff does contribute to flooding on the CNS Yard. It should be noted that previous analyses indicated that the protective berm/curb in the switchyard and cooling tower areas were not overtopped.

Twenty-one (21) result nodes defining CNS Yard areas of interest are presented in Figure 2.1.4-1 and Table 2.1.4-1. Figure 2.1.4-2 shows the location of twenty-two (22) safety building doors

related to SSCs result nodes and Table 2.1.4-2 provides the peak depth, elevation, and maximum velocity modeled at the nodes during the 6-hour simulation. Additionally, both tables provide the North Carolina State Plane NAD83 Coordinate System Northing and Easting in US survey feet along with a brief description of the location and the duration of inundation (SOF 2.1.4-05). Inundation durations were approximated for each location assuming a flood arrival time defined by a flood depth of 0.1 ft. The end of inundation was determined when the flood depth was within 0.1 ft of the end of simulation depth (e.g., if the depth at the end of the simulation is 0.1 ft the inundation is assumed to end of the time when flood depths fall below 0.2 ft). This provides a consistent method of inundation duration calculation between all locations.

For comparison, Table 2.1.4-1 includes peak LIP design basis 1-D water surface elevations from calculation CNC-1114.00-00-0040 Rev 29. Due to the limits of the 1-D model used in the design basis analysis, a node-by-node comparison is not possible and some locations lack suitable information for comparison. Maximum ICM LIP model inundation elevations in the CNS Yard area are shown in Figure 2.1.4-3. Areas outside the CNS Yard area have been clipped from this figure as they are at higher ground elevations than covered by the color scale used in the figure and do not impact SSCs identified in the 2.3 Walkdown report (Reference 9). Water surface elevation and depth hydrographs as well as velocity hydrographs at each node location for the LIP simulation are provided in CNS-194292-009, Rev 0. (Reference 16).

FIGURE 2.1.4-1
CNS ICM MODEL PMP RESULT NODE LOCATIONS



HDR

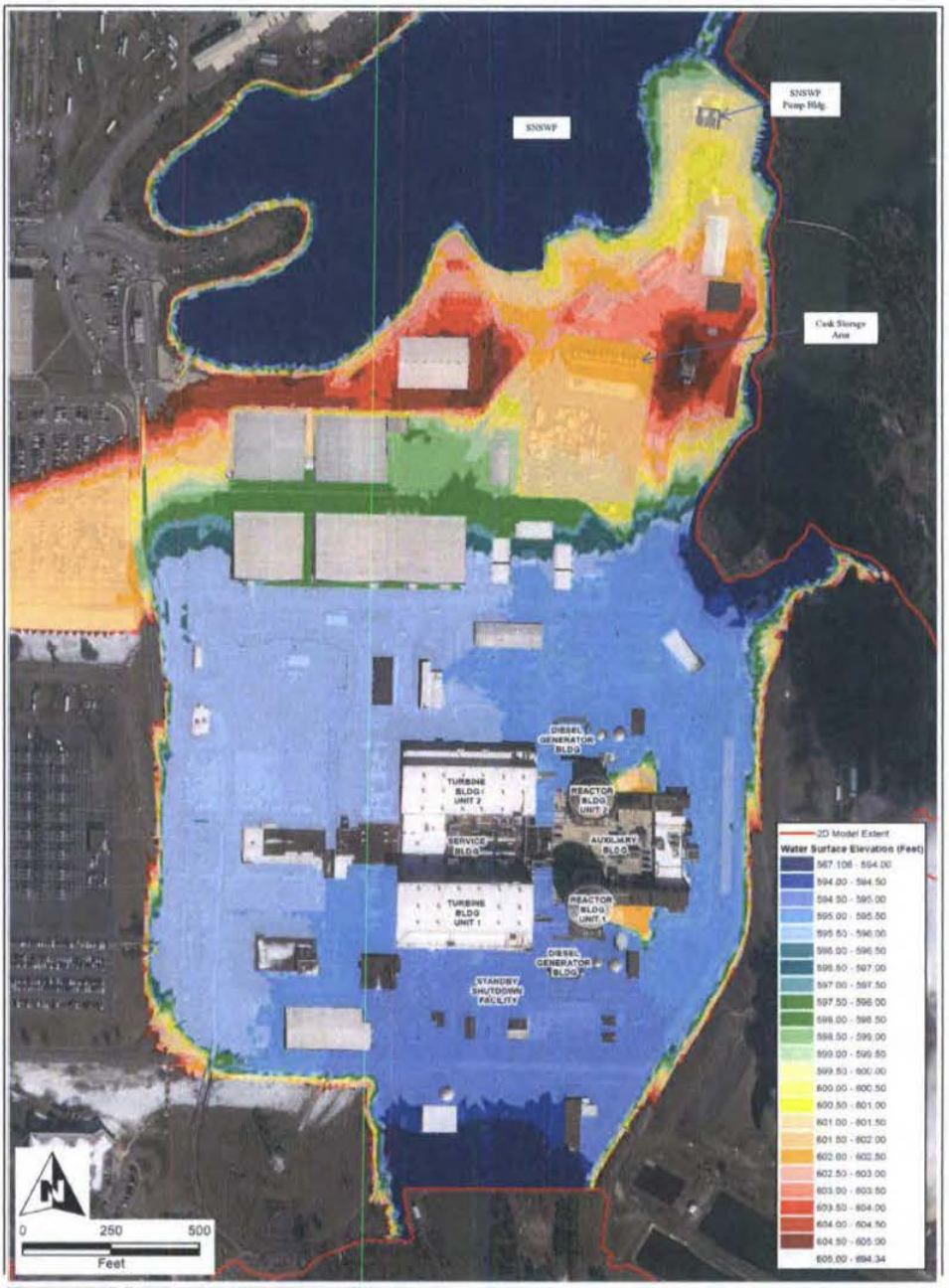
Catawba Nuclear Station: 2D Result Node Locations
Calculation CNS-194292-009 (LIP and LIP 2-D Model Results)
Infoworks ICM: Base Case
Hydrology: LIP

FIGURE 2.1.4-2

CNS ICM MODEL PMP RESULT NODE LOCATIONS AT DOORS TO BUILDINGS WITH IDENTIFIED SSCs

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

FIGURE 2.1.4-3
CNS ICM MODEL LIP RESULT MAXIMUM WATER ELEVATIONS AROUND THE MAIN POWER BLOCK STRUCTURES



HDR

Catawba Nuclear Station: Maximum Water Surface Elevation Main Complex
Calculation CNS-194292-009 (LIP and LIP 2-D Model Results)
Infoworks ICM: Base Case
Hydrology: LIP

Note that areas outside the CNS Yard area have been clipped form this figure for purposes of presentation of color levels.

TABLE 2.1.4-1
CNS ICM MODEL LIP RESULT NODES

Node	Description	Northing	Easting	Maximum Depth, ft	Maximum Elevation, ft msl	Maximum Velocity, ft per second	Inundation Duration, Hrs:min	Design Basis Maximum Water Surface Elevation, ft mls[2]
1	SNSW Pond	482984.23	1380827.67	9.1	583.0	2.5	1:35	583.5
2	Cask Storage Pad 1	481583.85	1380801.93	0.3	602.1	0.7	0:40	NA
3	Cask Storage Pad 2	481510.18	1380789.02	0.3	602.0	0.8	0.50	NA
4	Turbine Building Unit #2 - Southwest Corner	480254.85	1380198.79	1.8	595.6	0.9	3:50	595.9
5	Turbine Building Unit #2 - Northwest Corner	480493.57	1380204.65	1.8	595.6	1.3	4:20	595.9
6	Turbine Building Unit #2 - Northeast Corner	480486.25	1380603.01	1.7	595.5	1.1	4:00	595.9
7	Turbine Building Unit #2 - Southeast Corner	480247.16	1380618.02	2.1	595.5	0.2	4.35	595.9
8	Diesel Generator Building Unit #2	480505.66	1380694.18	1.3	595.5	0.5	1:45	595.9
9	Reactor Building Unit #2	480442.68	1380739.21	1.9	595.4	0.5	5:05	595.9
10	Auxiliary Building - North Corner	480336.13	1381037.25	1.4	595.3	1.1	2:35	595.9
11	Auxiliary Building	480160.38	1381065.81	1.8	595.2	1.9	3:30	594.9- 595.9
12	Auxiliary Building - South Corner	479998.55	1381031.76	1.5	595.2	1.6	3:40	594.9
13	Reactor Building Unit #1	479907.01	1380730.06	1.2	594.9	0.3	3:55	594.9
14	Diesel Generator Building Unit #1	479846.60	1380680.27	0.9	594.9	0.3	1:40	594.9
15	Turbine Building Unit #1 - Northeast Corner	480107.66	1380615.09	1.4	594.9	0.0	2:20	594.9
16	Turbine Building Unit #1 - Southeast Corner	479872.23	1380588.00	1.1	594.9	1.2	2.15	594.9
17	Turbine Building Unit #1 - Southwest Corner	479880.65	1380194.76	2.1	595.4	2.6	3:25	594.9
18	Turbine Building Unit #1 - Northwest Corner	480110.96	1380196.96	1.8	595.5	0.0	4:20	594.9
19	Switchyard	480253.80	1379222.00	0.7	632.8	0.5	4:35	NA
20	WC Pond Berm	479254.00	1381117.10	0.6	620.6	0.7	3:15	NA
21	Cooling Tower Berm	479726.00	1381156.10	0.4	620.7	2.2	1.50	NA

Notes:
[1] Reference Figure 2.1.4-1 for yard node locations.
[2] Existing CNS design basis maximum water elevations are from CNC-1114.00-00-0040 Rev 29.

TABLE 2.1.4-2
CNS ICM MODEL LIP RESULT LOCATIONS AT DOORS TO BUILDINGS WITH IDENTIFIED SSCs NODES

Door No.	Bldg.	Destination	Description	Northing	Easting	Maximum Depth, ft	Maximum Elevation, ft msl	Maximum Velocity, ft per second	Inundation Duration, Hrs:min	Design Basis Maximum Water Surface Elevation, ft mls[2]
AX300A	U2 UHI	Sump B	543+0 Common Area (Room 214)	1380796.997	480449.787	1.9	595.4	0.7	5:15	595.9
AX301B	U1 UHI	Sump A	543+0 Common Area (Room 216)	1380786.9	479904.6	1.2	594.9	1.1	4:00	594.9
AX660	U2 Ext DH	Sump C	U2 CA Pump Rm (Room 260)	1380739 157	480437.66	1.9	595.4	0.5	5:05	595.9
AX661	U1 Ext DH	Sump D	U1 CA Pump Rm (Room 250)	1380730.97	479912.27	1.1	594.9	0.3	2:15	594.9
AX656B	U2 El Pen Rm	Sump C	U2 CA Pump Rm (Room 260)	1380659.697	480346.072	1.9	595.5	0.4	3:55	595.9
AX658A	U1 El Pen Mr.	Sump D	U1 CA Pump Rm (Room 250)	1380651.8	480007.3	1.0	594.9	0.4	2:00	594.9
AX600	U1 FB	FDT	543+0 Common Area (Room 255)	1381012.77	480005.45	1.6	595.3	0.3	3:05	594.9
AX627	U2 FB	FDT	543+0 Common Area (Room 255)	1381008.6	480269.7	0.9	595.2	1.3	1:30	595.9
AX629	U2 FB	FDT	543+0 Common Area (Room 255)	1381020.2	480333.5	2.0	595.3	0.4	3:30	595.9
AX600B	U1 FB	FDT	543+0 Common Area (Room 255)	1380987.7	479986.4	1.3	595.3	0.1	2:05	594.9
AX629B	U2 FB	FDT	543+0 Common Area (Room 255)	1381002.3	480352.9	0.8	595.1	0.9	1:25	595.9
AX664D	ASB	FDT	543+0 Common Area (Room 255)	1381032.1	480091.9	1.3	595.3	0.5	1:55	594.9- 595.9
AX605D	ASB	FDT	543+0 Common Area (Room 255)	1381032.2	480101.3	1.5	595.2	0.2	5:00	594.9- 595.9
AX617	ASB	FDT	543+0 Common Area (Room 255)	1381024.3	480207.5	1.5	595.2	0.2	5:00	594.9- 595.9
AX671	ASB	FDT	543+0 Common Area (Room 255)	1381036.3	480215.0	1.4	595.3	0.2	2:30	594.9- 595.9
AX666A	ASB	FDT	543+0 Common Area (Room 255)	1381017.2	480265.2	1.9	595.3	0.3	3:35	594.9- 595.9
AR5	ASB	FDT	543+0 Common Area (Room 255)	1381024.4	480198.9	1.6	595.3	0.3	3:05	594.9- 595.9
AR6	ASB	FDT	543+0 Common Area (Room 255)	1381015	480119.0	1.6	595.2	0.0	4:10	594.9- 595.9
AX302B	D/G 1A	D/G Sump	1A D/G Room (Room 302)	1380724.154	479799.405	1.7	594.9	1.2	3:30	594.9
AX304B	D/G 1B	D/G Sump	1B D/G Room (Room 304)	1380630.07	479801.22	2.3	594.9	0.5	4:35	594.9
AX306B	D/G 2A	D/G Sump	2A D/G Room (Room 306)	1380751.4	480517.6	2.1	595.4	0.8	3:35	595.9
AX308B	D/G 2B	D/G Sump	2B D/G Room (Room 308)	1380633.6	480519.62	2.0	595.5	0.6	4:25	595.9

Notes:
[1] Reference Figure 2.1.4-2 for door node locations.
[2] Existing CNS design basis maximum water elevations are from CNC-1114.00-00-0040 Rev 29

2.2 Flooding in Reservoirs

The Duke Energy dams and reservoirs on the Catawba River are regulated by the FERC under Catawba-Wateree FERC Project No. 2232 and are maintained to standards required by 18 CFR Subpart 12. The flood standard imposed by the FERC on the Duke Energy dams is the PMF. The NRC requires the estimation of the design-basis flood that would cause adverse effects to the CNS site. By NRC definition, "a design-basis flood is a flood caused by one or an appropriate combination of several hydrometeorological, geoseimic, or structural-failure phenomena, which results in the most severe hazards to SSCs important to the safety of a nuclear power plant". The CNS site adjoins Lake Wylie which is formed by the Catawba River and Duke Energy's Wylie Dam. As shown in Figure 2.2-1, the

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

The hydrometeorological flood standard imposed on the CNS site is the PMF associated with one of the Duke Energy dams/reservoirs adjoining or located upstream of the CNS site.

FIGURE 2.2-1
OVERVIEW AND PROFILE OF CATAWBA WATEREE RIVER DAMS

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

Section 5.5.1 of American Nuclear Society (ANS) 2.8, under "Hydrologic Dam Failures," states that "critical dams should be subjected analytically to the probable maximum flood from their contributing watershed. If a dam can sustain this flood, no further hydrologic analysis shall be required." There are (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) All are licensed by Duke Energy and are maintained in accordance with FERC guidelines. Dam-specific PMP and PMF documentation was reviewed, demonstrating that each of the dams can safely pass the FERC-approved PMF. A description of the upstream dams, including FERC Exhibit F drawings showing layout and sections of dam structures, is provided in Appendix D.

The Duke Energy Catawba-Wateree Developments underwent a PMF evaluation in 1992 (References 28 through 34) to determine each dam and reservoir hydrologic (PMP) and hydraulic (PMF) performance to maintain compliance with FERC regulations. Law Environmental Inc., Kennesaw, Georgia, developed PMF evaluations (Reference 37) for the Catawba-Wateree FERC projects using the USACE HEC-1 software to develop rainfall-runoff hydrographs from the sub-basins that comprise the Catawba River basin. The National Weather Service's DAMBRK model was used to route the PMF flood waters through the respective Developments. The Catawba River PMP values for the respective Duke Energy Developments are based on Hydrometeorological Reports 51/52 and use elliptical-shaped isohyetal patterns to maximize the PMP rainfall over a given Catawba-Wateree development's basin. The hydrologic and hydraulic analysis for the FERC Catawba-Wateree developments was the basis for the CNS-2.1 Fukushima Study for flooding from reservoirs. The FERC-approved Catawba-Wateree Legacy HEC-1 model (Reference 37) was adapted to develop the 2013 Fukushima 2.1 PMF inflow hydrographs for the various Catawba-Wateree Developments. The 2013 Fukushima 2.1 PMF is based on a 216-hour rainfall event comprised of three 72-hour precipitation sub-events including the 40 percent PMP, 0-rainfall, and the HMR51 PMP.

The Legacy HEC-1 model was used to develop two sets of inflow hydrographs (216-hour event) for Cowans Ford Dam and Wylie Dam consistent with the respective sub-basins designated for each Development during the 1992 and 1998 FERC PMF studies (References 37, 38, 39, and

12). The 22 sub-basins that comprise the Wylie Dam drainage basin are shown in Figure 2.2-2 and 2.2-3.

FIGURE 2.2-2
WYLIE DEVELOPMENT HYDROLOGIC SUB-BASIN FROM 1992 FERC PMF STUDY

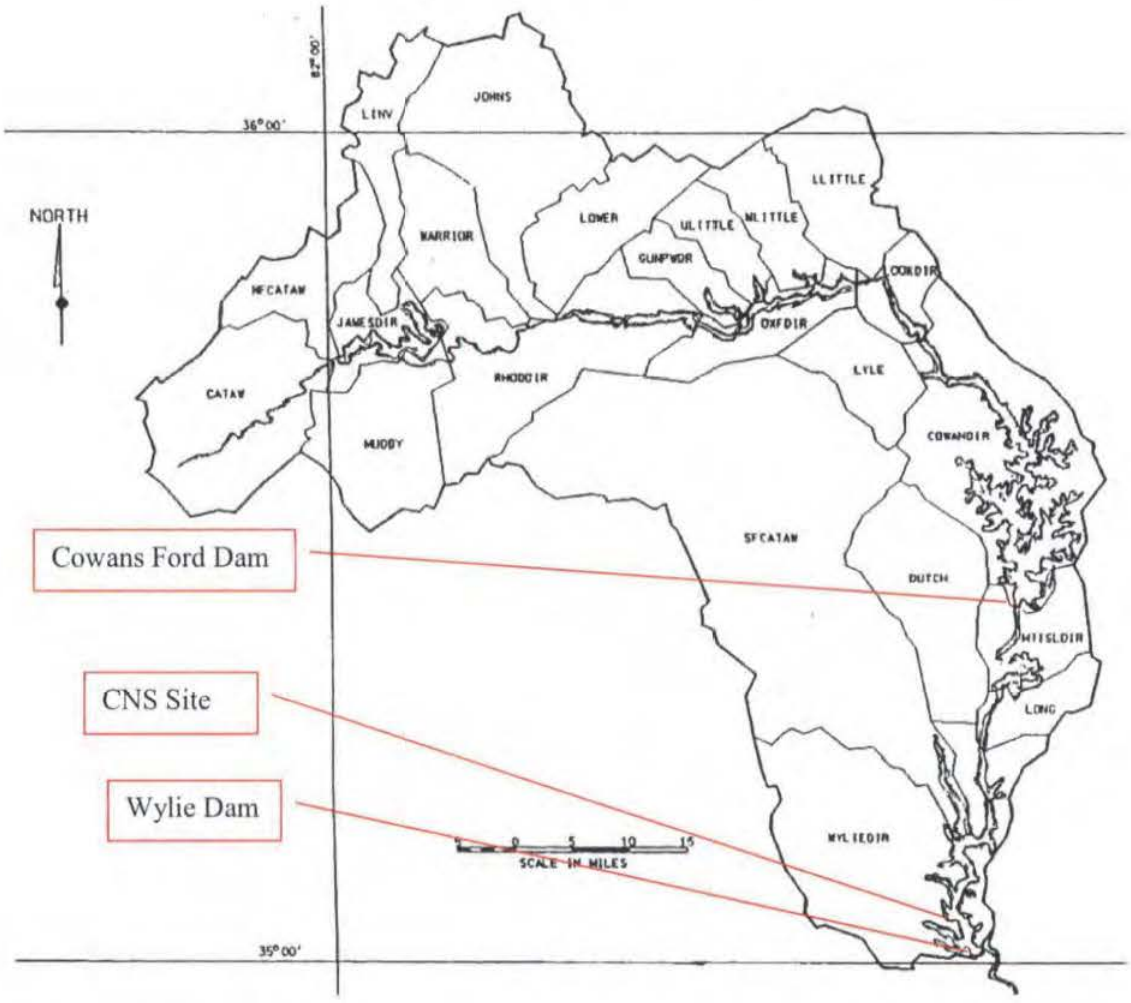


FIGURE 2.2-3
CATAWBA-WATEREE BASIN INCLUDING SUB-BASIN DELINEATION AND DESIGNATED SUB-BASIN NUMBER. THE CNS 2.1 FUKUSHIMA STUDY IS LIMITED TO TWENTY TWO SUB-BASINS, NUMBERS 0-21

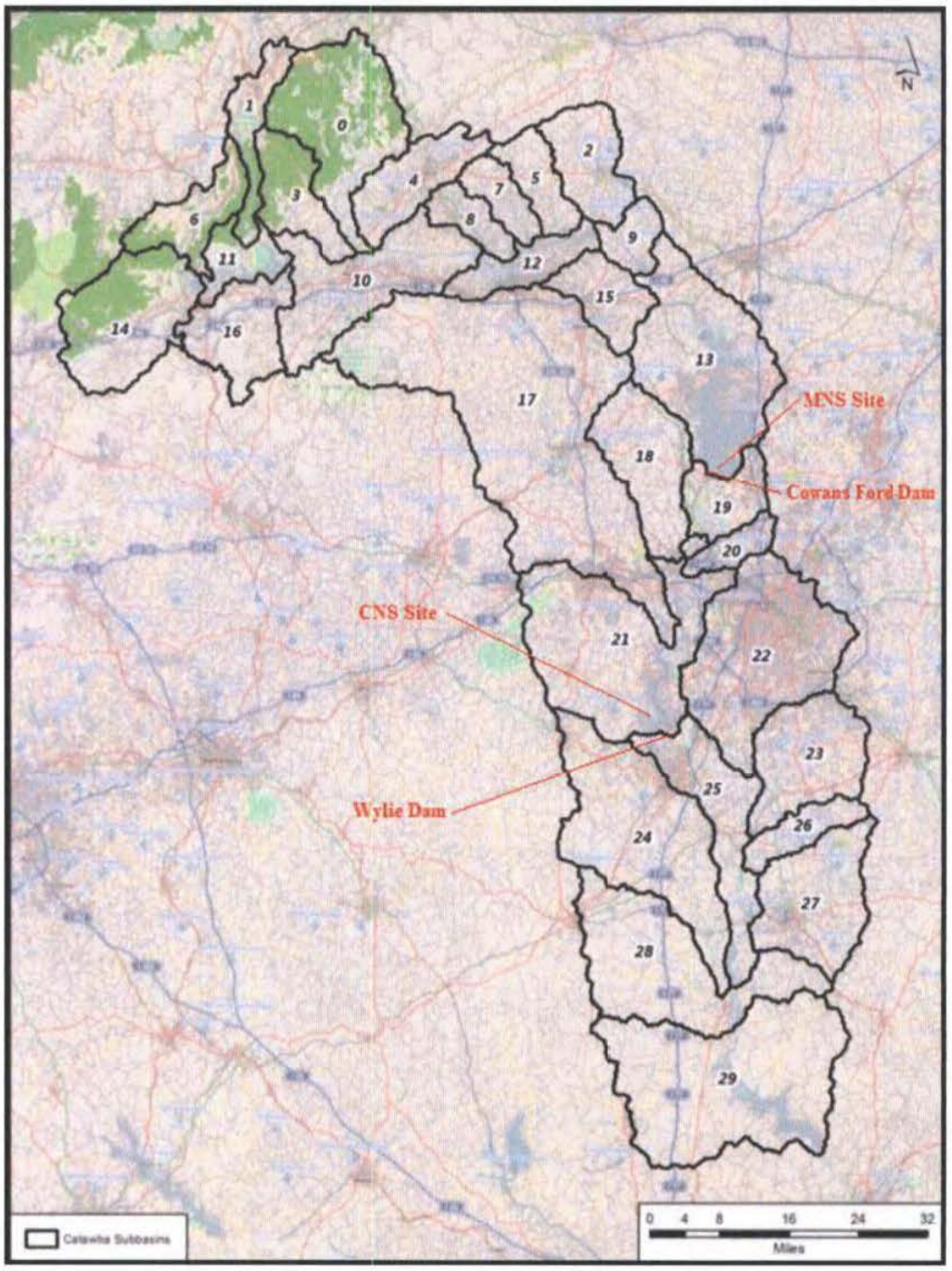


Table 2.2-1 provides a list of the sub-basins associated with each of the respective Duke Energy dams.

TABLE 2.2-1
CATAWBA-WATEREE PMF ANALYSIS SUB-BASINS

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

The 1992 Catawba-Wateree DAMBRK PMF routing model used 93 cross sections to describe the 225-mile profile length and reservoir geometry of the Catawba-Wateree River system shown in Figure 2.2-1, including the [redacted] reservoirs. For the Fukushima 2.1 reservoir flooding evaluation, the DAMBRK routing model was replaced to provide current state-of-practice software using the USACE, Hydrologic Engineering Center – River Analysis System (HEC-RAS) version 4.1. HEC-RAS was incorporated for its options of a Geographic Information Systems (GIS) interface, multiple river branch/tributary interface, dam spillway and breaching options, interactive user interface and greater cross-section detail than the previous one-branch DAMBRK model capabilities. The hydraulic model was identified as Catawba River System HEC-RAS Model (Catawba River Model). The Catawba River Model incorporates the area bounded by the headwaters of [redacted] through the Catawba River tailrace [redacted] Dam, including significant tributaries to the Catawba River/Duke Energy Development reservoirs. The main stem Catawba River Model length is approximately 165 miles. The 2013 Catawba River Model accounts for significant tributaries off the main stem of the Catawba River and within the [redacted] respective Duke Energy Development reservoirs. The respective reservoir-tributary reaches and their geo-referenced cross sections are used to account for reservoir volume in the HEC-RAS model. The 2013 Catawba River Model identified 47 main stem and tributary reaches that required lateral/direct inflow hydrographs versus the 22 main stem tributary sub-basins employed in the HEC-1 model and 16 lateral inflows in the DAMBRK model. The additional lateral and direct inflow hydrographs for the 2013 HEC-RAS model were developed by applying a drainage area weighting method to distribute the inflow between the original HEC-1 inflow hydrographs and the additional lateral/direct inflow hydrographs in the 2013 HEC-RAS model.

(b)(3) 18 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3) 18 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3) 18 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

(b)(3) 18 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Verification of Catawba River Model was previously conducted during development work for supporting FERC-required analyses (Reference 37, 1992 Law Engineering Catawba-Wateree PMF Study). The 1992 model verification was based on using available rainfall and runoff data from across the entire 4,750-square-mile drainage basin (Lake James to Wateree). Review of the storms used for 1992 model calibration and verification indicated they were generally moderate to low in return frequency; therefore, the storms selected for the Fukushima 2.1 HHR HEC-RAS analysis model verification were based on the largest storms of record available.

The FERC-approved Catawba-Wateree PMF model hydrology was based on development of synthetic unit hydrographs for each of the 22 sub-basins, numbered 0 to 21, shown in Table 2.2-1 (Reference 18). To respond to guidelines presented in NUREG/CR-7046 addressing application of linear unit hydrograph theory in PMF analysis, the largest historic floods of record for the Catawba River were used to test the models ability to simulate historic flood elevations along the upper segments of the Catawba River (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The Catawba River Model (HEC-1 and HEC-RAS) was used to verify the ability of the model unit hydrographs and routing parameters (cross sections and roughness) to reproduce historic flood levels of record. Floods of record for 1916 and 1940 that occurred over the drainage basin represented in Figure 2.2-2 were reconstructed from historic precipitation and runoff records (SOF-2.2-01). The 1916 and 1940 events were not used in the 1992 Catawba-Wateree PMP/PMF Study (Reference 37) development of the synthetic regional unit hydrograph; therefore, they were judged to be a valid verification record for the hydrologic and hydraulic river model. The July 1916 hurricane precipitation event is identified as the "Flood of Record" for the Catawba River. The Duke Energy hydro dams in existence at that time and within the 2.1 Fukushima Study areas are (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam experienced a peak discharge of approximately (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) cfs during the July 1916 event representing 66 percent of the 1992 (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) discharge of (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) cfs (Reference 31). The August 1940 event is described as the largest flood of record for the (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 August 1940 rainfall over the upper third of the Catawba River basin varied between 100-year and 400-year return periods (Reference 27).

The Catawba River sub-basin precipitation comparison between the (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) July 1916, and August 1940 events are shown in Table 2.2-2.

TABLE 2.2-2
MODEL VERIFICATION SUB-BASIN PRECIPITATION COMPARISON

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

The basin average rainfall for the three events shown in Table 2.2-2 are:

- Cowans Ford PMP (FERC-1992) – 33.25 inches
- July 1916 – 19.94 inches, or 60 percent of the Cowans Ford PMP

- August 1940 – 13.17 inches, or 39.6 percent of the Cowans Ford PMP.

The (b)(3):16 U.S.C. § 824o-1(d); (b)(4); (b)(7)(F) Dam experienced an (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) and the Catawba River Model was modified to reflect physical site parameters (embankment and spillway crest elevation) at the time of the event. Catawba River Model verification results are presented for both non-failure and failure of the Lookout Shoals structure in Table 2.2-3.

TABLE 2.2-3
CATAWBA RIVER MODEL RESULTS COMPARISON WITH 1916 FOOD EVENT

1916 Observations and Calculations	2013 HEC-RAS w/o Breach	2013 HEC-RAS w/Breach
(b)(3):16 U.S.C. § 824o-1(d); (b)(4); (b)(7)(F)		

The Catawba River Model dam discharge parameters were modified to reflect physical site parameters that existed at the time of the August 1940 flood event. This was a required step in the verification process to replicate "as-existed" conditions in 1940. Catawba River Model results for the 1940 event are presented in Figure 2.2-4.

The modeled results reflect good correlation with the observed conditions for both the July 1916 and August 1940 flood events. The results support the use of SCS synthetic unit hydrographs developed in 1992 for the FERC-approved Catawba-Wateree HEC-1 Model. The use of these unit hydrographs is appropriate to account for the hydraulic performance of the (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) events used for the Fukushima 2.1 HHR. Additional details regarding the model verification are presented in Appendix E, "FERC-Approved Cowans Ford PMF and its Application to the Catawba River in Compliance with NUREG/CR-7046 – Appendix I".

FIGURE 2.2-4

CATAWBA RIVER MODEL, AUGUST 1940 FLOOD PROFILE ALONG CATAWBA RIVER.

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

The following sections summarize use of the Catawba River Model to perform reservoir flooding evaluation for the CNS site.

Baseline Model Runs:

Initial non-failure model runs for both the fair-weather and PMF events were performed to establish baseline hydraulic model performance results for comparison with future modeling scenarios in determining relative impacts at CNS. The non-failure model runs were used to test applicable hydro plant and spillway operations criteria including available hydro units, starting reservoir elevation, spillway capacity constraints, and design storm event. The fair-weather design storm is the base flow for the Catawba River and its modeled tributaries within the 2013 Catawba River Model.

The Duke Energy Developments on the Catawba River are regulated by the FERC under Catawba-Wateree FERC Project No. 2232. The Catawba-Wateree relicense application was submitted to the FERC in 2006 including the Comprehensive Relicensing Agreement, Appendix A: Proposed License Articles, A-1.0 Reservoir Elevation Articles (Reference 11). The FERC license establishes reservoir target elevations that are below normal full pond elevations for each of the Developments. External flooding evaluations for the CNS 2.1 Fukushima Study use the FERC reservoir target elevations as initial reservoir elevations. This assumption is consistent with CNS design basis analysis as shown in UFSAR Table 2-59 and is supported by historic reservoir operating levels. Calculation CNS-194292-010, Rev 0 provides supporting details for the [redacted] reservoir operating levels established during the FERC relicensing of the hydro projects in 2006.

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

All Duke Energy FERC Developments have debris management programs established in follow up to their Catawba-Wateree License with the FERC. However, in follow up to NRC-ISG, Section 4.2.2.4 (Reference 59), the 2013 Catawba River Model includes a 5-percent spillway capacity reduction at [redacted] and a 5-percent spillway capacity reduction at [redacted] The two spillway capacity reductions are implemented to address the uncertainty of debris on the

(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)

respective reservoirs. Uncontrolled spillways at (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) are not subject to flow restriction from gate piers; therefore, debris can flow over these structures.

No significant reservoir storage reduction has been included in FERC-licensed reservoir capacity due to sedimentation. The Duke Energy Developments reservoir storage was verified during FERC relicensing for (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) reservoirs using bathymetry surveys (1996-2005) in support of the 2006 Hydro Relicense Application (Reference 11) with the exception of Oxford Development. Based on the relicensing bathymetric surveys at the six reservoirs and review of shoreline management conducted during FERC relicensing, there has been no significant storage reduction in Lake Hickory (Oxford Dam).

Develop Design-Basis Flood Scenarios: (fair-weather, seismic, flooding)

Fair-weather scenarios involve a piping failure at an upstream dam and allow the Catawba River Model simulation to determine if downstream Development dams achieve sufficient overtopping to warrant potential failure. If no downstream dams indicate potential for overtopping, then the fair-weather external flood is routed through each downstream reservoir using storage and spillway capacity similar to a precipitation flood event. Each upstream reservoir was tested for fair-weather failure impacts and the elevations at the Lake Wylie cross sections near the CNS were reviewed for comparison to the CNS Intake Dike and SNSWP Dam crest elevations to determine site impacts. Wave impacts are considered to determine if adequate freeboard is provided at the site under these flooding scenarios.

Fair-weather models were simulated using average annual median inflows to each reservoir based on daily hydrology developed during the FERC relicensing (Reference 11). Fair-weather dam failures of upstream dams were simulated using the Catawba River Model. Storage and spillway capacity at each dam is adequate to discharge the upstream dam breach flow without causing overtopping at the downstream dams (Reference 18) (SOF 2.2-02).

In similar fashion to the fair-weather scenarios, the combined effects scenarios involve a piping failure at an upstream dam triggered by a seismic event during a half PMF event (ANS 2.8 Section 9.2 (Reference 1)) and allow the Catawba River Model simulation to determine if downstream Development dams achieve sufficient overtopping to trigger potential failure during a half PMF event. All Catawba River Developments are able to store and discharge the FERC-required PMF; therefore, the half PMF event does not produce an overtopping at any dam.

Test model simulations were performed starting at the upstream dam of (b)(3);16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) during a half PMF event with a triggered piping dam failure at the peak half PMF reservoir elevation. Each dam located downstream of the assumed seismic-induced failure site was evaluated for possible cascading failure due to overtopping from the combined half PMF and upstream breach failure. For all half PMF plus dam failure combined effects run cases, no downstream dams experienced overtopping. Each run case included routing of the half PMF plus a seismic-induced failure where the PMF runoff plus breach discharge floodwater was routed through each of the seven modeled reservoirs. In addition, the combined effects external peak flood elevation at the Lake Wylie cross section near the CNS Intake Dike and the CNS-SNSWP Dam were reviewed for comparison to the CNS Intake Dike and SNSWP Dam crest elevations to determine the level of freeboard margin available prior to considering wind-driven wave site impact. Wave impacts were added to determine if adequate freeboard is provided at the CNS site.

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

The design-basis flood scenarios are developed consistent with NUREG/CR-7046 Section 3.3, 3.4, and 3.9 and Appendices D and H (Reference 60).

HDR used the National Inventory of Dams (NID) for the states of North Carolina and South Carolina to identify dams within the Catawba River system upstream of Wylie Dam that have potential to impact both the Duke Energy hydroelectric dams and the CNS site. Calculation CNS-194292-014 (Ref. 22) provides details about the NID search, which focused on reservoir

storage capacity and hazard classification as the key screening parameters for determining potential downstream impacts.

Experience with existing FERC Catawba-Wateree PMF models (Reference 12) was used in the HHA evaluation of flooding from upstream reservoirs through evaluation of the insignificant contribution of the [redacted] plus small dams in the Wylie drainage basin and construction of PMF model inputs for the [redacted] reservoirs and dams (Reference 22) (SOF 2.2-04). Modeling using [redacted] upstream basin PMFs was identified as the bounding reservoir flooding events through a series of model scenarios including the fair-weather and seismic failure plus half PMP model runs. The selection of the [redacted] using the HHA process is consistent with the existing licensing basis PMF analysis. [redacted] is the largest storage reservoir on the Catawba-Wateree at approximately [redacted] ac-ft of storage at normal maximum pool elevation [redacted] ft msl. The storage and discharge associated with the Cowans Ford Dam is consistent with the modeled capability to manage upper Catawba River flood events and regulate discharges to downstream reservoirs. Duke Energy's operation of all reservoirs in the Catawba River chain provides for a well-managed water system that is operated to support all uses of the reservoirs including hydropower, recreation, municipal water systems, and steam power generation at plants located along the river managed by Duke Energy.

(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)

(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)

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

Each of the Duke Energy Developments underwent a PMF evaluation in 1992 to re-analyze the hydrologic (PMP) and hydraulic (PMF) performance to maintain compliance with the FERC dam safety regulations. The Catawba River PMP values for the respective Duke Energy Developments are based on Hydrometeorological Reports 51/52 and uses elliptical-shaped isohyetal patterns to maximize the rainfall over a given Duke Energy Development's basin. The CNS-2.1 Fukushima Study employs the FERC-approved Catawba-Wateree Legacy HMR52-HEC-1 model (Reference 37) to reevaluate the CNS site for flooding from rivers and streams (external flood event).

2.2.1 Probable Maximum Flood – Lake Norman

As shown in Figure 2.2.1-1, the 2013 (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) is based on a 216-hour rainfall event comprised of:

- 40 percent HMR51 PMP (centered over the centroid of the Cowans Ford Dam drainage basin) (Reference 38)
- 72 hours of zero precipitation
- 72-hour full PMP

Forty (40) percent of the PMP was used versus the 500-yr rainfall based on comparison of NOAA Atlas 14 point precipitation rainfall 72-hour totals at each of the seven dam locations and the 40 percent values for each dam sub-basin. In several locations, the 500-yr precipitation total was approximately equal to the 40 percent PMP total (Reference 24).

Table 2.2.1-1 shows maximum reservoir elevation and discharge at each upstream dam for the hydraulic model simulation for the (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) shown in Figure 2.2.1-1. Figure 2.2.1-2 shows the corresponding Wylie reservoir stage hydrograph near the CNS site for the (b)(3) 16 U.S.C. § 824o-1 (d), (b)(4), (b)(7)(F) discharge routed through Wylie Dam. These modeling results do not assume a failure at any upstream dam.

FIGURE 2.2.1-1

SUB-BASIN INFLOW HYDROGRAPHS FOR (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) **PMF SCENARIO CF_ACS_PMF_1B2**

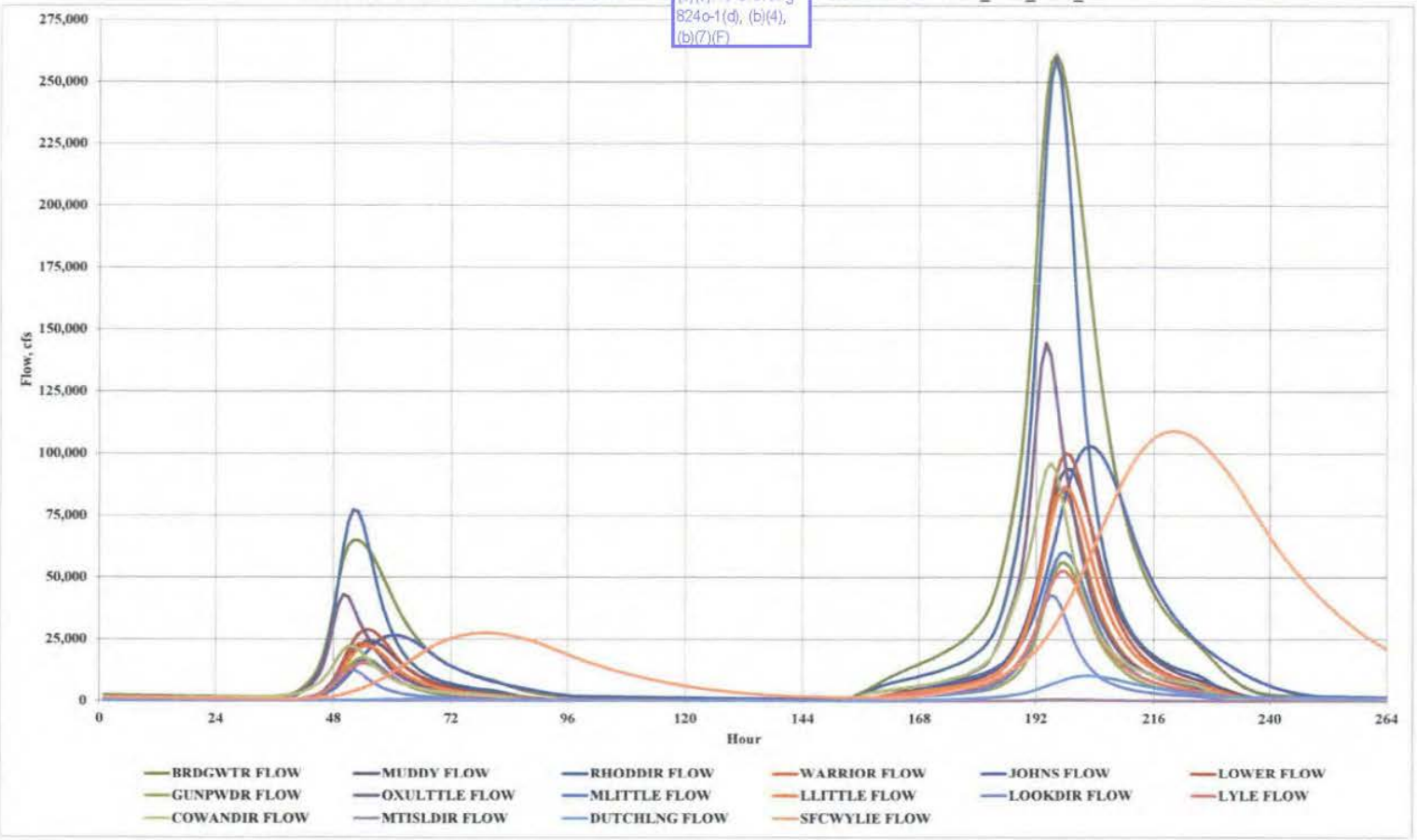


TABLE 2.2.1-1
CATAWBA RIVER MODEL RUN CF_ACS_PMF_1B2 RESULTS (REFERENCE 18)

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

FIGURE 2.2.1-2

WYLIE RESERVOIR STAGE NEAR CNS SITE (REFERENCE 18)

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

CF ACS PMF 1b2

2.2.2 Probable Maximum Flood – Lake Wylie

As shown in Figure 2.2.2-1, the 2013 Wylie Dam PMF is based on a 216 hour rainfall event comprised of:

- 40 percent HMR51 PMP (centered over the centroid of the Wylie Dam drainage basin) (Reference 39)
- 72 hours of zero precipitation
- 72-hour full PMP

Table 2.2.2-1 shows maximum reservoir elevation and discharge at each upstream dam for the hydraulic model simulation for the Wylie Dam PMF for model run Wylie_PMF_2A with no assumed dam failures. Figure 2.2.2-1 shows the corresponding Wylie reservoir stage hydrograph with peak elevation (approximately 590.7 ft msl) less than the CNS Yard elevation. These modeling results do not assume a failure at any upstream dam. In this model scenario, there is approximately 0.8 ft of reservoir elevation difference between the peak elevation at Wylie Dam and the upstream cross section results shown in Figure 2.2.2-2.

TABLE 2.2.2-1
CATAWBA RIVER MODEL RUN RESULTS FOR WYLIE PMF 2A SCENARIO (REFERENCE 18)

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

FIGURE 2.2.2-1
SUB-BASIN INFLOW HYDROGRAPHS FOR WYLIE PMF SCENARIO WYLIE_PMF_2A

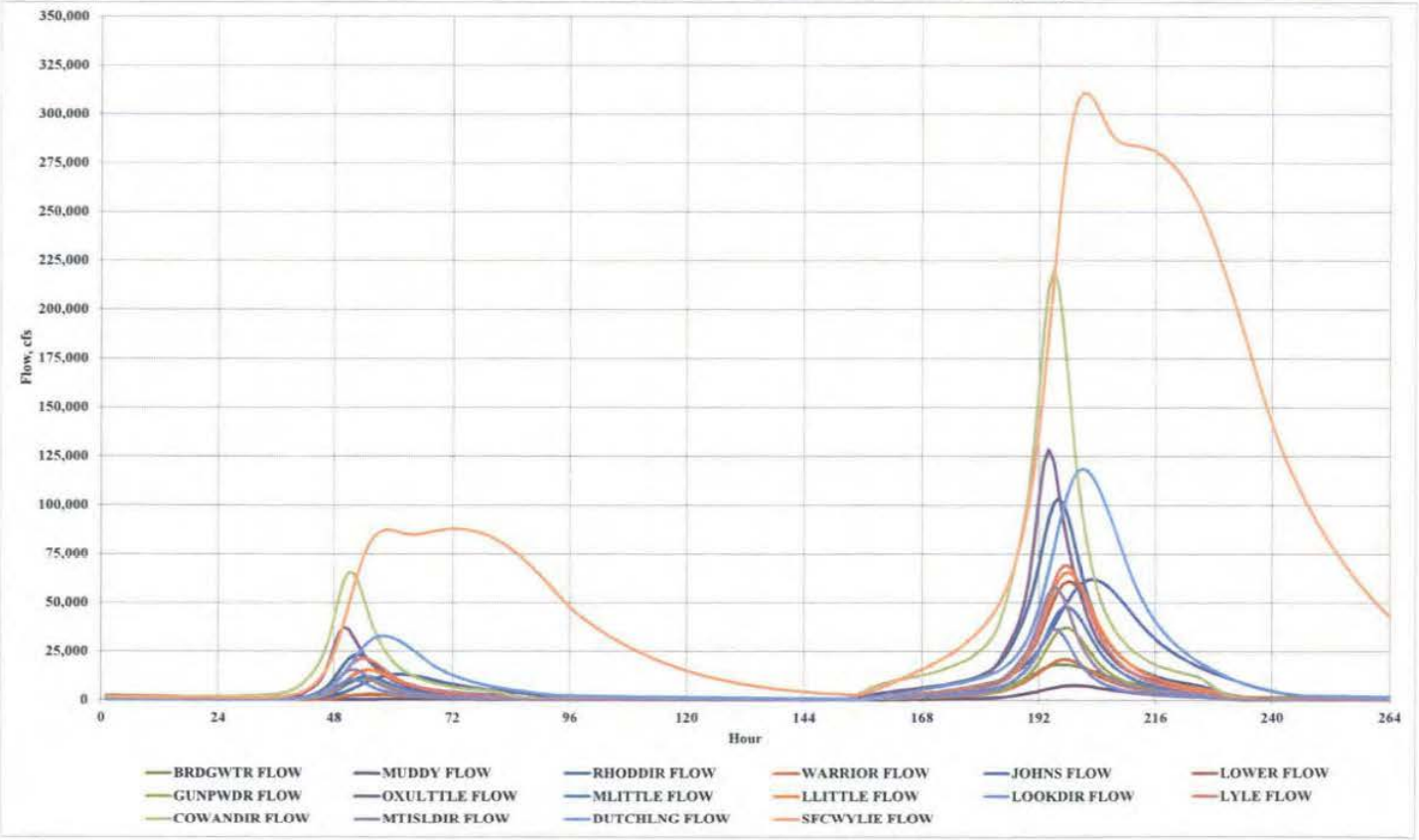


FIGURE 2.2.2-2
WYLIE RESERVOIR STAGE NEAR CNS SITE (REFERENCE 18)

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

— Wylie PMF 2a

2.3 Dam Failures

2.3.1 Potential Dam Failure

Per ANS 2.8, Section 5.5.4, "if no overtopping is demonstrated, the evaluation may be terminated and the embankment may be declared safe from hydrologic failure" (Reference 1). Overtopping should be investigated for either of these two conditions:

- PMF surcharge level plus maximum (1 percent) average height resulting from sustained 2-year wind speed applied in the critical direction; or
- Normal operating level plus maximum (1 percent) wave height based on the probable maximum gradient wind.

Consistent with NUREG/CR-7046 and considering the detailed hydrologic and hydraulic analyses performed for this reevaluation, a hierarchical hazard assessment (HHA) was performed on the Catawba River basin upstream of Wylie Dam to determine if other non-Duke Energy hydroelectric dams have the potential to impact both the Duke Energy hydroelectric dams and the CNS site. HDR utilized the National Inventory of Dams (NID) to develop a list of dams in the Catawba River basin with focus on reservoir storage capacity and hazard classification as the key parameters of interest. For the screening analysis, HDR removed the inconsequential dams (Low Hazard and Not Classified dams) from consideration per the ISG, Section 3 dam failure guideline (Reference 59). The remaining small dams (non-critical dams) were evaluated for additional failure flood impact based on location and calculated cumulative volumes upstream of each Duke Energy Development along the Catawba River from Lake James to Lake Wylie. These small dam volumes were used to estimate the maximum possible increases in reservoir water surface elevations for normal operating reservoir levels and at FERC-approved Cowans Ford PMF reservoir levels assuming the entire volume was instantaneously added to each reservoir (Reference 22). The small relative increases in elevation projected using this simple screening analysis do not produce a flood hazard to the Duke Energy Catawba River dams or the CNS site. Taking into account the variable timing of potential individual failures, the flood-wave travel times and attenuation and failure of the small tributary dams can be removed from

consideration in determining the flood hazard reevaluation design-basis flood at CNS (Reference 22).

Breach Parameters:

Dam failures are typically identified as overtopping or piping failure modes and can occur at either concrete gravity or earthen embankment sections of dams. In addition, the type of failure mode (overtopping versus piping) also has a bearing on the hydraulic performance of the affected structure. Piping failures are generally limited to sunny day or fair-weather dam failures while overtopping failures are attributed with a hydrological event or upstream cascading dam failures. Seismic-induced dam failures could use piping failure mode breach parameters to simulate the failure of a dam structure.

Overtopping and piping failure breach parameters were developed for the Duke Energy Developments. The Duke Energy reservoirs on the Catawba River are regulated by the FERC and are maintained to standards required by 18 CFR Subpart 12.

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

Regression equations are available that utilize dam features and reservoir storage volume to estimate dam breach parameters. The dam breach parameters of interest typically used to determine the structures hydraulic performance include:

- Bottom breach elevation,
- Breach side slopes,
- Bottom breach width,
- Average breach width, and

- Reservoir storage volume.

The regression methodologies chosen to support the development of breach parameters in analyzing the downstream impacts of Duke Energy Catawba River dam failures include:

- Froehlich,
- Walder and O'Connor
- MacDonald-Langridge Monopolis, and
- Wahl.

The regression methodologies above are consistent with Section 7 of the ISG document (Reference 59). The breach parameter development details and results are presented in Calculation CNS-193049-016 (Reference 66). Geometric-based breach parameters (bottom and average breach width, bottom breach elevation, side slopes) that are developed from regression methodologies are compared with the Duke Energy Developments' physical site geometry (prepared valley abutments and slopes, bottom valley width and elevation, and documented rock and PWR layers) to determine potential site constraints that limit final breach development.

Dam breach parameters are used in the HEC-RAS model to describe the shape, size, and progression rate of the breach. Developed dam breach parameters are inserted in the Catawba River Model through HEC-RAS Geometry file, Inline or Lateral Structure, Breach Plan Data module. The required dam breach parameters include:

- Final bottom breach width (ft)
- Final bottom breach elevation (ft msl)
- Left and right side slopes of the breach (horizontal slope component)
- Full formation time (hours – defined as failure time)
- Failure mode (overtopping or piping)
    - For piping failure, define elevation at which piping occurs within the embankment.
    - For overtopping failure, define the trigger for failure which is typically a reservoir elevation at which the breach is initiated.

- Breach progression (1:1 scale of time fraction versus breach development fraction)
  - There are two progressions, linear (HEC-RAS default) and sine wave.
  - Sine wave progression is typically chosen to reflect the physical erosion process of breach development from initially slow to rapid growth and then slower growth near the end (earth embankments).

TABLE 2.3.1-1
FINAL DAM BREACH PARAMETERS FOR PIPING AND OVERTOPPING MODE FAILURES AT DUKE ENERGY HYDROELECTRIC DAMS (REFERENCE 18)

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

2.3.2 Dam Failure Permutations

The Catawba River Model unsteady flow dam failure simulations are based on 1-hour time increments for inflow hydrographs, computation intervals of 1 minute, hydrograph output intervals of 1 minute, with total simulation time of 11 days.

The supporting calculation CNS-194292-010 Rev 0 (Reference 18) includes modeling details for sensitivity model runs developed to test dam failure outcomes at the CNS Site relative to changes in Manning's *n*-values, number of upstream dams involved in cascading failures breach size, and failure time. The final scenario matrix developed to support the determination of the Fukushima 2.1 flood hazard reevaluation for the CNS Site is shown in Table 2.3.2-1.

The Catawba River Model Combined Effects half PMF plus dam failure event results for the Fukushima 2.1 flood hazards reevaluation (without wind-driven wave impacts) are presented in Tables 2.3.2-2 and 2.3.2-3. Table 2.3.2-2 results are based on the

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

TABLE 2.3.2-1
HEC-RAS MODEL FINAL RUN MATRIX (REFERENCE 18)

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

TABLE 2.3.2-2
HALF PMF PLUS (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) MODEL RESULTS
INCLUDING RESERVOIR STAGE NEAR CNS SITE (REFERENCE 18)

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

TABLE 2.3.2-3
HALF PMF PLUS WYLIE DAM CATAWBA RIVER FAILURE MODEL RESULTS INCLUDING RESERVOIR STAGE NEAR CNS SITE (REFERENCE 18)

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

The Catawba River Model Combined Effects Precipitation Flood event plus dam failure (CNS site flood bounding event) results for the Fukushima 2.1 flood hazards reevaluation (without wind-driven wave impacts) are presented in Table 2.3.2-4 and Figures 2.3.2-1, -2, and -3 showing hourly stage hydrographs at three CNS Site boundary locations and Figure 2.3.2-4 showing a Catawba River flood stage profile. Table 2.3.2-4 results are based on the

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

TABLE 2.3.2-4

PMF PLUS (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) **MODEL RESULTS INCLUDING RESERVOIR STAGE NEAR CNS SITE (REFERENCE 18)**

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

FIGURE 2.3.2-1
WYLIE RESERVOIR STAGE NEAR CNS SITE INTAKE STRUCTURE (REFERENCE 18)
LAKE WYLIE WATER LEVEL ABOVE [redacted] FT MSL FOR [redacted] HOURS

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

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

CF ACS PMF 1b4 — — — CF ACS PMF 8j4

FIGURE 2.3.2-2
WYLIE RESERVOIR STAGE NEAR CNS SITE SNSWP DAM (REFERENCE 18)
LAKE WYLIE WATER LEVEL ABOVE [redacted] FT MSL FOR [redacted] HOURS

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

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

CF ACS PMF 1b4 - - - CF ACS PMF 8j4

FIGURE 2.3.2-3
WYLIE RESERVOIR STAGE NEAR CNS SITE COMPARISON BETWEEN UPSTREAM CROSS SECTION NEAR INTAKE AND ALLISON CREEK DISCHARGE STRUCTURE (REFERENCE 18)

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

- - - CF ACS PMF 8j4 - CNS Yard Intake
..... CF ACS PMF 8j4 - CNS Yard Discharge

FIGURE 2.3.2-4

CATAWBA RIVER WATER SURFACE PROFILE COMPARISON BETWEEN NORMAL POOL (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) **(REFERENCE 18)**

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

2.4 Storm Surge and Seiche

The CNS site is located on an inland reservoir over 150 miles from the Atlantic coastline of North and South Carolina and is not subjected to storm surge or seiche flooding communicated from ocean wave-driven effects. The spatial scale of a strong storm system that would significantly drop atmospheric pressure would typically be very large compared to the size of Lake Wylie or other reservoirs in the Catawba River Basin, so the pressure differential across the lake would not be large enough to result in significant water surface variations. Standard guidance for flooding analysis in reservoirs, such as USACE Engineering Manual 1110-2-1420 "Hydrologic Engineering Requirements for Reservoirs" (1997), do not typically recommend consideration of water level increases caused by atmospheric pressure gradients; consideration of water level is typically limited to wave analysis (through forcing by sustained winds) and water set-up (again from sustained winds). Standard guidance does not state a need to assess atmospheric pressure extrema on the potential that low pressure would have to raise water levels. Because the influence of atmospheric pressure gradients on water levels is negligible, such analysis is not needed for determining freeboard requirements. In addition, storm surge and seiche flooding have been reviewed in the FERC-required evaluation of the Catawba River hydropower developments and are not considered credible events to produce maximum water levels near the sites. A seiche caused by landslide is not considered credible based on the topography and geology around the reservoirs. However, storm surge and seiche wave impacts were evaluated for maximum hurricane wind-driven wave formation using a similar analysis to guidance provided in the NRC NUREG/CR 7046 Appendix F (Reference 60).

2.4.1 Seiching Analysis

The seiching calculations were performed assuming a seiche with one mode of oscillation in a rectangular-shaped basin of constant depth. Increasing modes of oscillation are less common and less threatening because the energy in these modes is dampened more rapidly (Dean and Dalrymple, 1984) (Reference 2). The length of the representative rectangular basin was chosen using engineering experience to consider the shape of the reservoir and potential seiches that may occur.

Oscillation Period: The oscillation period of the seiche was determined using the following equation from Dean and Dalrymple (1984, Reference 2):

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

where L is the length of the basin, g is the acceleration due to gravity, and h is the depth of the basin.

Seiching Amplitude: The amplitude of the seiche was determined using the following formula from the U. S. Bureau of Reclamation (1981, Reference 55) for calculating wind setup:

$$S = \frac{U^2 F}{1400D}$$

where U is the wind velocity in mph, F is the fetch length in miles, and D is the average water depth in feet. The period of the seiche was used as the duration of the wind speed in the calculation of the wind setup. This wind duration is considered conservative because it would take longer for the wind setup to develop which would decrease the wind speed for the fetch calculation. The wind speeds were converted to the duration of the seiche using the following methodology from the Shore Protection Manual (USACE 1984) (Reference 51):

$$\frac{U_t}{U_{3,600}} = 1.277 + 0.296 \tanh(0.9 \log_{10} \frac{45}{t})$$

where U_t is the wind speed at the duration t, $U_{3,600}$ is the one hour wind speed, and t is the duration of interest.

Table 2.4.1-1 provides a summary of the seiche analysis results from CNS-194292-017 Rev 1 (Reference 63).

TABLE 2.4.1-1
SEICHING WIND-DRIVEN WAVE RESULTS

Site Location	Fetch Length (ft)	Condition	Avg. Water Depth (ft)	Seiche Period, T (min)	Wind Speed for Wind Setup Calculation U_t (mph)	Wind Setup S (ft)	Maximum Water Surface Elevation (ft, msl)
SNSWP Dam	13,730	Normal	40	13	96.8	0.44	569.84
	13,730	CF ACS PMF 8j4	73	10	35.1	0.03	602.23
CNS Intake Area-1	20,270	Normal	40	19	95.6	0.63	570.03
	20,270	CF ACS PMF 8j4	73	14	34.6	0.04	602.24
CNS Intake Area 2	4,140	Normal	40	4	103.2	0.15	569.55
	4,140	CF ACS PMF 8j4	73	3	37.8	0.01	602.21

(SOF 2.4.1-01) CNS-194292-017 Rev. 1

The maximum water surface values shown in Table 2.4.1-1 are not bounding. Bounding maximum water surface elevations for the CNS site considering combined effects is produced by wind-driven waves. Report Section 2.8 summarizes the bounding wind-driven wave heights combined with normal and maximum flood inundation reservoir levels.

2.4.2 Wind-Driven Waves Analysis – Lake Wylie Source

Wind-driven wave heights were developed using the Coastal Engineering Design and Analysis System (CEDAS) Automated Coastal Engineering Software (ACES) at four locations around the CNS site based on proximity to water bodies and topography. These locations are similar to site areas previously studied and reported in the CNS UFSAR. The four locations are:

1. Lake Wylie face of the SNSW Service Pond Dam
2. CNS Pump House
3. CNS Yard/Intake
4. CNS Discharge Structure

The analyses were developed using water surface elevations for Lake Wylie presented in Table 2.4.2-1. There was a slightly lower peak water surface elevation at the (b)(3) 16 U.S.C. § 824n-1(d), (b)(4), (b)(7)(F) ft msl) from the HEC-RAS model results; but for this reevaluation, a single peak value of ft msl was used.

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

TABLE 2.4.2-1
WATER SURFACE ELEVATIONS USED IN CALCULATIONS

Location	Normal (ft, msl)	CF ACS PMF 8j4 (ft, msl)
1. Wylie-SNSW Service Pond Dam	(b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)	
2. CNS Pump House	569.4	
3. CNS Yard	569.4	
4. CNS Discharge Structure	569.4	599.8

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

(SOF 2.4.2-01) CNS-194292-013-01 Rev 2

Wind Speed for the Normal Reservoir Condition: The 116-mph wind speed associated with the hurricane analysis in the UFSAR Section 2.4.5.1 was used for the wave analysis during normal reservoir elevation. This wind speed was the licensing basis of the riprap design for the SNSWP Dam per Duke Energy calculation CNC-1150.02-00-0002, Rev 02 (Reference 4). The duration of this wind speed was assumed to be a 1-minute average in accordance with the U.S. Weather Service methodology for reporting hurricane wind speeds (SOF 2.4.2-02) CNS-194292-013-01 Rev 2 (Reference 19).

Wind Speed for PMF Condition: A 40-mph overland wind speed was used for the PMF wave analysis. This was consistent with the wind speed assumed in licensing basis as reported in UFSAR Section 2.4.3.6 (SOF 2.4.2-03) (CNS-194292-013-01 Rev 1). The American National Standard ANSI/ANS-2.8-1992 shows the 2-yr wind speed at this location to be between 40 mph and 50 mph, although no indication of further precision is provided. An analysis performed on a 50-year wind record from Charlotte Douglas International Airport (Reference 43) yielded a 2-yr fastest mile, 10 m elevation wind speed of 32.9 mph (Reference 19: CNS-194292-013-01 Rev 2). The site-specific frequency distribution was developed using the Fisher-Tippett Type I distribution which was developed by Gumbel (1958) (Reference 14). This distribution is used by

the National Weather Service (NWS) to derive frequency distribution for multiple meteorological parameters. The 40-mph fastest mile wind speed was selected for the Fukushima 2.1 reevaluation considering the data record and the wind speed provided by ANSI/ANS-2.8-1992 with a reduction factor in accordance with the Bureau of Land Reclamation (1981, Reference 55).

Fetch: The fetch lengths were developed using currently available aerial photography and GIS tools. Note: The CNS Yard location has a significantly longer fetch associated with the flood condition because the higher water elevation submerges low-lying areas of land.

Average Water Depth: The average water depth for the fetch associated with each location was approximated using cut-lines through Lake Wylie developed from GIS digital terrain files

Results for the wind-driven wave analysis for Lake Wylie source are provided in Table 2.4.2-2 (Reference 18). The reevaluation of the CNS site for potential flooding impacts from wind-driven waves was based on a bounding site flooding event that potentially inundates the CNS Yard by approximately 8 ft. At this level of inundation, there would be no additional wave shoreline run-up. Combined effects flooding including wind-driven water levels influencing CNS Yard inundation is discussed in Section 2.8.

2.4.3 Wind-Driven Waves Analysis – SNSWP Source

A separate analysis was performed to review the impact from wind-driven wave heights for the SNSWP under PMP flooding conditions without impacts from flooding on Lake Wylie. The analysis was based on methodology outlined in "Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams", ACET Technical Memorandum No. 2. United States Bureau of Reclamation, December 1981 (Reference 55). Wind speed used in this analysis was based on ANS 2.8 Section 9.2, Figure 1. An unmodified wind speed of 50 mph was assumed for PMF pond level analysis and is more conservative than the wind speed used in the analysis outlined in Section 2.4.2. A hurricane wind of 116 mph was used to evaluate available freeboard based on combined effects guidelines defined in ANS 2.8 Section 9.2.2. Due to the

short fetch distances the wind-driven waves were small and the analysis showed adequate freeboard for normal reservoir level plus hurricane wind and PMF plus 2-year winds, therefore, no additional refinement of the wind speeds used for this structure was performed (Reference 64). Table 2.4.3-1 provides a summary of inputs and outputs. Wave run-up plus setup were compared to available freeboard for normal and LIP Flood Pool elevations by simple addition of the peak static pool elevations and the wave effects heights based on no overtopping of the SNSWP Dam. Adequate freeboard is available during both events.

Licensing design basis for wind-driven riprap sizing and freeboard (Reference 4) assumed a hurricane wind of 116 mph. Section 2.8 discusses the adequacy of the riprap based on overtopping due to combined effects flooding.

TABLE 2.4.2-2
SUMMARY OF WIND-DRIVEN WAVE ANALYSIS – LAKE WYLIE WATER LEVELS

Location	Condition	Fetch (ft)	Avg. Water Depth (ft)	Wind Duration for Wave Generation (min)	Wind Speed for Wave Generation (mph)	Wave Celerity (ft/s)	Time for Wave to Travel Fetch (min)	ACES Output	
								Spectral Significant Wave Height, Hmo (ft)	Peak Wave Period, Tp (s)
1. Wylie-SNSW Service Pond Dam	Normal	6,330	40	5	102*	15	7	4.3	3.0
	CF ACS PMF 8j4	6,330	73	10	35*	10	11	1.4	2.0
2. CNS Pump House	Normal	6,330	40	5	102*	15	7	4.3	3.0
	CF ACS PMF 8j4	6,330	73	10	35*	10	11	1.4	2.0
3. CNS Yard	Normal	2,130	30	3	105*	11	3	2.7	2.1
	CF ACS PMF 8j4	20,330	63	25	34*	15	23	2.4	2.9
4. CNS Discharge Structure	Normal	3,010	35	4	103*	12	4	3.1	2.4
	CF ACS PMF 8j4	3,010	65	5	36*	8	6	1.0	1.6

(SOF 2.4.2-04) CNS-194292-013-01 Rev 2
Note: * The wind speeds shown in Table 2.4.2-2 are adjusted to the duration used in the analysis as described in Reference 19).

TABLE 2.4.3-1
SUMMARY OF WIND-DRIVEN WAVE ANALYSIS – SNSWP DAM UPSTREAM SLOPE

Location	Condition	Fetch (ft)	Avg. Water Depth (ft)	Wind Speed for Wave Generation (mph)	Wave Height (ft)	Wave Run-up + Setup (ft)	Peak Elevation (ft)	Freeboard (ft)
CNS-SNSW Pond Dam	Normal	984	20	52	1.28	1.05	(b)(3) 16 U.S.C. § 824o-1 (d), (b)(4), (b)(7)(F)	
	LIP Flood	1168	29	42	0.89	0.79	583.79	11.21
	Hurricane	984	20	116	2.20	1.91	575.91	19.09

(SOF 2.4.3-01) CNS-194292-021 Rev 0

2.5 Tsunami

Since the CNS site is not located on an open ocean coast or large body of water, tsunami-induced flooding will not produce the maximum water level at the site.

2.6 Ice-Induced Flooding

Since the CNS site is not located in an area of the U.S. subjected to periods of extreme cold weather that have been reported to produce surface water ice formations, ice-induced flooding will not produce a credible maximum water level at the site and is not considered a realistic external flooding hazard to CNS.

2.6.1 Ice Effects

Long-term air temperature records (1951 to 2011) available at the South Carolina State Climatology Office were reviewed to assess historical extreme air temperature variations at the CNS site. The analysis was also supported by onsite temperature data measured at the CNS site.

The climate at the CNS site is characterized by short, mild winters and long, humid summers. Local climatology data for Winthrop College near Rock Hill, South Carolina, for a period of December 1899 through March 1012 show an average annual minimum air temperature of 50.7° Fahrenheit (Reference 49: http://www.sercc.com/cgi-bin/sercc/cliMAIN.pl?sc9350).

There has not been a recorded event of significant surface ice formation on Lake Wylie or any of the 11 FERC-regulated Catawba River Developments FERC #2232 in the last 100 years.

2.6.2 Ice Jam Events

There are no recorded ice jam events in the upper reach of the Catawba River based on a search of the USACE's *Ice Jam Database* (SOF 2.6.2-01). Water temperatures in this area of the southeast U.S. consistently remain above freezing (Reference 50).

2.7 Channel Diversions

Due to the location of CNS on the banks of Lake Wylie and the upstream topography of the reservoir, channel diversion is not considered a credible flooding event. The Catawba River is highly regulated by a series of dams. Reservoirs are back-to-back and backwater effects of each dam mitigate reservoir velocities that would be necessary to produce channel diversion. Over a period of hundred plus years of regulation, the section of the Catawba River where CNS is located has not exhibited any tendency to meander toward or migrate away from the CNS site.

2.8 Combined Effects

Section 9 of ANS 2.8 outlines general criteria to be reviewed for addressing combined flood-causing events (Reference 1). As discussed in Sections 2.2 and 2.3, the evaluation of precipitation events was performed for inflows up to the PMF. Due to the magnitude of the storage and discharge capacity at the [redacted] FERC dam sites upstream of CNS, rainfall events less than the PMF did not produce the bounding flooding levels at Lake Wylie Dam and CNS. ANS 2.8 Section 9.2.1.1 provides three alternatives with combinations for precipitation events to be evaluated. Alternative I was fully developed for the flood hazard reevaluation. Alternatives II and III are not applicable to the Catawba River basin based on the climate and topography. Snowpack is not a meteorological event that occurs in the Piedmont Region of North Carolina. PMP is produced by hurricane events during July to October. ANS 2.8 Section 9.2.1.2 provides two alternatives to review seismic dam failures with precipitation events. As discussed in Section 2.3, Alternative II was identified as the applicable case for further evaluation since the potential breach volume in the upstream reservoirs during a half PMF would be larger than the 25-year flood.

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

Lake Wylie is a man-made impoundment located in the Piedmont Region of North Carolina protected from coastal events as well as extreme cold weather events. Lake Wylie is considered an "enclosed body of water" as defined in ANS 2.8 Section 7.3.3 for consideration of storm surge combined effects flooding. Based on this definition and guidelines noted in ANS 2.8 Section 9.2.3.2 for the streamside location of CNS, Alternative II was determined to be the most limiting case and was used for reviewing the possible combinations producing maximum flood

levels. Alternative II includes the consideration of a 25-year-return-period surge or seiche. The Catawba River reservoirs are not known to be subject to surges or seiches, and there was no available source for validating a return period for this analysis. As discussed in Section 2.4, storm surge- and seiche-generated waves were evaluated for Lake Wylie and found to produce a lower wave potential on Lake Norman than a simple wind-driven wave.

Combined effects for seismic dam failures, ANS 2.8, 9.2.1.2 and surge and seiche, ANS 2.8, 9.2.2 were considered but are bounded by ANS 9.2.1.1 based on 1-D Catawba River Model trial simulations. Upstream FERC-regulated Catawba River dam sites are all designed for PMF flooding, seismic loadings specified by FERC requirements, and have adequate spillway discharge capacity for flood events up to the PMF (Section 2.2). Surge and Seiche phenomena do not produce significant flooding at any of the reservoir sites based on available freeboard and physical limitations of topography and location discussed in Section 2.4. HDR calculations CNS-194292-010 Rev 0 (Reference 18) and CNS-194292-011 Rev 0 (Reference 17) provide additional information on the development of the combined effects flood.

Upstream dam failure as defined above in Section 2.3.1 was also reviewed.

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

This conclusion was reached through experience with previous FERC emergency action plan model dam breach simulations in addition to the series of Catawba River Model (HEC-RAS) runs performed for this reevaluation (Reference 18). No additional upstream dam failure was found through modeling releases of volumes of water that exceed the bounding ANS 9.2.1.1-event outlined above.

External flooding combined effects that were found conservative but realistic were the result of

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

2.8.1 CNS Yard Combined Effects

ANS 2.8 Section 9.2.1.1, Alternative I was used for evaluation of precipitation flood combined effects as described in report Section 2.2. This included the combination of mean monthly inflow to each upstream reservoir, median soil moisture conditions, antecedent rainfall event of 40 percent of the PMP over 72 hours followed by 72 hours of no rainfall, the full 72-hour PMP and 2-year wind speed applied in the critical direction. This evaluation was performed for the critical combination of the PMP applied over the

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

Peak Wylie reservoir elevations near the CNS site were determined using the Catawba River Model (HEC-RAS 1-D unsteady breach simulation). Water surface elevation hydrographs at cross sections located near the SNSWP Dam and CNS Intake structure along with the CNS Discharge structure were used in the ICM 2-D model for the CNS site (Section 2.1). Combined Effects wind-driven wave run-up was applied to the upstream 2-D Boundary Condition since it is associated with the longest fetch length and higher base flooding elevation, resulting in a more conservative combined effects assessment. HDR did not evaluate wind-driven wave run-up to the Allison Creek cove, CNS Discharge, due to the simultaneous opposite wind direction required to produce waves toward the CNS site.

The 2-D Zone boundary condition used in modeling the LIP described in Section 2.1 was modified along the SNSWP Dam and CNS Intake by inserting a Lake Wylie stage hydrograph from the (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam failure HEC-RAS model Run CF ACS PMF 8j4 described in Section 2.3. A second boundary condition was modified along Allison Creek Cove using a second downstream Lake Wylie stage hydrograph from HEC-RAS Model Run CF ACS PMF 8j4 (Reference 18). A modeling period of 24 hours was selected for review of the Catawba River Model output for cross sections near the CNS site. The length of the modeling period was selected to minimize the 2-D model run time while allowing adequate time for the complete routing cycle of inundation on the CNS site. Figure 2.8.1-1 shows the upstream boundary

condition stage hydrograph from the Catawba River Model using Plan CF ACS PMF 8j4 (SOF 2.8.1-02).

The 24-hour period was selected to capture the peak Combined Effects base flood (i.e., no wind-driven wave run-up contribution) peak stage, approximately [redacted] ft msl, as well as the rising and falling limbs of the hydrograph. During the model simulation, ICM calculates the overtopping flows (broad crested weir relationship) resulting from the stage hydrograph.

(b)(3) 16 U.S.C § 824o-1(d), (b)

Figure 2.8.1-2 provides the location of the 2-D boundary condition inputs along the 2-D Zone boundary.

FIGURE 2.8.1-1

STAGE HYDROGRAPH – HEC-RAS MODEL RUN CF ACS PMF 8J4 (REFERENCE 18)

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

FIGURE 2.8.1-2
2-D BOUNDARY CONDITION LOCATION (REFERENCE 17)

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

The 2-D boundary condition stage hydrographs and model network were used to create the 24-hour simulation to determine the base flood (i.e., no wind-driven wave run-up contributions) of Combined Effects scenario within the CNS Yard and SNSWP. The additional flooding contribution from wind-driven wave run-up was determined using a wave numerical model, (COULWAVE) (Reference 41). In order to evaluate the inundation potential due to the waves a methodology consistent with the Federal Emergency Management Agency's (FEMA) flood mapping (FEMA 2003) (Reference 68) was adopted to determine the maximum water surface elevation above the still water elevation that would be reached as result of the incident waves. COULWAVE was used to calculate the transformation of open-water waves from Lake Wylie propagating over the upstream model boundary (SNSWP Dam and CNS Intake structure) (Reference 17).

The model results, nodes were generated at an offset distance surrounding the perimeter of the main complex, near the Cask Storage area on the northern portion of the CNS Yard, and in the SNSWP to define locations of interest. Twenty-one (21) result nodes defining CNS Yard areas of interest are presented in Figure 2.8.1-3 and Table 2.8.1-1. Figure 2.8.1-4 shows the location of twenty-two (22) safety building doors related to SSCs result nodes and Table 2.8.1-2 provides the peak depth, elevation, and maximum velocity modeled at the nodes during the 24-hour simulation. The maximum ICM Combined Effects model inundation elevations with simulated wave impacts in the CNS Yard area are shown in Figure 2.8.1-5. Areas outside the CNS Yard area have been clipped from this figure as they are at higher ground elevations than covered by the color scale used in the figure and do not impact SSCs. Water surface elevation and depth hydrographs as well as velocity hydrographs at each node location for the Combined Effects simulation are provided in CNS-194292-011, Rev0 (Reference 17).

FIGURE 2.8.1-3
CNS ICM MODEL PMP RESULT NODE LOCATIONS

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

FIGURE 2.8.1-4

CNS ICM MODEL PMP RESULT NODE LOCATIONS AT DOORS TO BUILDINGS WITH IDENTIFIED SSCs

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

TABLE 2.8.1-1
COMBINED EFFECTS 2-D MODEL INUNDATION RESULTS AT NODES IDENTIFIED IN FIGURE 2.8.1-3

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

TABLE 2.8.1-2

COMBINED EFFECTS 2-D MODEL INUNDATION RESULTS AT DOORWAY NODES IDENTIFIED IN FIGURE 2.8.1-4

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

FIGURE 2.8.1-5
MAXIMUM WATER LEVELS IN CNS YARD DURING THE COMBINED EFFECTS PMF FLOOD EVENT (REFERENCE 17)

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

Combined Effects for seismic dam failures, ANS 9.2.1.2 and 9.2.2 were considered but are bounded by ANS 9.2.1.1 based on 1-D Catawba River Model trial simulations. Section 2.3 discussed the series of HEC-RAS dam failure model runs that were used to simulate combinations of fair-weather, half PMF plus seismic, and PMF plus failure impacts at CNS. Modeling results indicate that with the exception of the (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) plus dam failure, no other combined effects cases produced reservoir levels on Lake Wylie that exceeded (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft msl. The maximum Lake Wylie reservoir elevations near the CNS site due to (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) induced by a seismic event occurring at the peak of the half PMF event for a storm centered over the (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) were approximately equal at [redacted] ft msl. These two combined effects evaluation cases resulted in approximately [redacted] ft of freeboard margin at the CNS site and is significantly lower than the bounding CNS Yard flood case Lake Wylie elevation of [redacted] ft msl. Therefore, no other combined hydrologic flood scenario was found to produce water levels (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) HDR calculations CNS-194292-010 Rev 0 (Reference 18) and CNS-194292-011 Rev 0 (Reference 17) provide additional information on the development of the Combined Effects flood.

Upstream FERC-regulated Catawba River dam sites are all designed for PMF flooding and seismic loadings specified by FERC requirements and have adequate spillway discharge capacity for flood events up to the PMF (Section 2.2). Surge and Seiche phenomena are not expected to control at any of the reservoir sites based on available freeboard and physical limitations of topography and location discussed in Section 2.4.

Upstream dam failure as defined above in Section 2.3.1 was also reviewed. (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) This conclusion was reached through experience with (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) in addition to the series of Catawba River Model (HEC-RAS) runs performed for this reevaluation (SOF 2.8.1-03). No additional upstream dam failure was found through modeling to releases a volume of water that exceeds the bounding ANS 9.2.1.1 event outlined above.

2.8.2 SNSWP Dam Combined Effects

Combined effects for precipitation floods were considered for the SNSWP Dam. There is significant freeboard provided at the SNSWP Dam for combined effects resulting from upstream precipitation floods. There are no upstream reservoirs that contribute to flooding. The SNSWP Dam is designed for seismic conditions; and if a slope failure occurred, the release of water through a potential breach would flow into Lake Wylie and not on the site.

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

The reevaluation included a 2-D model simulation of the

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

The SNSWP Dam was also reviewed for rapid drawdown from the reevaluated bounding flooding event elevation of ft msl (Reference 65). Rapid drawdown slope factors of safety exceed current standard of practice (SOF 2.8.2-02). Table 2.8.1-1 shows the duration of the combined effects flooding wave is approximately 16.7 hours.

(b)(3) 16 U.S.C. § 824o-1(d), (b)

2.8.3 Cooling Towers Combined Effects

The NRC Standard Review Plan Section 2.4.4 requires the review of any onsite water control or storage structures that exist above site grade be evaluated for potential flooding of SSC important to safety. The CNS site includes six cooling towers at the south east end of the site adjacent to Allison Creek. These six towers are above grade. To comply with the Standard Review Plan requirement, modeling of a breach of one of the cooling towers was performed using the ICM CNS combined effects model. The modeled event was based on the assumption of a full cooling tower reservoir. A maximum water volume associated with Cooling Tower 2A of 119,210.26 ft^3 (approximately 2.74 acre-feet) is documented in Duke Energy calculation CNC-1115.00-00-0001, Rev 11 CNS Cooling Towers: Basin, Cable Tray Supports, Gates & Screens Amertap Ball Screens dated May 2013 (Reference 3). A failure of Cooling Tower 2A was simulated in the ICM 2-D model to evaluate flooding on the site since the cooling towers are at a higher elevation than the CNS Yard. A sensitivity analysis was performed on the assumed breach width of the concrete structure. This analysis assumed three breach widths, approximately 92 ft, 184 ft, and 263 ft. These three breach widths correspond to approximately 11 percent, 22 percent, and 31 percent of the total circumference of Cooling Tower 2A per Duke Energy drawing 73-42352, Rev D Schematic Views Class 700 Round Mech. Draft Tower. Each of the breaches was assumed on the perimeter of Cooling Tower 2A in order to maximize the potential flooding toward the CNS Yard and is assumed to fully form the breach instantaneously at the beginning of the simulation. This sensitivity analysis indicated that the modeled peak water surface elevation and general flow paths are not significantly affected by breach sizes within this range. Figure 2.8.3-1 provides a comparison of stage hydrographs at the top of the access road for the Cooling Tower area (North Carolina State Plane NAD83 Coordinate System Easting and Northing: 1381271 US ft, 479869 US ft). The maximum inundation depth difference between the three breach assumptions is less than one inch; and based on a review of model results, none of the three breach width assumptions significantly impact the CNS Yard due to the small volume in the Cooling Tower reservoirs. The 184-foot (22 percent) breach width was utilized in the final model setup. Figure 2.8.3-2 indicates the location of the final modeled breach.

The ICM model results show an assumed breach of a Cooling Tower reservoir produces isolated flooding with variable water surface elevations across the Cooling Tower yard area and does not significantly impact the CNS Yard when compared to the LIP or PMF combined effects events. Breached flood waters are shown to overtop the protective curb/berm at the upper rim of the Cooling Tower yard area; however, the maximum depth on the CNS yard is isolated to a small area within the security fencing and is less than 1 inch in average depth. Figures 2.8.3-3 and 2.8.3-4 provide representative stage, elevation, and velocity hydrographs.

This volume of water was not sufficient to bound the licensing basis LIP event discussed in Section 2.1 (SOF 2.8.3-01).

Figure 2.8.3-1 depicts the maximum inundation elevations in the Cooling Tower area. Additional details are provided in CNS-194292-011 Rev. 0.

FIGURE 2.8.3-1
COOLING TOWER BREACH WIDTH SENSITIVITY DEPTH HYDROGRAPHS (NORTH CAROLINA STATE PLANE NAD83 COORDINATE SYSTEM EASTING AND NORTHING: 1381271 US FT, 479869 US FT) (REFERENCE 17)

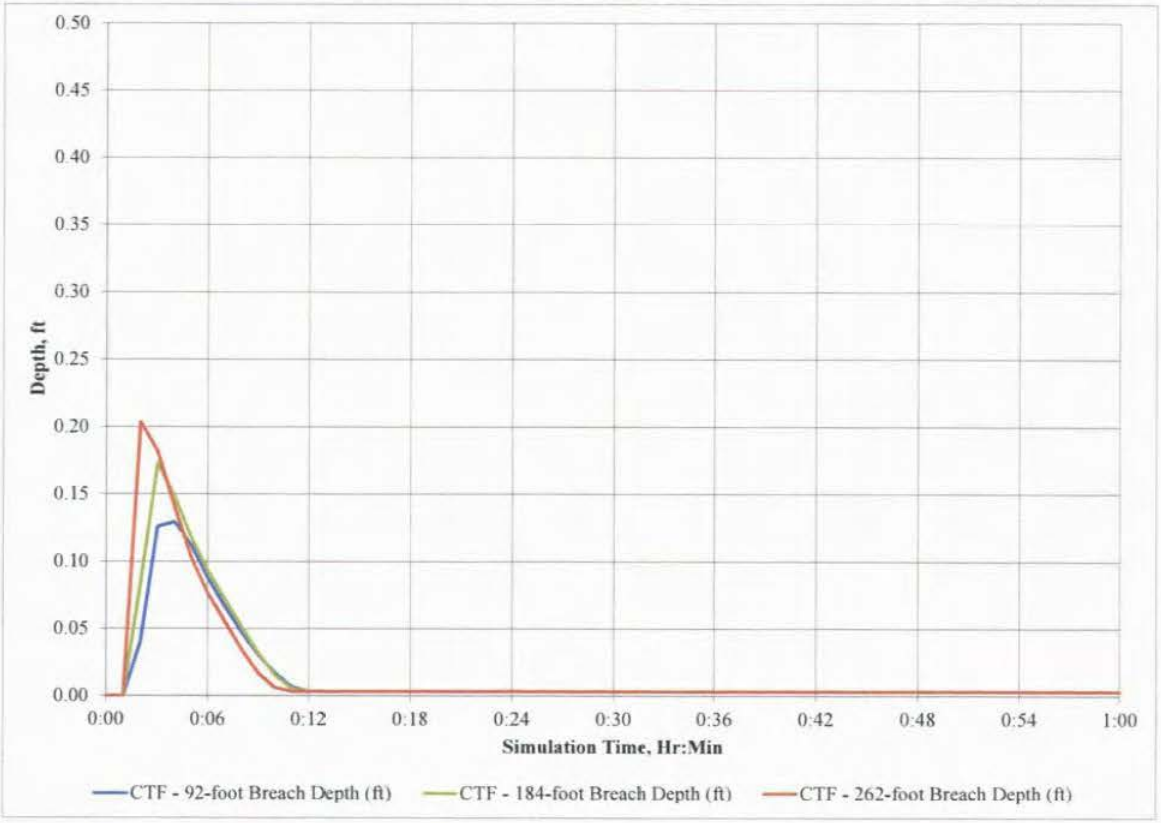
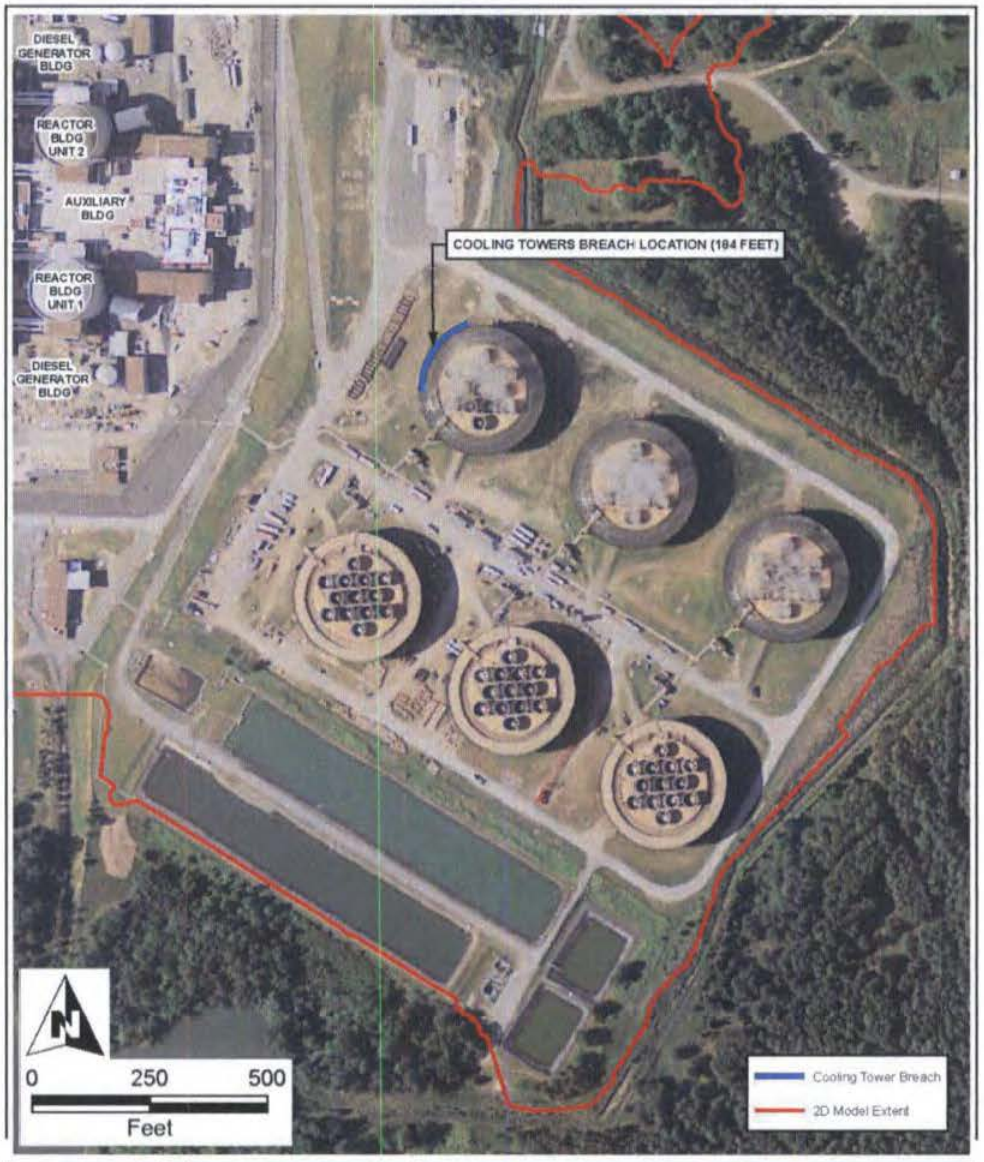


FIGURE 2.8.3-2
CNS COOLING TOWER 2A BREACH SIMULATION (REFERENCE 17)



HDR

Catawba Nuclear Station: 2D Boundary Condition
Calculation CNS-194292-011 (Combined Effects Model Results)
Infoworks ICM: Base Case
Hydrology: Combined Effects

FIGURE 2.8.3-3

CNS COOLING TOWER AREA REPRESENTATIVE ELEVATION AND DEPTH HYDROGRAPH DURING HYPOTHETICAL FAILURE OF COOLING TOWER 2A - NORTH CAROLINA STATE PLANE NAD83 COORDINATE SYSTEM EASTING AND NORTHING: 1381271 US FT, 479869 US FT (REFERENCE 17)

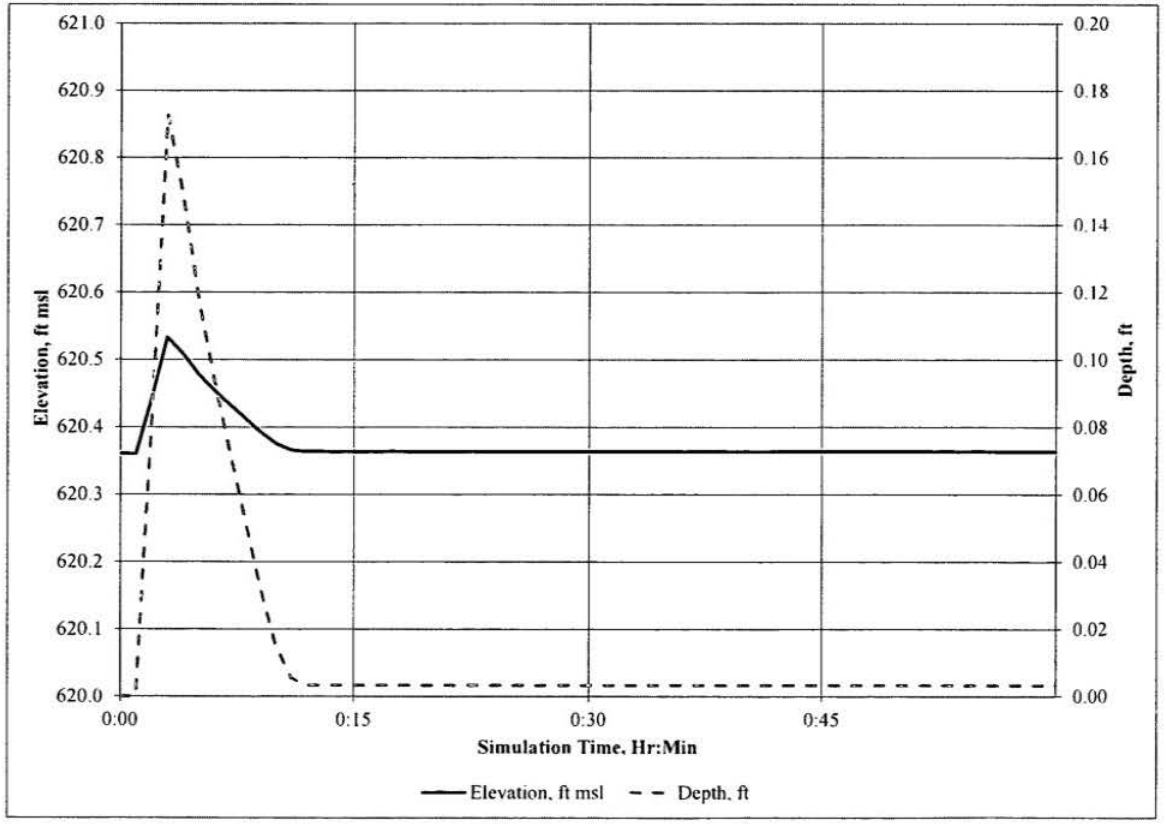


FIGURE 2.8.3-4

CNS COOLING TOWER AREA REPRESENTATIVE VELOCITY HYDROGRAPH DURING HYPOTHETICAL FAILURE OF COOLING TOWER 2A - NORTH CAROLINA STATE PLANE NAD83 COORDINATE SYSTEM EASTING AND NORTHING: 1381271 US FT, 479869 US FT (REFERENCE 17)

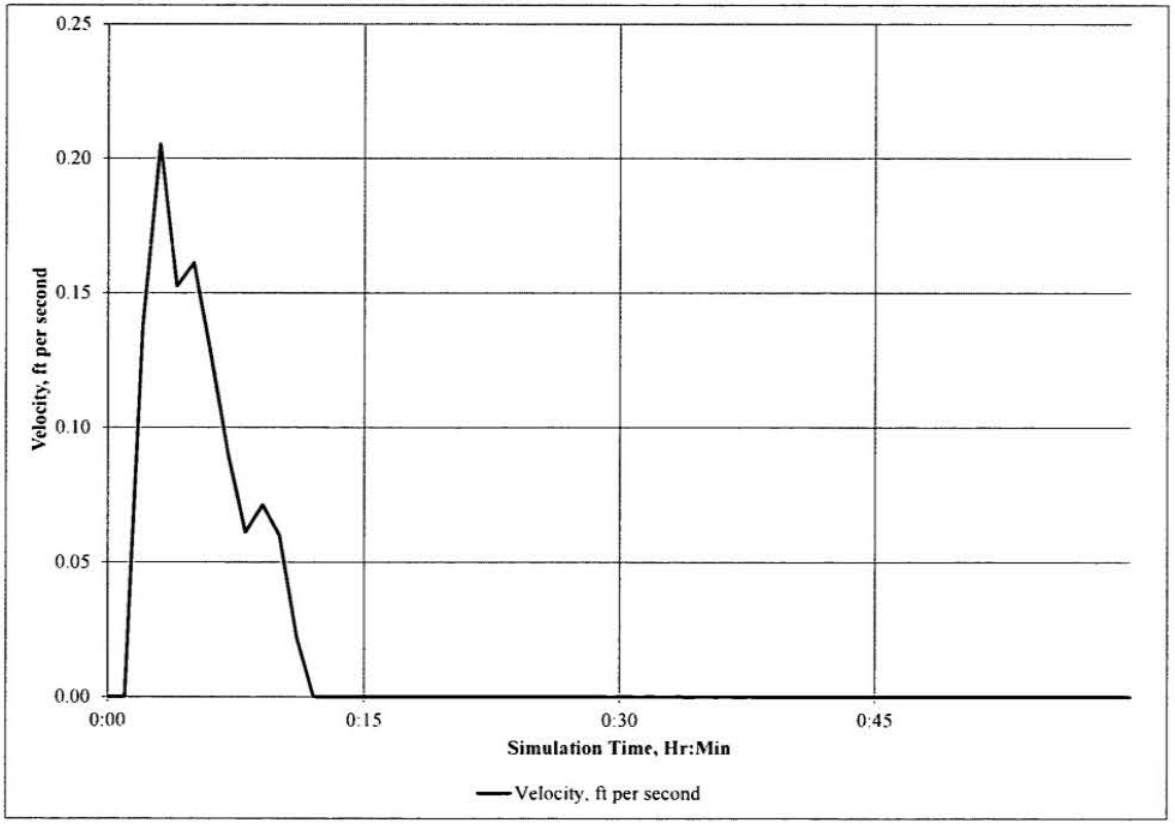
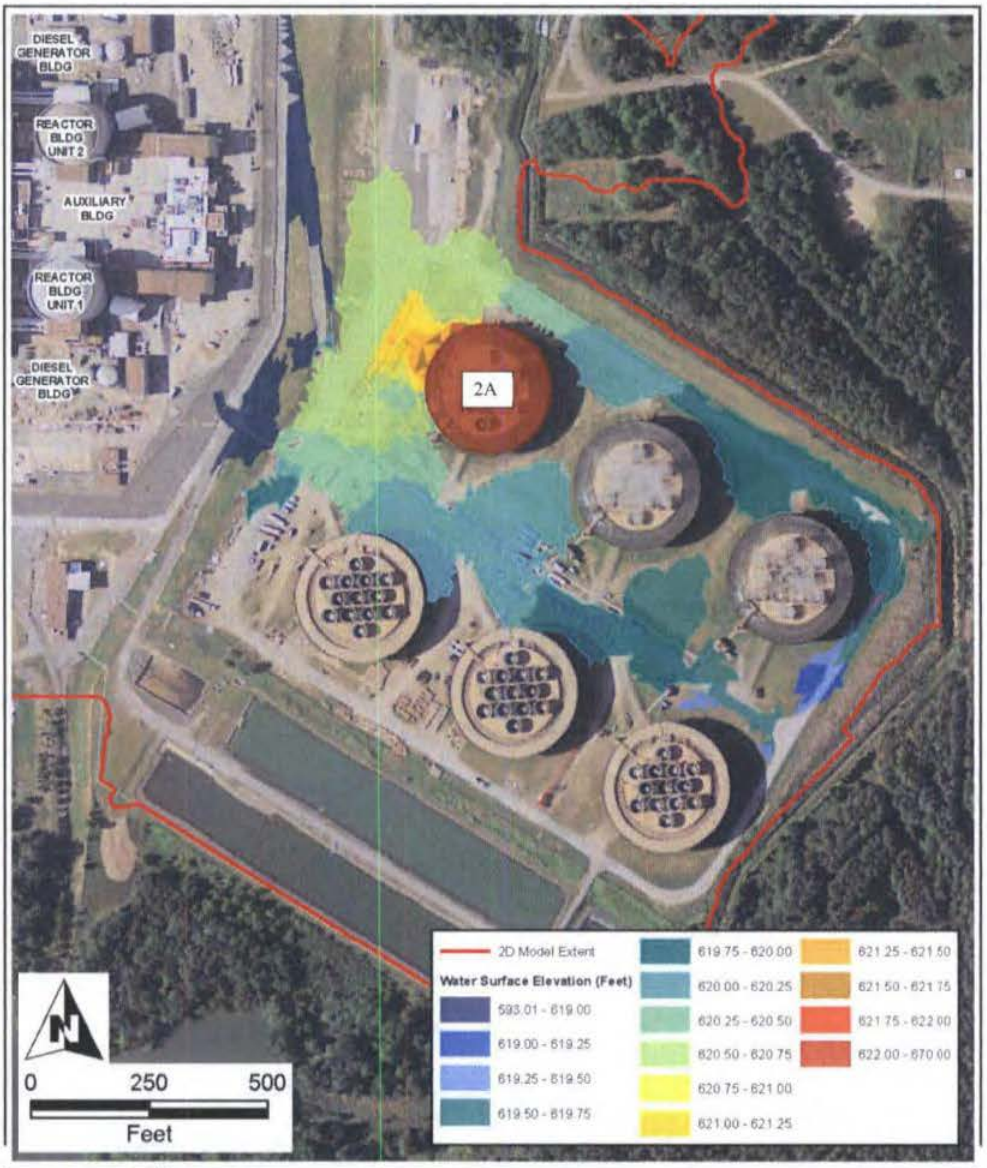


FIGURE 2.8.3-5
MAXIMUM INUNDATION IN COOLING TOWER AREA DURING A HYPOTHETICAL FAILURE OF COOLING TOWER 2A (REFERENCE 17)



HDR

Catawba Nuclear Station: Maximum Water Surface Elevation over Cooling Tower
Calculation CNS-194292-011 (Combined Effects Model Results)
Infoworks ICM: Base Case
Hydrology: Combined Effects

Section 3

Comparison of Current Design Basis and Reevaluated Flood Causing Mechanisms

Comparison of current design basis and reevaluated flood causing mechanisms is provided in this section through use of tables and text. Items discussed include SNSWP Dam and CNS Yard impacts related to flooding from, Cooling Tower breach, flooding on Lake Wylie, PMF, LIP, and combined effects. It should be noted that due to the different methods used to estimate flood hazard impacts at the CNS site between the current design basis and the reevaluated flood causing mechanisms a direct comparison is not always possible. This can be seen in the LIP analysis where new 2-D modeling methods provide significantly more simulation results at thousands of points where the current design basis is based on a simple 1-D cross section average approximation of water levels in the CNS Yard. This can also be see in comparison of combined effects stream routing modeling results where the reevaluation analysis used a combination of 1-D dynamic routing model to simulate the large Catawba River PMF routing and used a 2-D model to provide a more refined focus on the local CNS site impacts. The current design basis did not find an overall precipitation-based flooding event with or without assumed dam failures that exceeded the nominal CNS Yard elevation of 593.5 ft msl. The reevaluation flood hazard analysis

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

Table 3-1 provides a general summary of the flood causing mechanisms reviewed for the current design basis (Section 1) and the reevaluation (Section 2). The values provided in the table representing the reevaluation elevations are approximated from the different models used for the analysis and do not represent an absolute maximum inundation level at the site but in general were selected to match the location referenced in the current design basis.

TABLE 3-1
CURRENT DESIGN BASIS AND REEVALUATION FLOOD ELEVATIONS

Flood Causing Mechanism	Current Design Basis Flood Elevation	Reevaluation Flood Elevation	Reevaluation Flood Delta From Design Basis
Local Intense Precipitation			
Unit 1 side of the Yard	594.9 ft msl	595.5 ft msl	+0.6 ft
Unit 2 side of the Yard	595.9 ft msl	595.6 ft msl	-0.3 ft
Flooding in Reservoirs			
Lake Wylie CNS Intake	(b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft msl	(b)(3) 16 U.S.C. § 824o-1 (d), (b)(4), (b)(7)(F) msl[1]	(b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
SNSWP Dam	ft msl	msl[2]	
Dam Failures	ft msl[4]	msl[3]	
Storm Surge and Seiche/Wind-Wave Run-up Lake Wylie	577.8 ft msl	569.55 ft msl	-8.25 ft
Tsunami	N/A	N/A	N/A
Ice Induced Flooding	N/A	N/A	N/A
Channel Diversion	N/A	N/A	N/A
Combined effects			
Lake Wylie CNS Intake	593.7 ft msl	603.2 ft msl[5]	+9.5 ft
SNSWP Dam	594.4 ft msl	603.2ft msl[5]	+8.8 ft
Allison Creek CNS Discharge	593.4 ft msl	599.8 ft msl[6]	+6.4 ft

Notes:
[1] Catawba River Model Run Case Wylie_PMF_2A
[2] LIP ICM 2-D Model Result
[3] (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
[4] Wateree centered half PMF with Cowans Ford and Mt Island dam failure.
[5] (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)
[6]

3.1 SNSWP (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)

One area of significant difference between the design basis and the reevaluation is with the determination of the Catawba River PMF and combined effects flooding. The design basis hydraulic model appears to have been based on a "level pool" model that did not consider unsteady flow between dams. Experience with flood modeling of the Catawba River Dams through review of FERC-required PMF evaluations and modeling experience performing emergency action plan (EAP) analyses was considered for the reevaluation. Existing FERC compliance models have been developed using unsteady models and show a difference between the simple assumption of level pool routing and more detailed unsteady flow routing using the St Venant equations. Additionally, the hydrology applied for the design basis used a historic hurricane event plus a multiplier rather than current practice of applying HMR51/52. These modeling differences, coupled with the application of antecedent rainfall events preceding the PMF, lead to some differences in routing elevations at the (b)(3) 16 U.S.C. § 824o-1(d); (b)(4); (b)(7)(F) dams upstream of the CNS 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)

3.2 Cooling Tower Breach

A failure of a Cooling Tower by breach of the reinforced concrete water reservoir was not documented in the UFSAR. The reevaluation performed to support this report determined that for an assumed breach of the concrete reservoir, there are no significant flooding impacts to the CNS Yard (SOF 3.2-01). The amount of water stored in the cooling tower reservoirs and the topography of the foundation plateau to the southeast of the CNS Yard do not produce a significant source of flooding to effect the CNS Yard.

3.3 Riverine Flooding on Lake Wylie

Flooding on Lake Wylie

As outlined in Section 3.1, there was a difference in modeling performed for the design basis compared to the reevaluation. There are a number of reasons for this including the lack of computer models that could adequately simulate the complex hydraulic process of rainfall and runoff in the early 1970s. The reevaluation used different hydrology and hydraulic methods that produced different results for the PMF. Smaller events evaluated including upstream dam failures and the half PMF show better correlation between modeled reservoir elevations at

(b)(3) 16 U.S.C. § 824o-1(d), (b)

(b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b) Dam and Dam. This is likely due to the similarity in the volume and discharge parameters used in the models for flooding simulations where the spillway discharge capacities were not significantly exceeded. For the design basis PMF analysis, a historic hurricane event was modified using a real storm pattern to distribute the rainfall over the basin. In the reevaluation, HMR51/52 was used incorporating the elliptical storm pattern produced using these recommended methodologies. PMP estimates from HMR51 are based on the analysis of many historic storms which occurred over a significantly large portion of the United States producing a regional maximization of possible rainfall. The difference in the storm shape used for each evaluation resulted in various differences in storm centering and results from the PMF modeling along with the unsteady flow model used for the reevaluation. The reevaluation

generally yielded slightly higher reservoir levels at the dams. This should be expected since the rainfall distribution was fit to the basin using guidance from HMR52. Overall, the design basis flooding analysis was not significantly lower than the reevaluation considering all inputs and modeling capabilities. It is observed that applying backwater analysis methods to the design basis flows and stage would probably yield better agreement between the flooding results at the CNS site.

PMF Bounding Event

The bounding reevaluation flooding event was found to be (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) (ANS 2.8 9.2.2.1, Alternative I). The design basis bounding event was the half PMF plus a (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) the design basis modeling for the PMF did not result in (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam overtopping and failing.

The reevaluation analysis determined the bounding case through a series of model runs following a similar methodology as used for the original design basis. However, the revaluation analysis found that

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

Probable Maximum Flooding

The reservoir routing method described in the CNS UFSAR appears to be based on "level pool," storage assumptions including analysis of Lake Wylie reservoir levels. This method of hydraulic

analysis was common standard-of-practice at the time when the modeling was performed. Maximum reservoir levels reported in UFSAR Section 2.4.3.5 appear to be captured at the Wylie Dam. Because Lake Wylie is a river-shaped reservoir with storage tributaries, "fingers," located along the length of the reservoir, consideration of backwater impacts along the reservoir length from the dam to the CNS site (approximately 4.8 river miles) would likely result in higher water levels than recorded at the Wylie Dam in the licensing basis analysis for the extreme flood events evaluated. The reevaluation PMF analysis, summarized in Section 2, uses a "state of the practice" hydraulic dynamic routing model that considers backwater impacts in the routing of the flood flows in each reservoir and a comparison of water levels at Wylie Dam with water levels near the CNS site on the main reservoir channel confirms that water levels would likely be higher at CNS than model estimates using level pool routing.

NUREG 7046 recommends application of HMR51/52 (HMR applicable to the CNS site) to develop the PMP for the evaluation of the PMF (Reference 60). Previous licensing basis PMP-PMF analyses used a regional storm basis with adjustment for conservatism. A direct comparison of PMP values between previous studies and the analysis used for this review was not made; however, a comparison of the results of the licensing basis PMF routing and the updated HMR51/52 and HEC-RAS routing was reviewed and shows higher reservoir levels at

(b)(3): 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam and Dam. The combined effects PMP event for the (b)(3): 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam drainage area produced a

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

3.4 Local Intense Precipitation

The original design basis LIP was based on PMP from HMR33 and a 6-hour duration. The 6-hour event included a peak 1-hour rainfall of 9.2 inches. In 1984, revised LIP calculations were performed using HMR51/52 resulting in a 6-inch increase in a total of 6 hours of rainfall and a 10-inch increase in the peak 1-hour rainfall. The licensing design LIP was applied to the site using a 1-D model which is difficult to use when no defined channel exists. Site topographic

geometry approximations were used to create flow channels for the simulation and the existing catch basins were also included in the analysis to help remove the intense rainfall from the CNS Yard.

When comparing the 2-D modeling results analysis to the CNS licensing basis 1-D channelized analysis documented in the UFSAR, there are many differences that can be attributed to methodology, models, site drainage, and base hydrology assumptions. In many cases, several of these factors may be involved simultaneously, which makes it difficult to isolate individual sources of difference. The two primary factors identified by HDR as the largest contributors of deviations from the previous licensing basis analyses are the varying modeling methodologies (i.e., software used) and the total LIP rainfall amount (i.e., 1-hr LIP versus 6-hr LIP). The 2-D model is also capable of simulating water spilling from the building roofs and focusing this runoff at distinct areas of the CNS Yard more appropriately than the simple 1-D model. The maximum depths at specific nodes were not reviewed due to the differences in the output format (1-D cross section vs. 2-D mesh elements) but the overall maximum depth of inundation in the CNS Yard near critical structures was comparable as illustrated in Tables 1.2.1-3 and 2.1.4-1.

3.5 Dam Failures

Upstream dam failures were assumed in the licensing design basis flooding analyses and in the reevaluation analyses. Each upstream dam was evaluated for fair-weather failure and for producing cascading failures at sequential downstream dams.

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

Both analyses considered combined effects flooding assuming a seismic failure of an upstream dam during a half PMP event. The licensing design basis analyses found the failure of (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam during a half PMP event to be the CNS site bounding flooding event. The reevaluation analyses using the HEC-RAS unsteady flow model did not find any combination of seismic dam failures plus half PMP flooding to bound the flooding at CNS.

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

As noted in Sections 2.2 and 2.3 the reevaluation analysis following guidelines in ANS 2.8 9.2.2.1, Alternative I to be the bounding CNS Site flooding event producing a maximum of [redacted] ft of inundation on the CNS Yard and water levels [redacted] ft msl for periods between [redacted] hours as shown in Table 2.8.1-1.

(b)(3) 16 U.S.C. § 824o-1(d), (b)

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

(b)(3) 16 U.S.C. § 824o-1(d), (b)

3.6 Storm Surge and Seiche

Storm surge and seiche were addressed in the licensing design basis assuming hurricane-driven winds produced a seiche and the setup from the wave calculation was the surge component that was added to the wave height. The reevaluation did not find that storm surge and seiche was a probable physical flooding event for the CNS site as described in Section 2.4. The location of the Catawba River combined with the highly regulated river reaches, dams, isolates the river from storm surge associated with large bodies of water and tidal effects. Seiching calculations using methodology included in NRC/CR 7046 were used for Lake Wylie and found to produce small wave impacts less than derived using wind-driven waves outlined in Section 2.4.2.

3.7 Tsunami

Tsunami-induced flooding is not expected to affect the site for the reasons listed above in Section 2.5.

3.8 Ice-Induced Flooding

Ice-induced flooding is not expected to affect the site for the reasons listed above in Section 2.6.

3.9 Channel Diversion

Channel diversions are not expected to affect the site for the reasons listed above in Section 2.7.

3.10 Combined Effects

Combined effects flooding reviewed in the licensing design basis and reevaluation includes applicable sections of ANS 2.8 Section 9.2. The licensing design basis identified the combined Cowans Ford Dam failure plus half PMP flood and 2-year mean wind speed waves as the bounding CNS Site flooding event (Section 1.2.8). The reevaluation identified

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

The stage hydrographs were used with the CNS Site 2-D model and 2-yr mean wind speed waves to simulate flooding at the CNS Site and inundation (Section 2.8). The licensing design basis analysis did not find a river flooding event that resulted in inundation of the CNS Yard. The reevaluation found the CNS Yard would be inundated by as much as ft (not considering the SNSWP), and the inundation duration ft msl would be for approximately hours at various locations on the site as shown in Table 2.8.1-1. Inundation durations at the 22 door locations identified in the current design basis are shown in Table 2.8.1-2 with the inundation durations ranging from 11.25 hours to 18.6 hours. Water elevation in the SNSWP is expected to reach elevation ft msl or ft above normal pond level and ft above the design crest of the dam. Water levels in the SNSWP are contained within the natural topography around the pond and do not drain toward the CNS powerblock buildings. A failure of the SNSWP Dam would release water back into Lake Wylie and not flood the site.

(b)(3):16 U.S.C. § 824o-1(d), (b)

(b)(3):16 U.S.C. § 824o-1(d), (b)

(b)(3):16 U.S.C. § 824o-1(d), (b)

(b)(3):16 U.S.C. § 824o-1(d), (b)

(b)(3):16 U.S.C. § 824o-1(d), (b)

Section 4

Interim Actions Taken or Planned

4.1 LIP Actions

Flooding from the LIP event is managed with available Type I catch basins, procedures for securing doors in critical Safe shutdown facilities, sump areas, sump pumps, and sand bags.

Strategies to address specific water ingress areas are included in detail in Enclosure 1.

4.2 Combined Effects PMF Flooding

Flooding from the Combined Effects event is managed using strategies to protect the core, spent fuel, and containment in the event that severe site flooding does occur.

These strategies are addressed in detail in Enclosure 1.

Section 5

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Enclosure 1

FLOODING INTERIM ACTIONS FOR

Catawba Nuclear Station, Units 1 and 2

Docket Numbers 50-413 and 50-414

Renewed License Numbers NPF-35 and NPF-52

1 Evaluated Site Flooding Scenarios

This enclosure describes the interim actions taken at the Catawba Nuclear Station (CNS) in response to reevaluated flood elevations that exceed the Current Licensing Bases (CLB) flood elevations. The results of the Hazard Reevaluation Report (HRR) are summarized in Table 3-1 of Attachment 1 and the two flood event scenarios under which the Current Licensing Basis is exceeded are the Local Intense Precipitation (LIP) event and the Probable Maximum Flood (PMF) event due to a Combined Effect event. This condition has been entered in the CNS Corrective Action Program (CAP) (C-14-1940).

Impacts associated with the increased flood elevations and durations and the associated interim actions taken in response to the reevaluated LIP flood event are described in Section 2, and for the reevaluated PMF event are described in Section 3 of this enclosure.

2 Local Intense Precipitation (LIP) Event

2.1 LIP Event Overview

The Hazard Reevaluation Report identifies the Local Intense Precipitation (LIP) event as a flood event that exceeds the CLB of the plant. The LIP evaluation is described in Section 2.1 and the results summarized in Table 3-1 of Attachment 1.

The plant nominal grade is 593.5 ft msl (mean sea level), and the CLB credits existing flood protection features to provide protection from an external flooding event for an elevation up to 594.9 feet msl for Unit 1 and 595.9 ft msl for Unit 2.

The reevaluated LIP event for the Unit 1 side of the yard results in a water surface elevation of 595.5 ft msl, which exceeds the current design bases value of 594.9 ft msl by 0.6 feet. The Unit 2 side of the yard flood reevaluation results in a water surface elevation of 595.6 ft msl, which is lower than the current design bases value of 595.9 ft msl by 0.3 feet.

A comparison of the CLB and the HRR methodology and analysis is provided in Attachment 1, Section 3.

2.2 LIP Impacts

The results of the reevaluated LIP flood hydrographs show that flood levels exceed the current design bases flood protection elevation for a short and limited duration at critical water ingress locations. Debris impact is negligible due to lack of a credible debris field, low flow velocities and shallow flow depths during the reevaluated LIP event.

Based on the limited duration and a minor increase in flooding elevation compared to the current design bases values, the impact of the reevaluated LIP flooding to SSCs does not present a significant hazard to the safe operation of the plant. In addition, as part of the NTTF Recommendation 2.3 Flooding Walkdowns, credited flood protection features were inspected, and potential deficiencies entered and addressed in the plant CAP system. Impacts to Systems, Structures and Components (SSC) will be assessed as part of the Integrated Assessment.

Interim actions implemented to address the minor increase in flooding elevations and durations due to the reevaluated LIP are discussed in Section 2.3, LIP Interim Actions.

2.3 LIP Interim Actions

CNS procedure AP/0/A/5500/030 (Plant Flooding) includes severe weather warning notification from the Duke Meteorological department for forecasted severe weather and an initiating criterion for a projected rainfall event of 7 inches or more in 6 hours. This communication is provided to site personnel and provides adequate warning time for site preparation.

The HRR analysis conservatively assumed that all storm drains are blocked. However, the CLB credits the CNS site drainage system to remain functional during the LIP event. This functionality is assured through the implementation of a quarterly Preventive Maintenance (PM) program by CNS Site Services and an annual PM inspection program by CNS Design Engineering. In addition, NSD 104 (Material Condition, Housekeeping, Foreign Material Exclusion and Seismic Concerns) dictates general housekeeping policies that decrease the likelihood of significant debris inventory available to challenge the capacity of the site drainage system. A full time site services team routinely maintains the general site areas in an orderly fashion.

An interim action has been implemented to provide additional assurance of the storm drain system functionality by revising the plant flooding procedure AP/0/A/5500/030 (Plant Flooding) to perform a walk down inspection of the site drainage system catch basins when warning of a significant rain event is received. This walk down will visually inspect the storm drain system to confirm that the storm drain inlets are clear of debris. The inspection is expected to take approximately 3 hours for the actual inspection, which is well within the severe weather warning timeframe.

Training requirements were assessed for the revision to AP/0/A/5500/030 (Plant Flooding) during the walkthrough validation. It was determined no formal classroom training for the procedure revisions is required because the procedure steps consist of simple tasks.

Reasonable simulations were performed for the additional steps added to plant procedure AP/0/A/5500/030 (Plant Flooding) and the credited activities can be performed concurrent with environmental conditions that are expected to simultaneously occur.

The combination of the existing and revised procedures provides reasonable assurance that the storm drains will remain functional during the LIP event, providing mitigation for the postulated increased water elevations due to the reevaluated LIP events. Any additional leakage into critical areas of the plant can be handled by the credited site flood protection features, including internal drains and sump pumps.

The interim action strategies implemented have been evaluated for all plant operating modes and reasonable concurrent environmental and site conditions. The implementation of the interim actions will not reduce the effectiveness of site security requirements.

3 Probable Maximum Flood (PMF) Event

3.1 PMF Event Overview

The Hazard Reevaluation Report identifies the PMF due to a Combined Effect event as a flood event that exceeds the CLB values of the plant. The Combined Effect evaluation is described in Section 2.8 and the results summarized in Table 3-1 of Attachment 1.

The reevaluated PMF storm event is (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) and consists of an antecedent storm equivalent to 40% of the probable maximum precipitation (PMP) event that lasts for 72 hours, followed by a dry period of 72 hours, and then a 72-hour storm at event at 100% PMP. The resultant water surface elevations at (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

A comparison of the CLB and the HRR methodology and analysis is provided in Attachment 1, Section 3.

3.2 PMF Impacts

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

During the PMF event, flood water from (b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Based on a review of orthoimagery, debris that could impact the site is expected to consist of generally large, woody floating debris such as fallen trees or floating docks. This debris is expected to reach the CNS site at the CNS intake, which is located in a protective cove that is not directly aligned with the main channel flow of Lake Wylie. Due to the topography of the cove and the adjacent area, the flow velocities are lower than the flow velocities in the main channel, limiting larger and heavier debris from reaching the site. Debris conveyance across the CNS site from Lake Wylie would be the result of relatively shallow overland flow velocities.

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

Interim actions implemented to address the increase in flooding elevations and durations due to the reevaluated PMF are discussed in Section 3.3, PMF Interim Actions.

3.3 PMF Interim Actions

(b)(3) 16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) which is one of the dams on the Upper and Lower Catawba basin controlled by (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) An evaluation was performed and appropriate trigger points were established that indicate when an antecedent storm event has occurred, when conditions indicating a PMP event are likely, (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 triggers are based on a combination of the overall Catawba River basin performance, water elevations along the basin, the meteorological forecast, and the gate positions and discharge volumes through multiple dams along the basin, which are monitored and controlled by The trigger points established are at 144 hours, 72 hours, 15 hours and 5 hours prior to a potential flooding event at the CNS site. These triggers are documented in procedure "Catawba-Wateree Project CNS High-Water Notifications Process". CNS procedure RP/0/B/5000/030 (Severe Weather Preparations) has been revised to include CNS site preparation and responses to these trigger points for entry to site procedures and implementation of interim actions. (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)

Strategies have been implemented at the CNS site to prevent flood waters from reaching the site yard. The reevaluated PMF analysis shows that elevated water levels in Lake Wylie inundate the site from the north and as the flood progresses, from the south side due to backwater effects.

The two primary components of the interim actions are the installation of portable dams on the north and south side of the plant to prevent flooding from increased water elevations on Lake Wylie, and the installation of storm drain plugs to prevent water from back-flowing through the storm drain system due to elevated tailwater conditions in Lake Wylie.

Engineering evaluations were performed to support the interim actions taken. The evaluations included conservative case analyses on the portable dams and the intake slope for stability, scouring, debris impacts, overturning, sliding, and surcharge loading. An assessment was also performed for flooding from localized precipitation on site associated with the PMF on the watershed, and development of timing and sequence of deployment of interim action features.

The existing plant Emergency Response flooding procedure RP/0/B/5000/030 (Severe Weather Preparations) has been revised to include operator actions that correspond to the triggers received from [redacted] Three new Extensive Damage Mitigation Guidelines (EDMG) have been implemented to guide the installation and deployment of the portable dams, the storm drain plugs and sealing cable trenches, and associated equipment operation.

(b)(3) 16 U.S.C. § 824o-1(d), (b)

The need for training for revisions to the existing plant procedure RP/0/B/5000/030 (Severe Weather Preparations) and the new EDMG's was assessed through the Systematic Approach to Training (SAT) process and has been completed and formal training is not required. Equipment and material required to implement the interim actions associated with the PMF event has been acquired, and is being controlled through a Periodic Test (PT) procedure, PT/0/A/4400/002E (Flex Equipment Inspection). This procedure provides assurance that the equipment is available to perform the interim actions.

Reasonable simulations were performed for the steps added to plant procedure RP/0/B/5000/030 (Severe Weather Preparations) and for new EDMG's, and the credited activities can be performed concurrent with environmental conditions that are expected to simultaneously occur. The equipment identified in the procedures is available for use and is controlled through plant procedures.

The interim action strategies implemented have been evaluated for all plant operating modes, and reasonable concurrent environmental and site conditions. All required interim flood protection features have been acquired, pre-staged, and maintained to provide flood hazard protection. The implementation of the interim actions will not reduce the effectiveness of site security requirements as alternate site security measures will be implemented when needed.

The combination of existing, revised and new procedures provides reasonable assurance for providing protection from the postulated PMF Combined Effects event.

4 Summary of Interim Action Commitments

The following table summarizes interim actions taken or planned and their respective completion dates.

Catawba Nuclear Station - Flood Hazard Interim Action Commitments

Item Number	Initiating Event	Trigger	Interim Actions Taken or Planned	Implementation Date
1	Localized Intense Precipitation (LIP) Event	Flood Warning received from Duke Meteorological department per AP/0/A/5500/030 (Plant Flooding).	1. Revise AP/0/A/5500/030 (Plant Flooding) to include operator actions to clear or secure potential debris prior to event and during event.	March 12, 2014
Item Number	**Initiating Event**	**Trigger**	**Interim Actions Taken or Planned**	**Implementation Date**
2	Probable Maximum Flood (PMF)	Flood Warnings received from [redacted] per RP/0/B/5000/030	1. Add [redacted] procedures with appropriate triggers to CNS site.	March 12, 2014
			2. Revise RP/0/B/5000/030 to	March 12, 2014

(b)(3) 16 U.S.C. § 824o-1(d), (b)

(b)(3) 16 U.S.C. § 824o-1(d), (b)

	Event		include notifications from [redacted] and associated operator actions.	(b)(3) 16 U.S.C. § 824o-1(d), (b)
			3. Approve EDMG for portable dam installation.	March 12, 2014
			4. Approve EDMG for storm drain plug installation.	March 12, 2014
			5. Approve EDMG for pump operation.	March 12, 2014
			6. Approve inventory Periodic Test procedure to control equipment and material	March 12, 2014
			7. Acquire and stage required equipment and materials.	March 12, 2014
			8. Acquire and stage storm drain plugs.	March 12, 2014
			9. Training needs have been assessed and completed.	March 12, 2014
			10. Required site work has been completed.	March 12, 2014
			11. Engineering evaluations have been completed	March 12, 2014

Item Number	ADDITIONAL COMMITMENTS	
3.	A Flooding Integrated Assessment will be completed and report submitted to the NRC on or before March 12, 2016 (Letter CNS-14-028, Reference 1)	March 12, 2016.