ATTACHMENT 1

McGuire Nuclear Station Flooding Hazard Reevaluation Report

MCGUIRE 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

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REPORT VERIFICATION

PROJECT: **MNS 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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MNS FLOODING HAZARD REEVALUATION REPORT

TABLE OF CONTENTS

Section	Title	Page No.

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

MNS FLOODING HAZARD REEVALUATION REPORT

LIST OF FIGURES

Figure **Title** **Page No.**

LIST OF FIGURES
(Continued)

(b)(3):16 U.S.C. § 824o-1(d), (b)

LIST OF FIGURES
(Continued)

MNS FLOODING HAZARD REEVALUATION REPORT

LIST OF TABLES

LIST OF TABLES
(Continued)

LIST OF TABLES
(Continued)

Table	Title	Page No.

List of Acronyms

1-D, 2-D*	one dimensional, two dimensional, etc
ACES	Automated Coastal Engineering Software (ACES)
ASCE	American Society of Civil Engineers
CAL	Confirmatory Action Letter
CEDAS	Coastal Engineering Design and Analysis System
CFR	Code of Federal Regulations
cfs	cubic feet per second
DTM	Digital Terrain Model
ft	foot/feet
FERC	Federal Energy Regulatory Commission
FHR	Flood Hazard Report
fps	foot/feet per second
GIS	Geographic Information Systems
gpm	gallons per minute
HEC-RAS	Hydrologic Engineering Center's – River Analysis System
HHA	Hierarchical Hazard Assessment
HMR	Hydrometeorological Report
ICM	Integrated Catchment Model
in/hr	inch(es) per hour
LIP	local intense precipitation
mph	miles per hour
mi	miles
MNS	McGuire Nuclear Station
msl	mean sea level
NCDOT	North Carolina Department of Transportation
NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NRC	Nuclear Regulatory Commission
NSW	Nuclear Service Water

NTTF	Near-Term Task Force
NUREG	Nuclear Regulation
PMF	probable maximum flood
PMP	probable maximum precipitation
SCDOT	South Carolina Department of Transportation
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
SRM	Staff Requirements Memorandum
SSC	systems, structures, and components
STI	Supporting Technical Information
SWMM	storm water management model
TIN	triangulated irregular network
UFSAR	Updated Final Safety Analysis Report
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
Yard	MNS 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 current licensing 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 is prepared for the McGuire Nuclear Station (MNS) 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

McGuire Nuclear Station (MNS) is located in northern Mecklenburg County, North Carolina, approximately 17 miles north of Charlotte, North Carolina, and immediately east of Duke Energy Carolinas, LLC (Duke Energy) Cowans Ford Dam and Hydroelectric Station, which is regulated by the Federal Energy Regulatory Commission (FERC) under License 2232. Lake Norman which is impounded by Cowans Ford Dam is immediately north of the site. Lake Norman and Cowans Ford Dam are part of Duke Energy's Catawba River hydroelectric system containing 11 hydroelectric reservoirs and dams, and extending along approximately 221 miles of the Catawba River. Lake Norman forms the tailwater of Lookout Shoals Dam, located 34 miles upstream from Cowans Ford, and Mountain Island Lake forms the tailwater for Cowans Ford. Mountain Island Dam is located approximately 15 miles downstream from Cowans Ford (SOF 1.1-01).

The location and description of MNS presented in the *Updated Final Safety Analysis Report* (UFSAR) Chapter 1 and Chapter 2 includes reference to figures showing the general arrangement, layout, and relevant elevations of the station. The design of the MNS powerblock yard (Yard) grade is nominally 760 feet mean sea level (ft msl) (all elevations referenced in this report are based on National Geodetic Vertical Datum [NGVD] 1929).

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

Figure 1.1-1 shows an aerial view of the MNS site and Cowans Ford Dam (Reference Google Earth 2013). The maximum modeled Cowans Ford spillway discharge tailwater elevation is 698.50 ft msl, which is 61.50 ft below the McGuire yard elevation (SOF 1.1-02).

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

FIGURE 1.1-1
AERIAL VIEW OF MCGUIRE NUCLEAR STATION SITE

1.2 Current Licensing Design Basis Flood Elevations

1.2.1 Local Intense Precipitation

The plant safety-related buildings are protected against flooding due to a maximum local intense precipitation (LIP) also known as the point probable maximum precipitation (PMP) with a system of roof drains, a surface collection system, and ditches designed to direct runoff away from the plant to natural drainage channels. Roof drains designed to discharge 5 inches per hour have been installed on all safety-related buildings (SOF 1.2.1-01). Continuous architectural parapets are affixed to the roof at elevations 804.50 and 825.00 ft msl of the Fuel Handling Building and on the roof of the Reactor Building, respectively. Parapets are not on more than three sides on all other roof levels of the Auxiliary Building. The parapets on the Fuel Handling Building have been fitted with scuppers to permit water that may pond on the roof to be discharged onto the plant yard. The scuppers are 6 inches high by 9 inches long and are placed 3.5 inches above the finished roof surface. The calculated depth of water on the Fuel Handling Building roof during the LIP assuming all roof drains are clogged was calculated to be 11.5 inches. The Fuel Handling Building roof has been checked for this loading. The parapets on the Reactor Building are not fitted with scuppers. The Reactor Building roof is designed to withstand the hydrostatic load resulting from the maximum accumulation of water to the crest of the parapet during the LIP. There is no parapet at the south end of the main level of the Auxiliary Building at Elevation 784 ft msl. A pathway was created to discharge water that may pond on the roof during the LIP. The roof loading due to the maximum accumulation of water does not exceed the current licensing design basis loading for any portion of the Auxiliary Building roof (SOF 1.2.1-02).

All pipe sleeves, ventilators, and curbs penetrating the roof of safety-related buildings have been extended above the estimated level of ponding to eliminate flow paths during a local LIP. All hatches and other safety penetrations are adequately waterproofed to ensure their integrity during the LIP.

The buried storm drainage system is designed to remove precipitation of up to 4 inches per hour with additional precipitation ponding in the plant yard or overflowing the plant yard perimeter by sheet flow. Considerable storage of precipitation results from the 1-foot differential between the plant yard high points and ridge lines at elevation 760 ft msl and the top of the catch basins at elevation 759 ft msl. This creates pockets of storage around the plant yard, which have a capacity of approximately 155,000 cubic feet (SOF 1.2.1-03). Runoff is routed away from the plant buildings toward the catch basins with a minimum design ground slope of 1.4 percent. Although the yard drainage system, itself, was not designed to discharge the PMP, the system has been evaluated to ensure that the inundation of water due to PMP will not endanger any safety-related facilities (SOF 1.2.1-04).

The floodwater elevation due to a LIP was evaluated by applying the rational method of six 1-hour rainfall intensities. These intensities ranged from 2.4 inches per hour to 14.7 inches per hour and arranged in the following order: 2.4, 2.4, 3.6, 14.7, 4.5 and 2.4 inches for a total of 30 inches of precipitation over a 6-hr period (SOF 1.2.1-05). The rational method was applied to route the rainfall across the site. Two methods were used to analyze the effects of excess water backup on the structures. The first method of analysis assumed that there was perimeter runoff and that the storm drainage system was operating at one-half of its total capacity. This accounted for any debris or obstacles partially blocking the drain system. Using this method of analysis, the water was estimated to pond to an elevation of 760.28 (SOF 1.2.1-06) ft msl (Reference 6). This elevation is below the exterior doorway curbs of the safety-related structures. All exterior doorways are provided with curbs (or thresholds) at Elevation 760.5 ft msl (SOF 1.2.1-07) (Reference 6).

The second method of Yard inundation analysis assumed that the storm drainage system is completely inoperative or totally blocked and that the entire LIP runoff is discharged by sheet flow at the perimeter of the yard. The assumption was also made that the perimeter of the protected area would act as a weir for runoff to overflow the perimeter. Thus, when the quantity of flow from the PMP equaled the quantity of flow crossing the weir in a given period of time, equilibrium would be reached and the depth of ponding could be determined (level pool routing). With this method of analysis, some of the plant structures would act as obstructions to water flowing over the entire weir; therefore, the length of the weir was not assumed to be the entire distance around the plant but was divided into segments. These segments were estimated by reviewing flow paths through the Yard. Using this method, the water was estimated to pond to an elevation of 760.375 ft msl (SOF 1.2.1-08). This elevation is below the elevation of the exterior doorway curbs (or thresholds) of the safety-related structures. This analysis was performed to evaluate the assumption of the effectiveness of the subsurface storm drainage system. Based on this analysis, subsurface drainage is not required to be functional to protect the safety-related structures at the McGuire site. Therefore, based upon the current licensing basis

LIP calculations assuming no sub-drainage, the maximum water depth adjacent to the plant could reach Elevation 760.375 ft which results in 0.125 ft of LIP flood margin.

1.2.2 Flooding in Reservoirs

1.2.2.1 Catawba River Reservoirs

The main hydrologic/hydraulic features influencing the MNS plant site are the Catawba River and a series of five reservoirs that regulate the river upstream and along the shores where the MNS site is located (Lake Norman). The headwaters of the Catawba River are at the Blue Ridge Divide (Eastern Continental Divide) near Old Fort, North Carolina. The river generally flows 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 as shown in Figure 1.2.2.1-1.

FIGURE 1.2.2.1-1
CATAWBA WATEREE RIVER BASIN SHOWING DUKE ENERGY FERC HYDROPOWER DAMS

(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) [redacted] At full pond elevation 760 ft msl, Lake Norman has a surface area of approximately 32,339 acres (ac), a shoreline of approximately 603 miles, average depth of 33.5 ft and a volume of 1,093,600 acre-feet (ac-ft). Its total watershed is approximately 1,790 square miles (sq mi) (SOF 1.2.2.1-01) (References 32 and 67). Mountain Island Lake extends to and forms the tailwater of the Cowans Ford Station. Normal tailwater elevation is approximately 645 ft msl. The tailrace is an excavated channel with a length of approximately 810.00 ft.

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] Usable storage is defined as the water volume in each reservoir available to support hydropower generation and is typically measured from the normal full reservoir elevation to a maximum FERC-licensed drawdown elevation. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] 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).

FIGURE 1.2.2.1-2
MCGUIRE NUCLEAR STATION AND COWANS FORD DAM. (REFERENCE CF STI FIGURE AP-CF-1)

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 seven hydropower reservoirs upstream of the gage. On July 17, 1916, the Catawba River near the Cowans Ford Dam location reached an estimated flood discharge of 199,500 cfs (SOF 1.2.2.1-03).

Original Project studies were conducted prior to the release of Hydrometeorological Report (HMR) 51/52 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 MNS UFSAR.

The greatest storm recorded over the Cowans Ford Damdrainage 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 produced diminishing amounts of precipitation by transposing these storms inland some 200 mi to the Cowans Ford Dam 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 Cowans Ford Dam watershed, snow melt is not a consideration because of its southern location. Hourly incremental rainfall and rainfall in excess for the 54-hour period of the PMP are available for inspection in the "Regulatory Compliance Licensing Files" as UFSAR

Table 2-19, and details of the methodology used to conduct the hydrologic study are presented in former McGuire Site UFSAR Appendix 2F.

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 current licensing design basis study, an infiltration rate of 0.10 inch per hour was selected (SOF 1.2.2.1-04).

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 was routed through the Catawba River system to Cowans Ford Dam by means of a flood routing program developed by C. T. Main (November 1968). [redacted]

(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. Reservoirs with gated spillways were operated on the basis of first filling or attempting to fill the reservoirs to a predetermined control elevation and thereon assuming outflow equal to inflow until the gate capacity is exceeded. 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 were 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.

Personnel from [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The operation of the Catawba River system modeled in the C.T. Main computer program is consistent with Duke Energy hydro operations practice.

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

In cases where reservoir elevations overtopped the bulkheads of reservoirs, it was assumed the bulkheads did not fail. Appropriate discharges are provided for in the routing program considering these structures as broad-crested weirs. Where earth embankments are overtopped, it is assumed that there were no failures for a small amount of overtopping and the discharges followed that of broad crested weirs. However, when the flow depth reaches 2 ft or more, it was assumed that progressive failures of the embankments by erosion takes place along the overtopped crests and exposed ends.

[redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Eleven trial positions of the storm center were made to determine the most critical position for producing the maximum flood over Lake Norman. The storm center was positioned over each of the reservoir drainage areas in turn and then routed through the Catawba River system into Lake Norman. This methodology is standard practice to maximize the PMP rainfall isohyetals over the drainage to produce the most runoff impacts. The

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

[redacted]

TABLE 1.2.2.1-1

LAKE NORMAN PROBABLE MAXIMUM FLOOD FLOODING RESULTS

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

[redacted]

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

[redacted]

Based on the above discussion, there is adequate margin against faulty gate operation coincident with the PMF on Lake Norman.

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

[redacted]

FIGURE 1.2.2.1-3

COWANS FORD EAST EMBANKMENT AND MCGUIRE [redacted] FT MSL DIKE (REFERENCE 32)

(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)

[redacted]

Considering the plant layout, the MNS site can be characterized as a "flood-dry site," as described in Section 5.1.3 of the American National Standard Report, *Determining Design Basis Flooding at Power Reactor Sites*, because the safety-related structures of the existing MNS are above spillway discharge flooding elevations (Reference 11). The Yard is nominally 760 ft msl and during the discharge of the licensing basis PMF through the Cowans Ford spillway, discharged water is not expected to backup significantly over the river elevation of approximately 698.5 ft msl (SOF 1.2.2.1-06). This meets the intent of the definition of a "flood-dry site."

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 immediately south of the MNS Yard (Figure 1.2.2.1-2). 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*. Due to the small drainage area, the PMP (30.0 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 first 0.5 inch of rainfall and 0.1 inch per hour were subtracted to provide for interception and infiltration based on USACE-EM 1110-2-1411, 1952, Plate 19. The first 6-hr incremental runoff values are presented in Table 1.2.2.2-2 (SOF 1.2.2.2-03). A total of 48 hours was used for the analysis with the hourly values estimated using the graph in Figure 2, zone 6 (*Design of Small Dams*) multiplied by the PMP for 6 hours to determine the rainfall for 12, 24, and 48 hours. This produced a rainfall of 0.35 inch per hour for hours 7 through 12 and 0.1 inch per hour for hours 13 through 24 and 0.01 inch per hour for hours 25 through 48 (SOF 1.2.2.2-04).

TABLE 1.2.2.2-2
SNSWP DRAINAGE BASIN HOURLY INCREMENTAL RUNOFF

Hour	Runoff (inches)	Inflows (cfs)
1	1.8	310
2	2.3	397
3	3.5	603
4	14.6	2,517
5	4.4	752
6	2.3	397

Inflows for hours 7 through 12, 13 through 24, and 25 through 48 were 60, 17, and 2 cfs, respectively. Based on these small inflow values it can be concluded that the period after 6 hours did not need to be considered due to the size of the basin.

A spillway rating curve was developed for the discharge from the weir structure and outlet pipe using methods described in the *Handbook of Hydraulics* (Reference 68). The discharge calculations considered control at the inlet weir discharge up to elevation 742.5 ft msl (320 cfs) and the discharge through the outlet pipe controlling above SNSWP elevation 742.5 ft msl (SOF 1.2.2.2-05).

Inflows into the reservoir were converted into ac-ft for the first seven 1-hour intervals and the *Design of Small Dams* graphical method of flood routing was used to determine the maximum reservoir elevation. The maximum reservoir elevation of 746.9 ft msl is reached during the sixth hour of rainfall. This produces in a net excess inflow into the reservoir of 292 ac-ft and a surcharge on the pond of 6.9 ft. Additional details are provided in UFSAR Appendix 2G (Reference 11).

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

[redacted] Therefore, the toe of the dam is not expected to experience erosion from fast moving water and the dam is considered stable during the Cowans Ford PMF discharge conditions.

1.2.2.3 Groundwater

Due to the location of the MNS site on natural ground directly downstream of the Cowans Ford East Embankment, preconstruction groundwater levels were approximately 10 to 35 ft below plant yard grade of Elevation 760 ft msl (SOF 1.2.2.3-01). Reactor, Auxiliary and Turbine Building excavations in soil and weathered rock below plant yard grade were dewatered by eductor wellpoints located on the western, northern, and eastern perimeter of the excavation. Excavations in rock below plant grade were dewatered by excavated sumps located at convenient construction locations. The following description of the underdrain groundwater system is largely repeated from MNS UFSAR Chapter 2, 2.4.13.5.

A permanent Category I underdrain groundwater system was installed as shown on MNS UFSAR Figure 2-62 and Figure 2-63 to maintain the groundwater level below Elevation 717 ft msl for the Reactor Building and Elevation 712 ft msl for the Auxiliary Building. The underdrain system consists of a grid of interconnected flow channels at the top of rock or top of fill concrete below the foundation slabs. The grid of flow channels drains the entire foundation of the Reactor Building, and Auxiliary Building complex except for deeper pits which are designed for hydrostatic loads. Drilled holes through fill concrete into rock, at a maximum spacing of 8 ft on center, permit groundwater to flow from beneath the fill concrete slabs into the flow channels. All channels in the grid system drain by gravity to three sumps located in the Auxiliary Building. Groundwater collected in the sumps is pumped to the yard storm drain system or to Turbine Building sumps. Two 250 gallons per minute (gpm) Category I pumps, each capable of handling the total flow into the sumps for a pump cycle, maintain the water level automatically in each sump. In the unlikely event a pump fails to start and water rises above the normal operating level of the sump, the second pump will automatically start and will continue to operate as required. If either or both pumps fail to start, an alarm will alert the operator. Since the three sumps are interconnected by the grid drain channels at Elevation 712 ft msl, all six pumps are available to discharge groundwater. In the unlikely event that two pumps become inoperable in any one sump, groundwater would flow through the many redundant channels to the other sumps. Calculations for estimating the groundwater flow are presented in MNS UFSAR Former Appendix 2D Section 5.1.1 (Reference 11).

Four independent discharge lines, each capable of handling the system capacity, are provided to discharge groundwater from the Auxiliary Building sumps. Pump logic is provided to have the maximum system flexibility. Groundwater collected in sump C is pumped to a free outfall at the storm drain system through separate discharge lines for each pump. The free outfall drains to the storm drain system and prevents siphoning to the groundwater sump. In the event the storm drain system becomes blocked, the sump discharge would flow to adjacent catch basins or would discharge off the yard by sheet flow. The invert of the free outfall is located 2 ft above yard grade, 760 ft msl, to prevent flooding of the Groundwater Drainage System during the local PMP event.

Multiple redundancy of vital system components assure the ability of the system to function over the life of the plant. In the unlikely event that a single-flow channel or wall drain becomes blocked, groundwater will flow to the sumps through any of the many redundant drain routes available. Six Category 1 pumps, each capable of handling the total estimated flow, assure the function of the sump. Monitoring of pump operation provides assurance that the zoned wall filter, drains, and pumps are properly functioning. Since the zoned filter wall drain system is confined by building walls and the compacted earth backfill (or rock excavation at the foundation level), the wall drain system will remain passive during an earthquake as will the underdrain system. Since the top of the zoned wall filter is 5 ft below plant yard grade, there is no credible flood that will affect the underdrain system (SOF 1.2.2.3-02).

The postulated failure of the Nuclear Service Water pipe has been evaluated to determine the potential for flooding the groundwater underdrain system. The pipes for this system penetrate the zoned wall filter and allow the largest discharge of water into the underdrain system. The Nuclear Service Water System (NSWS) is a moderate energy fluid system and has been evaluated according to NRC Branch Technical Positions MEB 3-1 and APCSB 3-1. A through wall leakage crack, one-half the pipe diameter by one-half the wall thickness, would result in a flow of 666 gpm to the underdrain system. This flow plus the calculated groundwater seepage would result in a total flow of 696 gpm. Since six 250-gpm pumps are available to discharge groundwater, the postulated failure of the Nuclear Service Water pipe will not flood the underdrain system (SOF 1.2.2.3-03).

1.2.3 Dam Failures

As noted in Section 1.2.2.1, Duke Energy manages the Catawba River through a series of hydropower reservoirs and dams regulated by the FERC (Figure 1.2.3-1). [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) PMF routing did not result in upstream failure of any of the significant dams on the Catawba River (Table 1.2.3-1). 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.

TABLE 1.2.3-1

SIGNIFICANT DAMS UPSTREAM OF MNS

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

Table 1.2.3-2 provides a list of the upstream dams and drainage areas in sq-mi for each reservoir (SOF 1.2.3-01).

TABLE 1.2.3-2

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

Dam failure flood design considerations for the licensing basis at the MNS site included:

1. A Standard Project Flood (SPF) passing through Lake Norman 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.

2. PMF resulting from the PMP in the drainage area.

The seismic failure for each upstream dam was timed to coincide with the SPF Storm centered over its drainage area. At the hour in 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 MNS UFSAR Sections 2.4.4.2, 2.4.4.3, and 2.4.10 (Reference 11). The results of the test scenarios showing maximum reservoir elevations at each upstream reservoir are shown in Table 1.2.3-3 (SOF 1.2.3-02).

FIGURE 1.2.3-1
CATAWBA RIVER DUKE ENERGY FERC HYDROPOWER DEVELOPMENTS

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

TABLE 1.2.3-3
COMBINED EFFECTS SPF PLUS DAM FAILURE SIMULATION SCENARIO RESULTS

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

The effects of seismic failure of each individual dam, coincident with the SPF were analyzed; and of all of the various potential dam failures that were analyzed, the failure of an upstream (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] dam produced the highest water elevation at the McGuire site. The (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted]

(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] Dam were evaluated by performing a series of model simulations with different storm centers locations. Failure of dams was considered if the peak reservoir elevation exceeded the top of dam by 2 ft. The results of the PMF modeling are presented in Plate VI in Former Appendix 2F of the UFSAR. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted]

TABLE 1.2.3-4

PMF SCENARIO RESULTS

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

[redacted]

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

[redacted]

FIGURE 1.2.3-2

(b)(3):16 U.S.C. § 824o-1(d), (b)

MCGUIRE ELEVATION [redacted] FT MSL DIKE AT THE MNS INTAKE

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

[redacted]

FIGURE 1.2.3-3

(b)(3):16 U.S.C. § 824o-1(d), (b)

MCGUIRE ELEVATION [redacted] FT MSL DIKE AT MNS DISCHARGE

20.0'

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

[redacted]

FIGURE 1.2.3-4

COWANS FORD EAST EMBANKMENT SEISMIC FAILURE SECTION

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

The maximum flood stage of the Catawba River immediately downstream of Cowans Ford Dam was estimated following the postulated breach of the dam. The effects of the floodwaters from Lake Norman were evaluated for the following three combinations of events shown in Table 1.2.3-5.

TABLE 1.2.3-5

EFFECTS OF FLOODWATERS – EVENT COMBINATIONS

Seismic Event	Flood	Initial Breach Width (ft)	Peak Discharge (cfs)	Catawba River Stage (ft msl)
OBE	SPF	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted]		
OBE	SPF			
SSE				

Reference MNS UFSAR Section 2.4.10.

The MNS yard and associated safety-related facilities are located at Elevation 760 ft msl which is above (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] msl (SOF 1.2.3-04) and river stage and, therefore, are protected from inundation damage as a result of the dam breach.

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

1.2.4 Storm Surge and Seiche

The maximum wave height and wave run-up were calculated for Lake Norman based on theoretical methods. With an adjusted Probable Maximum Hurricane (PMH) maximum wind velocity of 96 mph blowing in the direction of the effective fetch to the earthen dike, the surge (wave setup) was computed to be 0.90 ft. Applying the maximum pressure gradient during a PMH, the seiche is estimated to be 1.93 ft (Reference 11). The following calculation methodology was assumed:

1. The eye of the hurricane is centered over the McGuire site.
2. The effective seiche distance is 9 miles assuming Lake Norman is at Elevation 760 ft msl.
3. The atmosphere pressure varies according to Formula 1 of the National Oceanic and Atmospheric Administration Report HUR 7-97.

The run-up associated with the breaking of significant and maximum waves caused by PMH is 9.41 ft and 11.92 ft, respectively, as shown in UFSAR Table 2-20 and Table 2-21. The most severe combination of surge (assumed to be setup), seiche, and wave run-up with Lake Norman at full pond (Elevation 760) results in water elevation of 774.75 ft msl for maximum waves and 772.24 ft msl for significant waves. This results in no overtopping of the (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] Dam embankment at (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] msl protective dike upstream of the plant yard. Therefore, wave run-up presents no problems to any safety-related facilities (SOF 1.2.4-01).

1.2.5 Tsunami

Tsunamis were never postulated to affect the site, and no flood elevation is given in the current licensing/design basis case basis of the plant. MNS 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 current licensing/design basis case basis of the plant. The climate in the Catawba River basin is moderate (minimum monthly mean water temperature for Lake Norman is in the low 40's) (SOF 1.2.6-01) and there has not been any recorded ice formation on the 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 current 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 Norman, the MNS could be safely shut down using the SNSWP. The SNSWP was constructed in a small tributary to the main channel of Mountain Island Lake and is protected from scour by topographic features.

1.2.8 Combined Effects

Combined flooding effects (PMP, PMF, dam failure and/or wind-driven waves) were reviewed for impacts at the MNS site. Maximum upstream water level elevation at the station occurs with the [redacted] Dam PMF as reported in Section 1.2.2.1. The maximum water surface elevations when combined with wind-driven waves are shown in Table 1.2.8-1 (SOF 1.2.8-01).

Coincident flood events (Reference 1), considered for (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam and the adjacent embankments include:

1. The PMF level (Elevation 767. 9) plus associated wind generated waves and run-up.

2. The Normal Full Reservoir level (Elevation 760) and associated wind-generated waves and run-up coincident with hurricane/seiche winds.

TABLE 1.2.8-1

COMBINED EFFECTS MAXIMUM RESERVOIR ELEVATION WITH WIND-DRIVEN WAVES

Location	**Elevation ft msl Plus 96 mph Wind**	**PMF Elevation Plus 40 mph Wind**
Setup (Surge) ft	0.90	0.16
Seiche ft	1.93	n/a
Maximum Wave Height [1] ft	13.54	5.05
Maximum Run-up [2] ft	11.92	4.95
Maximum Still Water Elevation ft msl	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)	
Maximum Water Elevation [3] ft msl		
Top Elevation of Structure ft msl		

Notes:
Reference MNS UFSAR Section 2.4.5 and Table 2-20.
1. The maximum wave height computed from significant wave heights.
2. Maximum run-up calculations include wind tide.
3. Total of Maximum Still Water elevation and wind-wave run-up and Seiche.

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

Possible downstream flooding impacts to the MNS SSCs were evaluated for a failure of the (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft msl embankment in Section 1.2.3. Adequate freeboard margin was found for the MNS Yard at Elevation 760 ft msl and the SNSWP Dam.

1.3 Licensing Basis Flood-Related and Flood Protection Changes

MNS has not made any licensing basis flood-related or flood protection changes.

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 were captured in the modeling performed to support the flooding assessment due to LIP and dam failure inundation using recent aerial and ground survey data (2013) along with updated drainage, utility trench location, and building geometry.

There has been construction of housing and support facilities directly around Lake Norman in the watershed since 1963, but the overall percentage of land use has not significantly changed since the construction of the Catawba-Wateree reservoirs and MNS. There is no significant change in land use around Lake Norman. Most of the Catawba-Wateree River watershed is comprised of protected forest lands.

1.5 Current Licensing Basis Flood Protection and Mitigation Features

A MNS site walkdown of flood protection features was performed in accordance with NEI 12-07, *Guidelines for Performing Verification Walkdowns of Plant Flood Protection Features*. A report was 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 current licensing design basis walkdown (Reference 6). Below is a summary of the findings presented in the November 15, 2012, report.

- The results of the walkdowns performed for the flood protection features that are credited in the current licensing basis did not identify any degraded, non-conforming, or unanalyzed conditions. There were no deficiencies determined by the corrective action program, and there were no observations that were dispositioned as deficiencies. As a result of the walkdown process, all of the flood protection features that are credited in the current licensing basis were inspected, and all were evaluated to be acceptable. The available physical margins for all of the flood protective features have been collected and documented on the Walkdown Record Forms.

- Flood protection features with cliff-edge effects denote that the safety consequences of a flooding event may increase sharply with a small increase in the flooding level. As a result of the walkdown process, none of the flood protection features were identified as having cliff-edge effects.

There are no planned or newly installed flood protection systems or flood mitigation measures identified as a result of the flood walkdown process. It was determined that no additional changes were required to further enhance the flood protection at the plant site.

Section 2

Flooding Hazard Reevaluation

The reevaluations in Section 2.0 are not part of the MNS licensing basis as documented in the UFSAR and are considered beyond licensing basis. 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 58).

MNS is located on the east bank of the Catawba River, just downstream of Cowans Ford Dam, a FERC-regulated hydroelectric development that consists of Lake Norman impounded by a concrete gravity dam and powerhouse, gated spillway, two embankments, and the McGuire and Hicks Crossroads dikes. 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 14). 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 floodwater elevation due to a maximum PMP was reevaluated using current practice HMR PMP and state-of-the-practice engineering software. The analysis evaluated the maximum water surface elevation within the MNS power block area resulting from the occurrence of the PMP and updated site topography and structure layout. This reevaluation analysis 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, one-dimensional (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 two-dimensional (2-D) flow equations for surface flow on relatively flat MNS 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 MNS 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 MNS 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 MNS, 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 by the red dot 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 18.8 inches per hour (in/hr) (Reference 17) as illustrated in Figure 2.1.1-2 (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 18.8 in/hr. (Reference 17) (SOF 2.1.1-02).

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

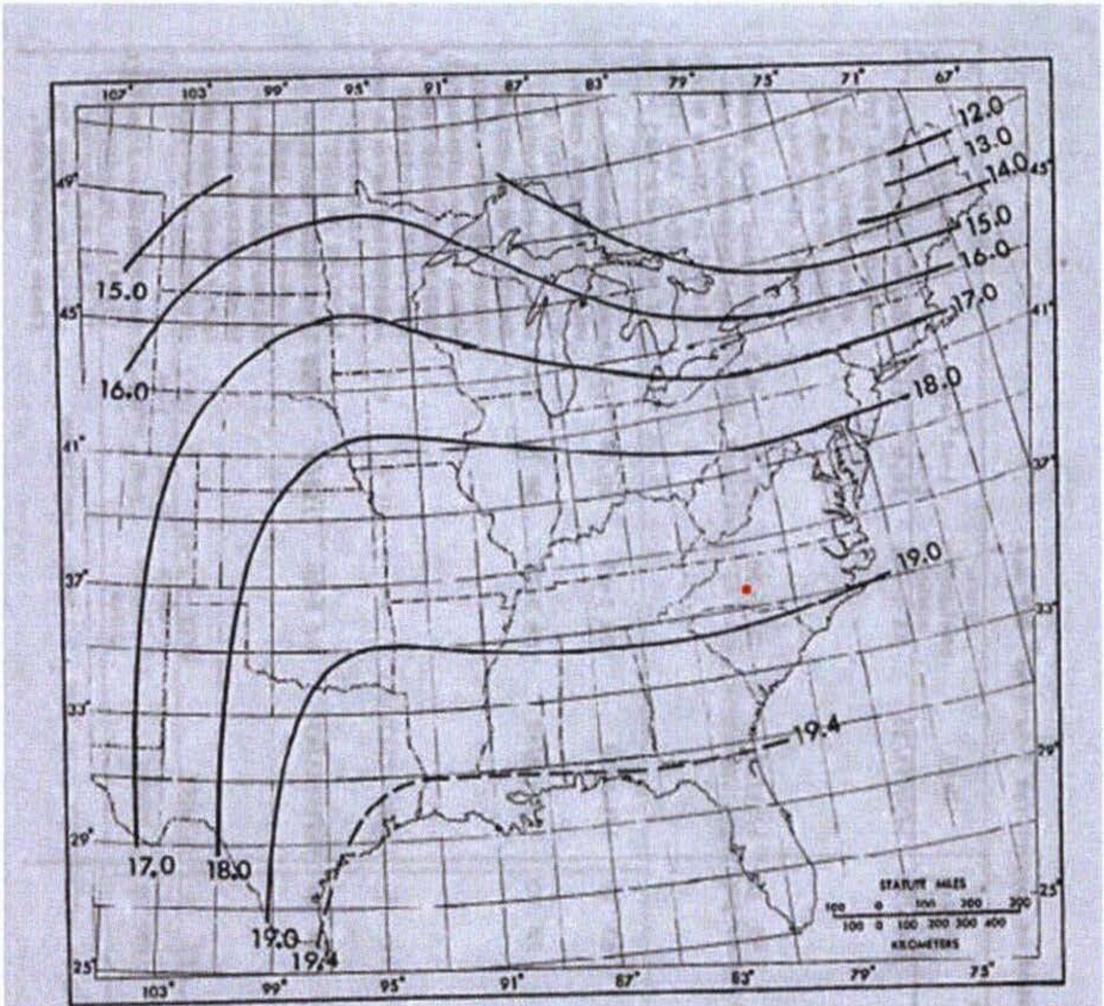


Figure 24.—1-hr 1-mi² PMP analysis based on figure 23 and 6-hr 10-mi² precipitation from HMR No. 51.

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

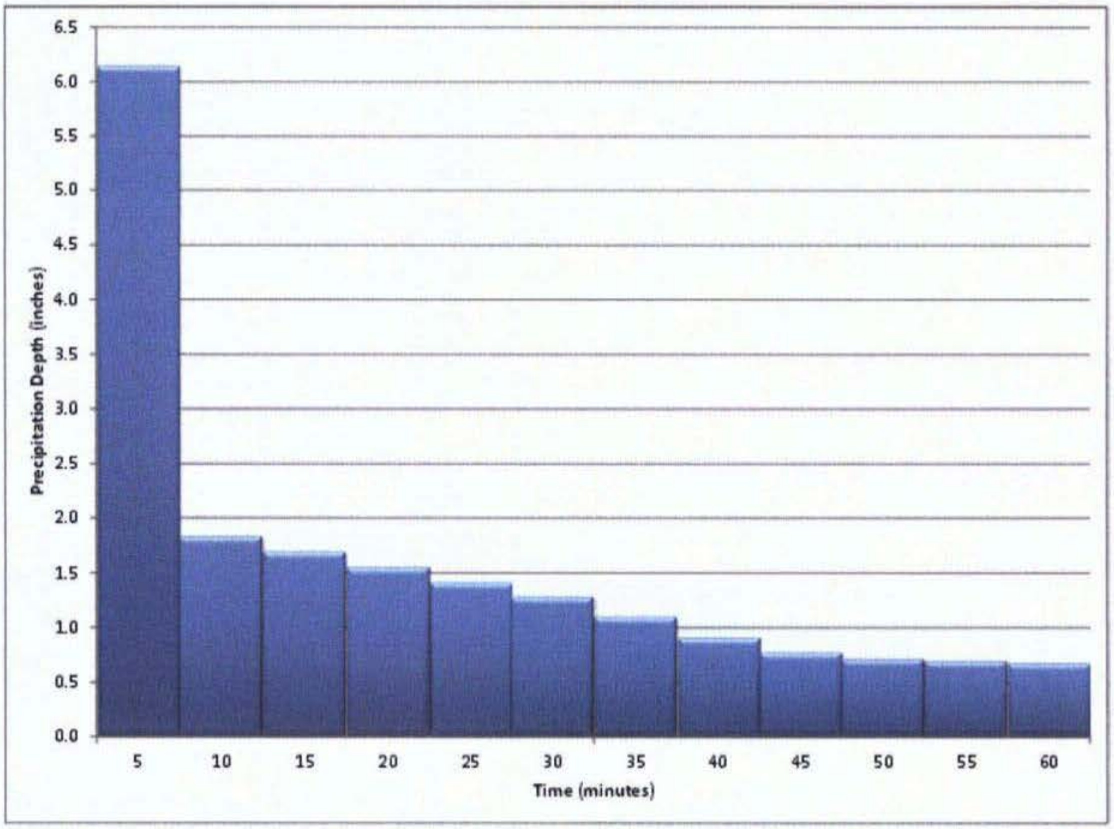
	1-mi² Point Rainfall			
	5-min	15-min	30-min	1-hr
PMP (in.)	6.1	9.7	13.9	18.8
Ratio to 1-hr PMP	*0.327*	*0.515*	*0.741*	*1.0-*

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 58). 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 MNS PMP

Time (minutes)	Incremental Precipitation (inches)
0	0
5	6.15
10	1.84
15	1.70
20	1.55
25	1.41
30	1.28
35	1.10
40	0.91
45	0.77
50	0.72
55	0.70
60	0.68
Total	18.8

FIGURE 2.1.1-2
MNS 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 MNS. ICM is a fully integrated 1-D and 2-D hydrodynamic model which allows for a more appropriate hydraulic simulation (versus a channelized 1-D approximation model) of the relatively flat topography found on the MNS site. One-dimensional 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 MNS 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 to be 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 MNS site-specific aerial photography and LiDAR survey data dated 2013 (Reference 17). The 2-D Zone defining the ICM model was extended to include the entire MNS Yard area and defined by the perimeter security barrier and the [redacted] ft msl Intake/Discharge Dike to the north. (b)(3):16 U.S.C. § 824o-1(d), (b) An additional area to the west adjacent to the Cask Storage yard was also included to model potential surface runoff impacts. The digital data was processed in ESRI ArcGIS software to create a triangulated irregular network (TIN) file, which was imported into the ICM software (ICM) (Reference 17). The survey data 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 MNS was developed using engineering drawings provided by Duke Energy.

Model boundary conditions were assigned to locations that would not impact flow calculations in areas of the MNS 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.

Including but not limited to the reactor, auxiliary, and turbine buildings, all buildings within the 2-D Zone 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 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).

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 the 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*, 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).

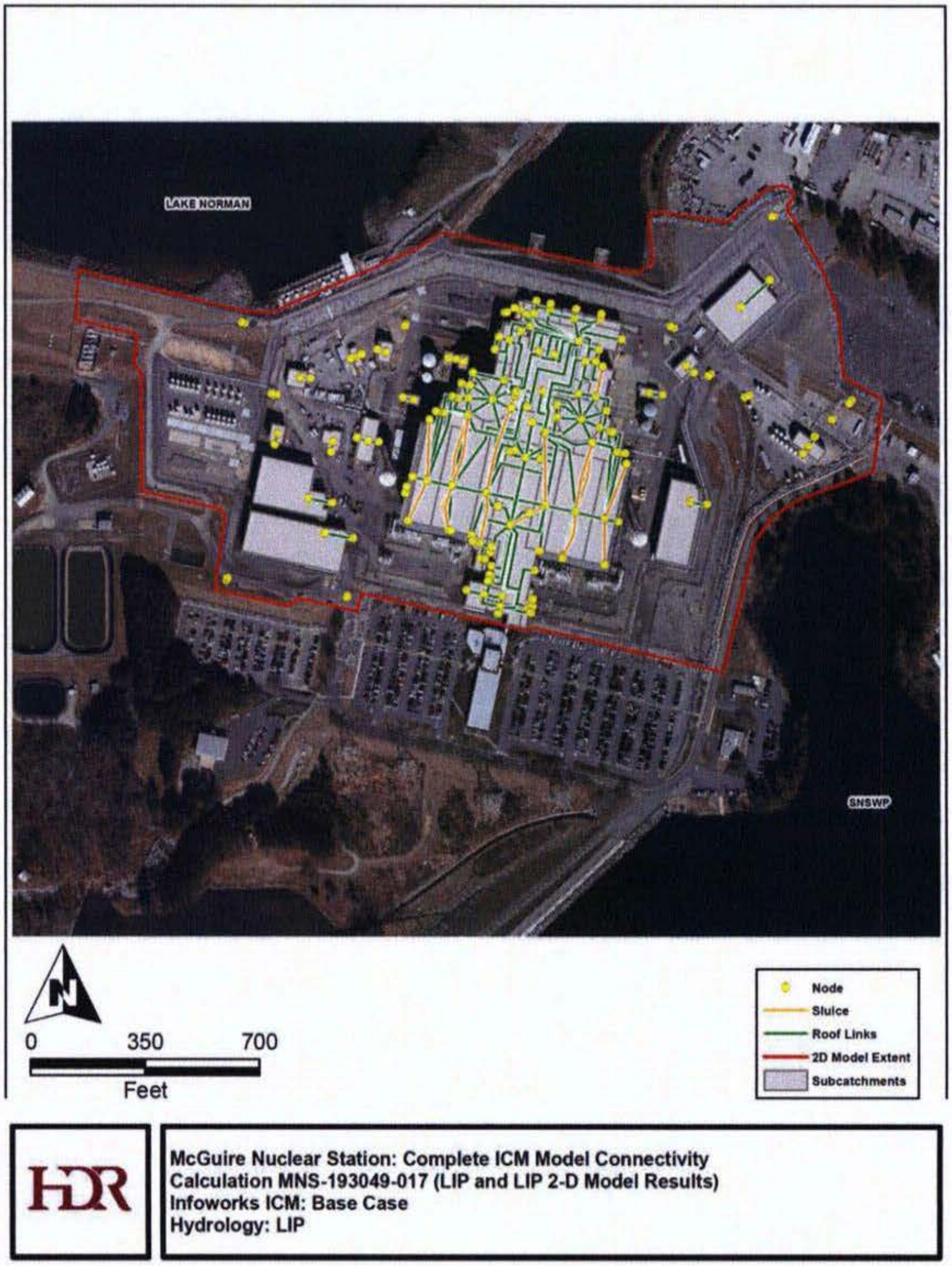
FIGURE 2.1.2-1
MNS DIGITAL TERRAIN MODEL EXTENTS SHOWN BY RED LINE

HDR

McGuire Nuclear Station: 2D Model Extent
Calculation MNS-193049-017 (LIP and LIP 2-D Model Results)
Infoworks ICM: Base Case
Hydrology: LIP

FIGURE 2.1.2-2
ICM MODEL OUTPUT MNS DIGITAL TERRAIN MODEL 3-D PERSPECTIVE LOOKING NORTHEAST. NO BUILDINGS SHOWN

FIGURE 2.1.2-3
MNS DIGITAL TERRAIN ROOF DRAINAGE CONNECTIVITY



Overland flow surface roughness 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
Riprap	0.036
Concrete/Asphalt	0.013

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 used as voids when creating the mesh. The resulting mesh contains 69,285 triangles and 59,513 elements.

FIGURE 2.1.2-4
MNS DIGITAL TERRAIN SURFACE ROUGHNESS ZONES

HDR

McGuire Nuclear Station: Roughness Zones over 2D Model Extent
Calculation MNS-193049-017 (LIP and LIP 2-D Model Results)
Infoworks ICM: Base Case
Hydrology: LIP

2.1.3 Security Barrier Model Features

Changes to the local site topography and support buildings within the security barrier have taken place since original construction and have been accounted for in the January 2013 aerial survey (Reference 17) and included in the ICM-site model.

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

2.1.4 MNS ICM Model Results

The evaluation of potential flooding on the MNS 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 2.1.1-2. 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 3-hour simulation to evaluate the PMP maximum flooding effects within the MNS Yard under existing modeled site characteristics. Runoff from the roofs of building was directly added to the 2-D mesh as noted in Section 2.1.2.

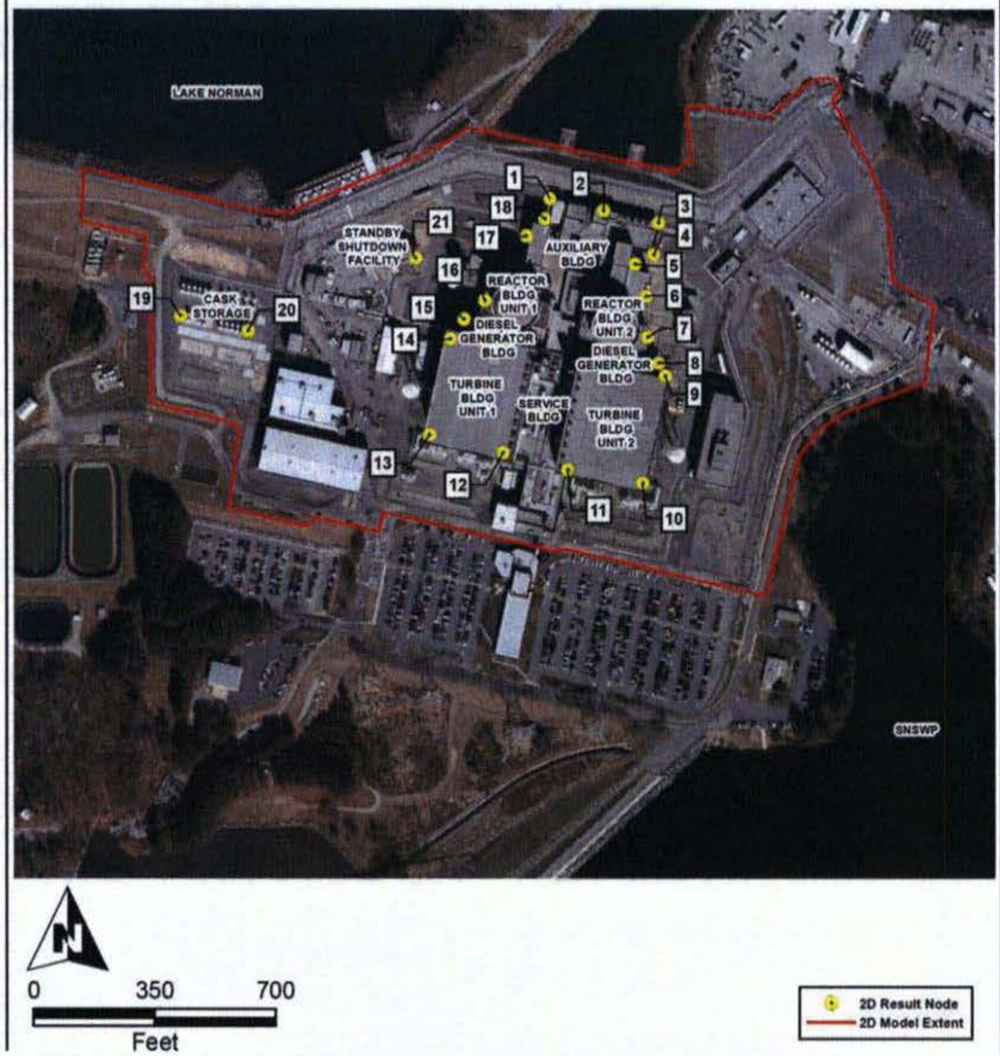
The 2-D modeled effects of the LIP result in variable water surface elevations modeled across the entire MNS Yard. Results of the LIP modeling are presented at 21 defined node points generated at an offset distance surrounding the perimeter of the main complex, near the Cask Storage area on the western portion of the MNS Yard, and near the Standby Shutdown Facility, in order to define locations of interest around the MNS power block as shown in Figure 2.1.4-1. Water surface elevation and depth hydrographs as well as velocity hydrographs and inundation duration estimates at each location for the LIP simulation were exported from the ICM model. Figure 2.1.4-1 shows the location of each result node, and Table 2.1.4-1 provides the North Carolina State Plane NAD83 Coordinate System Northing and Easting in U.S. survey feet along with a brief description of the location. Water surface elevation and depth hydrographs as well as velocity hydrographs at each node location for the LIP simulation are provided in MNS-193049-017, Rev 0 (Reference 17). 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 at the time when flood depths fall below 0.2 ft). This provides a consistent method of inundation duration calculation between all locations.

A representative maximum water surface elevation level in the MNS Yard around the main complex (i.e., Auxiliary, Reactor, and Turbine Buildings) is approximately 761.1 ft msl (SOF 2.1.4-01).

A representative average maximum water surface elevation level of inundation in the MNS Yard around the Cask Storage area is approximately 757.1 ft msl (SOF 2.1.4-02).

A representative average maximum water surface elevation level of inundation near the Standby Shutdown Facility is approximately 761.0 ft msl (SOF 2.1.4-03).

FIGURE 2.1.4-1
LIP MODEL FLOOD INUNDATION LEVELS NODE POINTS IN THE MNS POWER BLOCK YARD (REFERENCE 17)



McGuire Nuclear Station: 2D Result Node Locations
Calculation MNS-193049-017 (LIP and LIP 2-D Model Results)
Infoworks ICM: Base Case
Hydrology: LIP

TABLE 2.1.4-1

LIP MODEL FLOOD INUNDATION LEVELS, VELOCITY AND DURATION AT POINTS IN THE MNS POWER BLOCK YARD (REFERENCE 17)

Node	Description	Northing	Easting	Maximum Depth, ft	Maximum Elevation, ft msl	Maximum Velocity, ft per second	Inundation Duration, Hrs:min
1	Auxiliary Building NW	618611.2608	1419617.426	0.4	761.0	2.2	1:02
2	Auxiliary Building	618576.8283	1419775.755	0.5	761.1	0.9	1:07
3	Auxiliary Building NE	618542.3958	1419934.085	0.4	760.9	1.8	1:07
4	Auxiliary Building SE	618452.2637	1419919.566	0.8	760.8	1.2	2:10
5	Fuel Building Unit 2	618426.4368	1419865.183	0.5	760.8	0.5	1:18
6	Equipment Staging Building	618335.2231	1419898.113	0.6	760.8	0.9	2:22
7	Unit 2 Doghouse	618220.0075	1419902.861	0.6	760.8	3.1	2:07
8	Diesel Generator Unit 2	618147.4863	1419934.587	0.7	760.7	0.4	2:29
9	Turbine Building 2 NE	618110.9513	1419953.292	0.5	760.7	0.4	1:31
10	Turbine Building 2 SE	617810.0233	1419887.182	0.2	760.6	0.6	0:37
11	Turbine Building 2 SW	617847.3289	1419670.766	0.5	760.6	0.5	1:17
12	Turbine Building 1 SE	617895.2459	1419479.262	0.6	760.8	0.1	1:47
13	Turbine Building 1 SW	617945.5954	1419267.17	0.4	760.8	1.2	1:35
14	Turbine Building 1 NW	618215.0141	1419325.929	0.7	760.9	0.5	1:40
15	Diesel Generator Unit 1	618271.0594	1419367.649	1.1	760.9	0.4	1:37
16	Unit 1 Doghouse	618324.0081	1419426.28	0.8	761.0	1.8	1:32
17	Fuel Building Unit 1	618504.4268	1419547.774	0.7	761.0	2.1	1:57
18	Waste Solidification Building	618553.5809	1419601.994	1.4	761.0	0.6	2:25
19	Cask Storage West	618280.3843	1418546.168	1.9	757.1	0.5	2:23
20	Cask Storage East	618238.6769	1418741.128	1.0	757.1	1.1	1:45
21	Standby Shutdown Facility	618444.3	1419227.3	0.8	761.0	0.5	1:35

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 reevaluation of the maximum flood that would cause adverse effects to the MNS site. By NRC definition, a maximum 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 structures, systems, and components (SSCs) important to the safety of a nuclear power plant." The MNS site is located directly downstream of Cowans Ford Dam on a plateau located above the east bank of the riverine segment forming the Cowans Ford Dam spillway tailrace. [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The Hydrometeorological inflow design flood standard imposed on the MNS site is the PMF associated with one of the Duke Energy dams/reservoirs adjoining or located upstream of the MNS site.

Section 5.5.1 of American Nuclear Society (ANS) 2.8, under "Hydrologic Dam Failures," states "...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." [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

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 floodwaters 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 is the basis for the MNS-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 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 17 sub-basins that comprise the Cowans Ford Dam drainage basin are shown in Figures 2.2-1 and 2.2-2.

FIGURE 2.2-1
COWANS FORD DEVELOPMENT HYDROLOGIC SUB-BASIN FROM 1992 FERC PMF STUDY

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

FIGURE 2.2-2

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

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 11 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 Lake James (Catawba River, Paddy Creek, and Linville River) through the Catawba River tailrace below Wylie Dam, including significant tributaries to the Catawba River/Duke Energy Development reservoirs (Reference 19). The main stem Catawba River Model length is approximately 165 miles.

The hydraulic model performance sections of the Catawba River Model specific to the MNS-2.1 Fukushima Flood Hazard Evaluation Study are (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) reservoirs and dams, and the immediate (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam areas including the upstream reservoir adjoining the MNS Intake Dike and downstream Catawba River adjoining the MNS-SNSWP Dam (Figure 1.1-1). However, the Catawba River Model extends downstream of the immediate (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

The 2013 Catawba River Model accounts for significant tributaries off the main stem of the Catawba River and within the seven 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.

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-sq-mi 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 (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] shown in Table 2.2-1 (Reference 19). 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 between Lake James and Lake Wylie. 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 2.2-1 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.

[redacted] (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 Cowans Ford PMF, July 1916, and August 1940 events are shown in Table 2.2-2.

TABLE 2.2-2
MODEL VERIFICATION SUB-BASIN PRECIPITATION COMPARISON

Sub-basin		Cowans Ford: Basin Mean Rainfall - Inches		
Number	Location	PMP	1916	1940
0	Johns River	21.29	18.51	12.74
1	Linville River	17.44	19.29	13.17
2	Lower Little River	26.99	11.37	11.72
3	Warrior Creek	27.10	18.83	12.70
4	Lower Creek	30.29	16.48	11.05
5	Middle Little River	28.80	13.28	12.13
6	North Fork Catawba River	24.11	19.94	12.51
7	Upper Little River	30.83	14.58	12.00
8	Gunpowder Creek	33.25	15.37	11.80
9	Lookout Shoals Direct	25.46	8.03	9.49
10	Rhodhiss Direct	30.32	12.18	10.99
11	James Direct	27.46	15.73	11.92
12	Oxford Direct	28.07	12.79	12.06
13	Cowans Ford Direct	11.11	5.30	5.37
14	Catawba River	22.12	13.40	11.47
15	Lyle Creek	22.16	9.42	9.04
16	Muddy Creek	24.82	11.47	9.98
17	South Fork Catawba River	13.28	9.81	8.30
18	Dutchman Creek	5.32	5.96	5.40
19	Mountain Island Direct	1.81	4.00	4.94
20	Long Creek	0.30	4.06	4.96
21	Wylie Direct	0.15	5.08	5.01

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

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

[redacted]

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

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

[redacted]

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 2.2-3.

The modeled results reflect good correlation with the observed conditions for both the July 1916 and August 1940 flood events. The results support the Soil Conservation Service (SCS) synthetic unit hydrograph developed in 1992 for the FERC-approved Catawba-Wateree HEC-1 Model and is sufficient to account for the hydraulic performance of the Cowans Ford PMF event used for the Fukushima 2.1 Flood Hazard Report (FHR). Additional details regarding the model verification are presented in References 19 and 27.

FIGURE 2.2-3

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 NRC requires the estimation of the design-basis flood that would cause conservative but realistic external flooding effects to the MNS site based on one or more combinations of dam failure, rainfall, or seismic conditions. The design-basis flood scenarios were developed consistent with NUREG/CR-7046 Sections 3.3, 3.4, and 3.9 and Appendices D and H (Reference 58). HDR developed four primary design-basis flood determination scenarios based on fair-weather dam failure events, combined effect events involving the half PMF and seismic dam failure (piping failure simulation), PMF non-failure, and PMF with dam failure. Each of the four primary scenarios involves the determination of whether or not sufficient downstream dam overtopping conditions are achieved in a realistic (with respect to physical and engineering principles) yet conservative manner to create potential downstream cascading dam failures, thereby, increasing the adverse impacts at MNS.

The following sections summarize use of the Catawba River Model to perform reservoir flooding evaluation for the MNS 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 MNS. 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 12). The FERC has established reservoir target elevations that are below normal full pond elevations for each of the Developments. External flooding evaluations for the MNS 2.1 Fukushima Study use the FERC reservoir target elevations as initial reservoir elevations. This assumption is consistent with MNS current licensing design basis analysis as described in former Appendix 2F to the MNS UFSAR.

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 57), the 2013 Catawba River Model [redacted]

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

No significant reservoir storage reduction has been included in FERC-licensed reservoir capacity due to sedimentation. The [redacted]

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

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 elevation [redacted]

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

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 12). 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 19) (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 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 Bridgewater 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

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

Wave impacts were added to determine if adequate freeboard is provided at the site. As noted in Section 4.4.1 of Calculation MNS-193049-018 Rev 0 (Reference 19), there was one potential half PMF plus seismic-induced dam failure event that was identified during the combined effects scenario review process.

The Cowans Ford drainage basin half PMF plus seismic dam failure model run case is considered the bounding flood simulation (downstream backwater flood impacts) for this NUREG/CR 7046 case and was compared to the full PMF flood simulation to determine the most adverse external flood impact at MNS.

The design-basis flood scenarios are developed consistent with NUREG/CR-7046, Sections 3.3, 3.4, and 3.9 and Appendices D and H (Reference 58).

Experience with existing FERC Catawba-Wateree PMF models was used in the HHA evaluation of flooding from upstream reservoirs through evaluation of the insignificant contribution of the 135 small dams in the Cowans Ford drainage basin and construction of PMF model inputs for the five reservoirs and dams (Reference 23) (SOF 2.2-03). Modeling using the Cowans Ford drainage basin PMFs was identified as the bounding rivers and stream flooding event through a series of model scenarios including the fair-weather and seismic failure plus half-PMP model runs. The selection of the Cowans Ford PMF using the HHA process is consistent with the existing licensing basis PMF analysis.

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

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.

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 using elliptical-shaped isohyetal patterns to maximize the rainfall over a given Duke Energy Development's basin. The MNS-2.1 Fukushima Flood Hazard Evaluation Study employs the FERC-approved Catawba-Wateree

Legacy HMR52-HEC-1 model (Reference 37) to reevaluate the MNS site for flooding from rivers and streams (external flood event).

2.2.1 Probable Maximum Flood – Lake Norman

The flood inundation bounding PMF, non-failure modeling scenario was determined by modeling (Reference 19). Consideration was given to determining if the FERC-Cowans Ford PMF or Cowans Ford PMF with antecedent storm produced the highest Lake Norman reservoir elevation and highest Catawba River elevation at MNS-SNSWP Dam. In addition, overtopping potential from the precipitation event was evaluated for each dam and will be discussed in Section 2.3. [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

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

- The starting reservoir elevation for all Duke Energy dams is based on FERC target reservoir elevations.
- A one [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam
- A [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dams.
- [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) with top of gate elevation at [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft msl.

The 2013 Cowans Ford Dam PMF with antecedent storm 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 hours of full PMP

The resulting sub-basin inflow hydrographs are shown in Figure 2.2.1-1 and detailed in Calculation MNS-193049-018 Rev 0 (Reference 19).

Forty (40) percent of the PMP was used versus a 500-yr rainfall based on comparison of National Oceanic and Atmospheric Administration (NOAA) Atlas 14-point precipitation rainfall 72-hour totals (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) and the 40-percent PMP values for each dam sub-basin. In several locations, the 500-yr precipitation total was approximately equal to or greater than 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) Dam PMF inflow model scenario CF_ACS_PMF_1B4 shown in Figure 2.2.1-1. These modeling results do not assume a failure at any upstream dam. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The SNSWP Dam inundation is within the established armored section of the embankment for this scenario with no dam failure.

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

FIGURE 2.2.1-1

SUB-BASIN INFLOW HYDROGRAPHS FOR [redacted] PMF SCENARIO CF_ACS_PMF_1B4

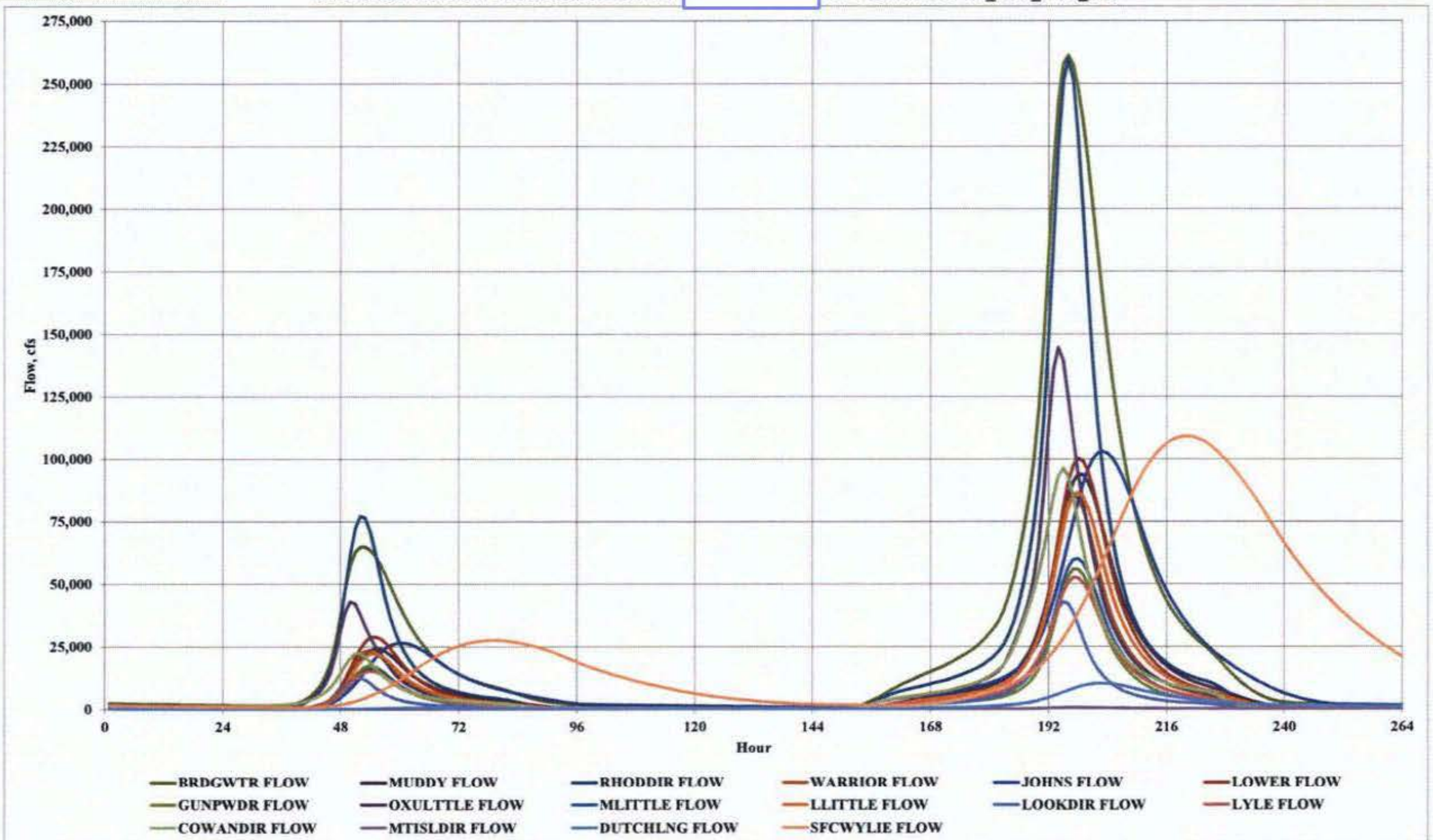


TABLE 2.2.1-1

CATAWBA RIVER MODEL RUN CF_ACS_PMF_1B4 RESULTS (REFERENCE 19)

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

[redacted]

FIGURE 2.2.1-2

LAKE NORMAN STAGE NEAR MNS SITE (REFERENCE 19) WITH PEAK ELEVATION [redacted] FT MSL

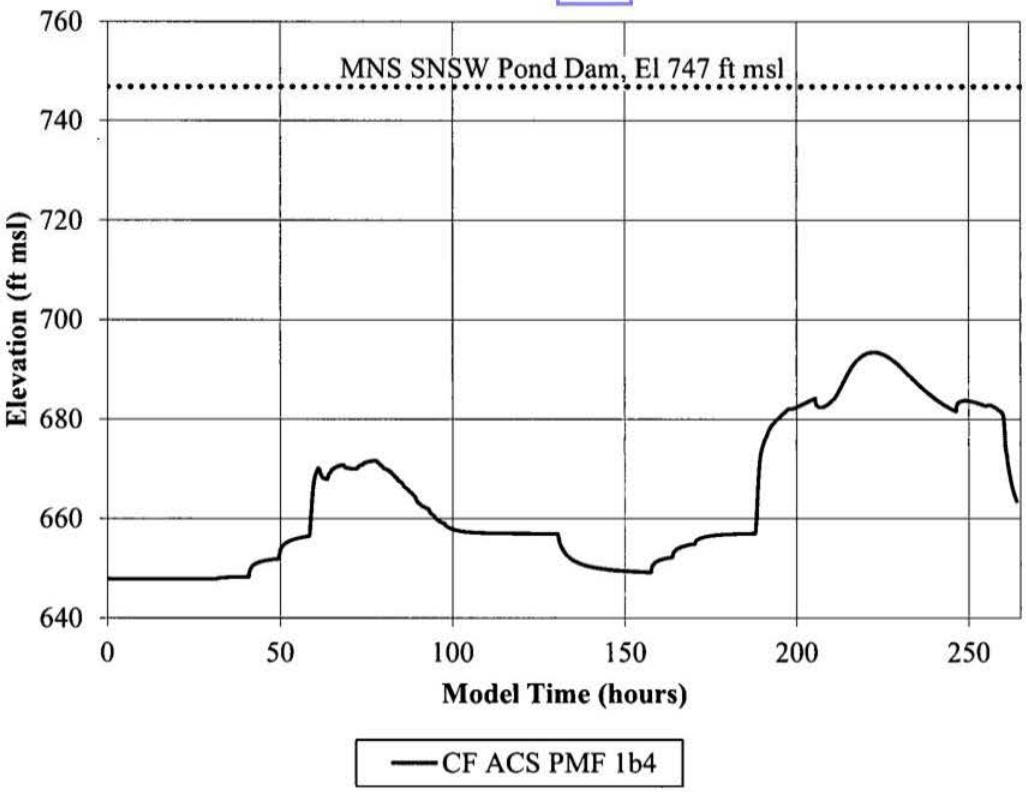
(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

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

[redacted]

FIGURE 2.2.1-3
CATAWBA RIVER STAGE NEAR MNS SNSWP DAM SITE (REFERENCE 19) WITH PEAK ELEVATION [redacted] FT MSL

(b)(3):16 U.S.C. § 824o-1(d), (b)



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 (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam to determine if other non-Duke Energy dams have the potential to impact both the Duke Energy hydroelectric dams and the MNS 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 as the key parameter of interest. For the screening analysis, HDR calculated cumulative volumes of all small dams upstream of each Duke Energy Development along the Catawba River (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) These small dam volumes were used to estimate the maximum possible increases in reservoir water surface elevations assuming the entire volume was instantaneously added to each reservoir (Reference 23). 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 MNS site. Each Development could easily pass the cumulative floods from the upstream breaches. If the entire cumulative volume of all small dams were instantaneously added to Lake Norman and assuming the lake was full, the Lake Norman level would rise from 760 ft msl to approximately 761 ft msl, conservatively assuming no spillway discharge, well below (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) ft msl. It is highly improbable that all 127 dams would fail during a single event. 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 analysis flood at MNS.

Duke Energy Hydroelectric Development Breach Parameters:

HDR developed overtopping and piping failure breach parameters for the Duke Energy Developments using regression methodologies for earth embankments to estimate:

- Bottom breach width,
- Top width,
- Average breach width with upper and lower limits,
- Average failure time with upper and lower limits, and
- Average breach discharge with upper and lower limits.

The breach parameter development details and results are presented in Calculation MNS-193049-011 (Reference 64).

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 FERC and are maintained to standards required by "18 CFR Subpart 12." These dams have been reviewed for stability associated with all requirements of the FERC and are considered well-maintained safe structures as documented in annual FERC inspections. All Duke Energy Developments have been remediated to safely pass their respective FERC PMF's. PMF remediation includes PMF flood wall extensions, rock-tendon anchors, sheet pile extensions in dike earth embankments, and spillway capacity increase (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The respective developments are typically comprised of concrete gravity sections (bulkheads and spillways), powerhouse structures, and compacted earth fill embankments/dikes. Regression equations are available that utilize dam features and reservoir storage volume to estimate dam breach parameters. The regression methodologies chosen to support the development of earth embankment 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 57). 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. Earth dam breach parameters used in the HEC-RAS model are shown in Table 2.3.1-1.

Overtopping failures of concrete bulkhead sections were developed 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) using existing FERC dam structure stability analyses with consideration given to the FERC PMF peak reservoir elevation at each dam (Reference 19). Spillways, powerhouses, and bulkheads were remediated to safely pass FERC PMF inflows, and (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Concrete bulkhead assumed breach sections (Reference 64) at [redacted] and are modeled to undergo rapid failure (0.1 hours), consistent with FERC-accepted failure criteria (Reference 14). Concrete dam breach parameters used in the HEC-RAS model are shown in Table 2.3.1-1.

TABLE 2.3.1-1

FINAL DAM BREACH PARAMETERS FOR PIPING AND OVERTOPPING MODE FAILURES AT DUKE ENERGY HYDROELECTRIC DAMS (REFERENCE 19)

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

2.3.2 Dam Failure Permutations

The PMF with dam failure scenarios developed for the Fukushima 2.1 flood hazards evaluation utilize a single PMF event that was determined through multiple simulations to produce the greatest potential for adverse impacts at MNS (Section 2.3.1). Simulation results from the PMF with non-failure scenarios were used to determine the significant PMF event that was utilized for the final dam failure scenarios (Reference 19). PMF evaluation criteria included the water surface elevation adjoining the MNS site, upstream reservoir storage capacity, and any upstream dam overtopping during the non-failure simulation of the PMF.

Numerous trial model runs were developed prior to the selection of the final model runs presented in Table 2.3.2-1. The purpose of the trial runs was to determine the potential range of reservoir elevations adjoining the upstream MNS embankment along with the Catawba River elevations adjoining the MNS-SNSWP Dam based on variability in breach parameters and breach locations at [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam. The model runs also considered the impact of variability in breach parameters and breach locations at [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

The Catawba River Model's 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 MNS-193049-018 Rev 0 (Reference 19) includes modeling details for sensitivity model runs developed to test dam failure outcomes at the MNS 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 MNS site is shown in Table 2.3.2-1.

TABLE 2.3.2-1
HEC-RAS MODEL FINAL RUN MATRIX (REFERENCE 19)

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

[redacted]

The external flooding evaluation at MNS, MNS-193049-018 Rev 0, completed as part of the NRC-Phase I, 10CFR50.54 (f) order for Fukushima NTTF Recommendation 2.1, determined the design-basis flood is associated with model run CF ACS PMF 8j4. The 1-D unsteady flow HEC-RAS model (Catawba River Model) developed for the Catawba River System was used to determine a realistic but conservative bounding design-basis flood. [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

The 1-D model stage hydrograph in Figure 2.3.2-1 illustrates the overtopping duration of the [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The stage hydrograph shown in Figure 2.3.2-1 is used as the boundary condition in the Innovyze ICM 2-D Model described in Calculation MNS-193049-019, MNS Site Analysis of Combined Effects (Reference 18) to determine potential impacts to the MNS Yard and Cask Storage Yard immediately adjoining the [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam. The stage hydrograph results adjoining the MNS Intake Dike and MNS-SNSW Pond Dam for the PMF non-failure event (CF ACS PMF 1b4) and PMF with failure event (CF ACS PMF 8j4) are shown in Figures 2.3.2-2 and 2.3.2-3, respectively. The stage difference shown in Figure 2.3.2-2 represents the inflow contribution of the upstream dam failures at [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam and [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam. The stage difference in Figure 2.3.2-3 represents the failure of [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Note The darkly shaded squares are part of the original document; no content has been redacted.

TABLE 2.3.2-2

FINAL RESULTS DESIGN-BASIS FLOODING NEAR MNS SITE)

Location	Zero Freeboard Crest Elevation (ft msl)	CF ACS PMF 8j4					
		Headwater Elevation (ft msl)	Tailwater Elevation (ft msl)	Discharge (cfs)	Model Time at Peak Headwater (hrs.)	Overtopping Duration (hrs.)	Freeboard (ft)
Bridgewater Dam	1220	1218.3		211,500	195.70	-	1.7
Rhodhiss Dam	1023.1	1023.8	981.3	503,500	207.33	9.38	(0.6)
Oxford Dam	959	962.1	905.1	829,500	208.15	5.55	(3.1)
Lookout Shoals Dam	866	866.0	821.2	823,500	204.80	0.05	0.0
Cowans Ford Dam	(b)(3):16 U.S.C. § 824o-1 (d), (b)(4), (b)(7)(F)		737.7	2,104,000	216.15	9.77	(3.5)
Cowans Ford East Embankment				1,833,000	216.15	9.77	(3.5)
Hicks Cross Roads Dike	(b)(3):16 U.S.C. § 824o-1 (d), (b)(4), (b)(7)(F)	778.6		76,000	216.38	9.82	(3.6)
West Rim Dike		778.6		76,000	216.40	9.90	(3.6)
Lake Norman Rim-Route 73 Highway	771.5	778.6		54,000	216.37	14.57	(7.1)
MNS-SNSW Dam	747		736.8		223.37	-	10.2
Mountain Island Dam	668.5	679.4	654.1	1,881,000	220.97	10.30	(10.9)

Notes:

[1] Table reflects the results from the HEC-RAS model for scenario CF_ACS_PMF 8j4 from Reference 19.

[2] Modeled reservoir elevations do not include impacts of wind-driven waves.

[3] Values in parentheses in "Freeboard" column indicate overtopping of zero freeboard crest elevation.

[4] Freeboard values shown in red indicate dam was overtopped, (negative freeboard).

FIGURE 2.3.2-1
LAKE NORMAN STAGE NEAR MNS SITE INTAKE STRUCTURE (REFERENCE 19)
LAKE NORMAN WATER LEVEL ABOVE [redacted] FT MSL FOR [redacted] HOURS PEAK STAGE [redacted] FT MSL

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

(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)

[redacted]

— CF ACS PMF 8j4

FIGURE 2.3.2-2

LAKE NORMAN STAGE NEAR MNS SITE (REFERENCE 19) UPPER AND LOWER BOUNDING DESIGN-BASIS FLOOD STAGE HYDROGRAPHS AT THE MNS [redacted] FT MSL INTAKE DIKE

(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)

[redacted]

CF ACS PMF 1b4 — CF ACS PMF 8j4

FIGURE 2.3.2-3
LAKE NORMAN STAGE NEAR MNS SITE (REFERENCE 19) UPPER AND LOWER BOUNDING DESIGN-BASIS FLOOD STAGE HYDROGRAPHS AT THE MNS-SNSWP DAM

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

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

FIGURE 2.3.2-4

CATAWBA RIVER WATER SURFACE PROFILE COMPARISON BETWEEN NORMAL POOL [redacted] **(REFERENCE 19)**

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

[redacted]

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

2.4 Storm Surge and Seiche

The MNS 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 Norman or other reservoirs in the Catawba River Basin. Therefore, 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), does not typically recommend consideration of water level increases caused by atmospheric pressure gradients; consideration of water level is typically limited to wave analysis (through forced sustained winds) and water setup (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 58).

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 ft. 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 50):

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

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.

The maximum water surface values shown Table 2.4.1-1 are not bounding. Bounding maximum water surface elevations for the MNS 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.

TABLE 2.4.1-1
SEICHING WIND-DRIVEN WAVE RESULTS

Site Location	Fetch Length (ft)	Condition	Reservoir Elevation (ft, msl)	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)
MNS Intake Fetch Line-1	27,530	Normal	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)	80	18	79.2	0.29	760.29
	27530	CF ACS PMF 8j4		98.5	16	34.6	0.04	778.54
Cowans Ford Embankment Fetch Line 2	20,587	Normal		80	14	79.9	0.22	760.22
	20,587	CF ACS PMF 8j4		98.5	14	34.8	0.03	778.53

The values for U_t were converted from the fastest mile design wind speed as noted in Section 2.0, MNS-193049-020 Rev 1 to the duration associated with the seiche period (Reference 61).

2.4.2 Wind-Driven Waves Analysis – Lake Norman Source

Wind-driven wave heights were developed using the Coastal Engineering Design and Analysis System (CEDAS) Automated Coastal Engineering Software (ACES) at six locations around the MNS site based on proximity to water bodies and topography (Figure 2.4.2-1). These locations are similar to site areas previously studied and reported in the MNS UFSAR. The analyses were developed using water surface elevations for Lake Norman presented in Table 2.4.2-1.

FIGURE 2.4.2-1
WAVE HEIGHT ANALYSIS STUDY LOCATIONS

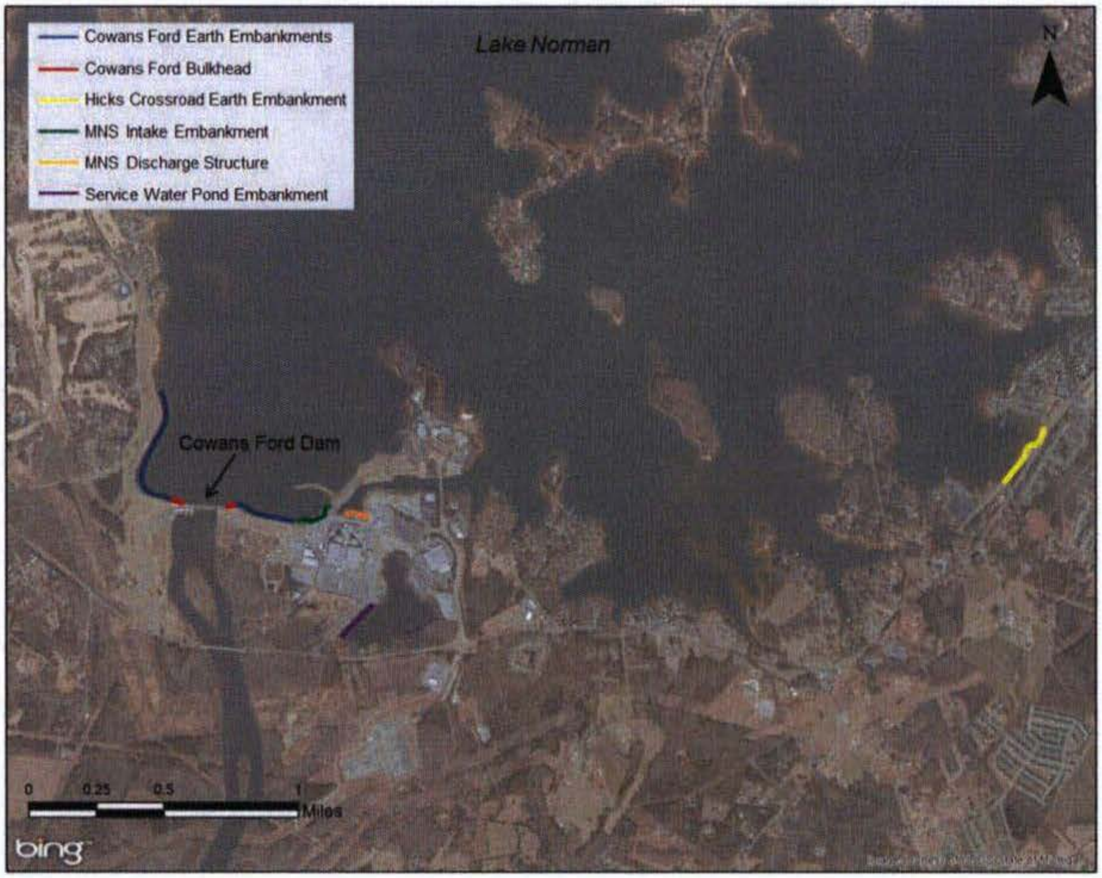


TABLE 2.4.2-1
WATER SURFACE ELEVATIONS USED IN CALCULATIONS

Location	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted]
1. Cowans Ford Earth Embankments	
2. Cowans Ford Bulkhead	
3. MNS [redacted] ft msl Intake Embankment	
4. Hicks Crossroads Earth Embankment	
5. MNS Discharge Structure	
6. SNSWP Embankment	

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

Locations 1-5 Reference 20 - MNS-193049-013-01 Rev 2.
NOTE: [1] Location 6 – PMF water elevation is based on PMP applied directly to the SNSWP drainage area (Reference 63).

The 96-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. 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. This wind speed is considered conservative since American Society of Civil Engineers (ASCE) (2005) (Reference 65) recommends a 3-second gust wind speed of 96 mph for each 100-year recurrence interval which corresponds to a 1-minute wind speed of 79 mph (MNS-193049-013-01 Rev 2 (Reference 20)).

A 40-mph overland wind speed was used for the PMF wave analysis. The American National Standard ANSI/ANS-2.8-1992 (Reference 1) shows the 2-year 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 (located within 20 miles of MNS) yielded a 2-year fastest mile, 10 m elevation wind speed of 32.9 mph (Reference 20). The 40-mph fastest mile wind speed was chosen 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 54). The 40-mph wind speed is representative of a 1.5-minute wind duration.

Results for the wind-driven wave analysis are provided in Table 2.4.2-2

TABLE 2.4.2-2
WIND-DRIVEN WAVE HEIGHT RESULTS

										ACES Output	
Location	**Condition**	**Design Windspeed (mph)**	**Averaging Duration of Design Wind Speed (min)**	**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)**	**Significant Wave Height, Hmo (ft)**	**Peak Wave Period, Tp (s)**
Cowans Ford Earth Embankments	Normal	96	1.0	27,650	80	20	79	22.5	20	7.1	4.4
	PMF	40	1.5	27,650	99	30	34	16.7	28	2.9	3.3
Cowans Ford Bulkhead	Normal	96	1.0	27,650	80	20	79	22.5	20	7.1	4.4
	PMF	40	1.5	27,650	99	30	34	16.7	28	2.9	3.3
(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)											
Hicks Crossroads Earth Embankment	Normal	96	1.0	12,602	20	10	81	16.9	12	4.4	3.3
	PMF	40	1.5	24,156	99	25	34	16.0	25	2.7	3.1
MNS Discharge Structure	Normal	NA*	1.0	NA*	NA*	NA*	NA*	NA*	NA*	NA*	NA*
	PMF	40	1.5	24,935	99	25	34	16.1	26	2.8	3.2
SNSWP Embankment	Normal	96	1.0	1,998	80	5	84	10.2	3	2.1	2.0
	PMF	40	1.5	1,998	86	5	36	7.4	5	0.9	1.4

*There is a negligible fetch length at this location during normal conditions and therefore no wave analysis was performed.

Note: The wind speeds shown in Table 2.4.2-2 under "Wind Speed for Wave Generation" are adjusted from the duration of the "Design Windspeed" to the duration used in the analysis as described in Section 4 of MNS-193049-013-01 Rev 2.

Sections of the [redacted] ft msl Intake and Discharge Dike were reviewed to determine possible wave overtopping and to estimate the amount of wave volume that could spill over the crest of the embankments. The wave spill over volume was approximated using a 1-D Boussinesq model (COULWAVE) to evaluate the overtopping during the PMF (Reference 21).

Impacts of wave action on the upstream slopes for [redacted] were reviewed using current state-of-the-practice methodologies outlined in Reference 22. Upstream riprap sizes for the [redacted] These structures will be overtopped by the static PMF level; therefore, combined wind effects were not evaluated. [redacted] therefore, were not evaluated for that wind-driven wave effect. During the PMF plus 2-year wind speeds combination, the static reservoir level is above the protective land and structural features; therefore, they were evaluated for this combined effect and found to have undersized rock slope protection. However, there is a large concrete cable trench structure located in the crest of the [redacted] msl dike that runs the length of the dike that would provide margin to minimize wave action erosion through the crest of the dike. The section of the [redacted] ft msl was found to have adequate upstream slope protection for both the wind wave cases. Also, the SNSWP Dam was found to have adequate sized riprap for both applicable wind wave cases (SOF 2.4.2-01).

2.5 Tsunami

The MNS 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. MNS is located inland (more than 150 miles from the Atlantic coast) and not on a waterway that would be subjected to effects of a Tsunami.

2.6 Ice-Induced Flooding

The MNS 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 MNS.

2.6.1 Ice Effects

Long-term air temperature records available at the North Carolina State Climatology Office were reviewed to assess historical extreme air temperature variations at the MNS site. The analysis was also supported by onsite temperature data measured at the MNS site.

The climate at the MNS 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 48: http://www.sercc.com/cgi-bin/sercc/cliMAIN.pl?sc9350).

There has not been a recorded event of significant surface ice formation on Lake Norman or any of the other 10 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 United States consistently remain above freezing (Reference 49).

2.7 Channel Diversions

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. Due to the location of MNS immediately downstream of Cowans Ford Dam and the upstream and downstream topography of Lake Norman and Mountain Island Lake, channel diversion is not a credible flooding event. The SNSWP was constructed in a small tributary to the main channel of Mountain Island Lake and is protected from scour by topographic features.

2.8 Combined Effects

Section 9 of ANS 2.8 outlines general criteria to be reviewed for addressing combined flood-causing events. 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 size of the storage and discharge capacity at (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] rainfall events less than the PMF did not produce the bounding flooding levels at Cowans Ford Dam and MNS. 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.

Lake Norman 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 Norman 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 MNS, 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 Norman 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 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. HDR calculations MNS-193049-018 Rev 0 (Reference 19) and MNS-1930492-019 Rev 0 (Reference 18) 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. The controlling external flooding event for the MNS site is an upstream dam failure during a combined effects PMF event (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] 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 (SOF 2.8-01). 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 precipitation flood events combined with potential failure of upstream dams and simple wind-driven waves.

2.8.1 MNS 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 [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Lake Norman peak elevations near the MNS site were determined using the Catawba River Model (HEC-RAS 1-D unsteady breach simulation).

The MNS [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft msl Intake Dike would not be overtopped due to the static PMF peak reservoir elevation and was evaluated for slope stability during the flood and post-flood event. The slope stability analysis included a rapid drawdown loading condition due to the short duration inundation of the upstream face of the dam by Lake Norman peak flood levels. The factors of safety-resisting sliding failures of the upstream and downstream slopes were greater than accepted standards of practice of the USACE and FERC indicating that no significant slope failure surface would be expected due to the combined effects PMF plus dam failure event. Details of the analysis are provided in calculation MNS-193049-022 Rev 0. (Reference 66).

Water surface elevation hydrographs at HEC-RAS cross-sections located near the MNS Intake Dike structure along with the Cowans Ford East Embankment were used in the ICM 2-D model for the MNS site (Section 2.1). Combined effects wind-driven wave impacts were applied to the upstream 2-D Boundary Condition representing the MNS [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft msl Intake Dike by approximating the wave splash over volume (Reference 18). In addition, the section of the [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) was added as a flow-boundary condition to simulate overtopping flows in this area and determine possible SSC impacts in the MNS Yard resulting in a combined effects assessment.

The 2-D Zone boundary condition used in modeling the LIP described in Section 2.1 was modified along the MNS Intake Dike by inserting a Lake Norman stage hydrograph from the Cowans Ford Dam failure HEC-RAS model Run CF ACS PMF 8j4 described in Section 2.3. A modeling period of 24 hours was selected from review of the Catawba River Model output for cross-sections near the MNS site. Figure 2.3.2-1 shows the upstream boundary condition stage hydrograph from the Catawba River Model using scenario CF ACS PMF 8j4 (SOF 2.8.1-02). During the model simulation, ICM calculates the combined effects overtopping and wind-driven wave splash over flows (broad crested weir relationship) resulting from the boundary condition stage hydrograph.

Figure 2.8.1-1 provides the location of the 2-D boundary condition inputs along the upstream 2-D Zone boundary. The upstream 2-D boundary conditions were placed from points A to B to C as shown in Figure 2.8.1-1 in order to assure that the maximum potential flooding occurs within the model. Points A – B start at the [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The additional wave run-up combined effects wave splash over was evaluated for segment B – C.

FIGURE 2.8.1-1
2-D BOUNDARY CONDITION LOCATION (REFERENCE 18)

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

HDR

McGuire Nuclear Station: 2D Boundary Condition
Calculation MNS-193049-019 (Combined Effects Model Results)
Infoworks ICM: Base Case
Hydrology: Combined Effects

FIGURE 2.8.1-2

APPROXIMATE EMBANKMENT CREST PROFILE NEAR 2-D EAST EMBANKMENT BOUNDARY CONDITION SHOWING AREA MODELED THROUGH COWANS FORD EMBANKMENT CREST (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) **FT MSL, NEAR THE A-B BOUNDARY CONDITION**

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

FIGURE 2.8.1-3

APPROXIMATE EMBANKMENT CREST PROFILE NEAR 2-D MNS INTAKE BOUNDARY CONDITION SHOWING A CREST ELEVATION ABOVE [redacted] FT MSL, NEAR THE B-C BOUNDARY CONDITION

(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)

[redacted]

The combined effects of wind-driven waves and the peak PMF static reservoir elevation were evaluated by reviewing Cowans Ford Dam structures that would be impacted by upstream waves. Fetch distances for the peak reservoir elevation and the resulting wind-driven waves were estimated in MNS-193049-013-01 and 02 (References 20 and 21). The contribution from wind-driven waves was evaluated for embankment structures that were above the static peak reservoir level of [redacted] ft msl. This was limited to the MNS [redacted] ft msl Intake and Discharge Dike. The contribution from wind-driven wave run-up was determined using a wave numerical model (COULWAVE) (Reference 21). COULWAVE was used to calculate the transformation of open-water waves from Lake Norman propagating over the MNS Intake and Discharge Dike crests for the PMF with upstream dam failure condition (Reference 18).

(b)(3):16 U.S.C. § 824o-1(d), (b)

The combined effects flooding 2-D simulation results in variable water surface elevations across the MNS Yard. Three primary inundation locations in the MNS Yard were identified and compared to the LIP inundation levels: the perimeter around the main complex (including the diesel generator area for Unit 1), the Cask Storage area in the western portion of the Yard, and the Standby Shutdown Facility. The most significant inundation was found on the western side of the MNS Yard with minimal inundation on the eastern side.

In summary:

- A representative maximum water surface elevation level of inundation in the MNS Yard around the main complex (i.e., Auxiliary, Reactor, and Turbine Buildings) ranges from approximately 760.0 ft msl to 760.7 ft msl. The maximum inundation elevation from the LIP was 761.1 ft msl (Table 2.1.4-1).

- A representative average maximum water surface elevation level of inundation in the MNS Yard around the Cask Storage area is approximately 757.6 ft msl. The maximum inundation elevation from the LIP was 757.1 ft msl (Table 2.1.4-1).

- A representative maximum water surface elevation level at the Standby Shutdown Facility is approximately 760.6 ft msl. The maximum inundation elevation from the LIP was 761.0 ft msl (Table 2.1.4-1).

- In general, the combined effects flooding of the MNS Yard is less than the LIP flooding evaluated in MNS-193049-017 Rev 0 (Reference 17).

- Wave splash over impacts on the riprap covered downstream face of the MNS Intake and Discharge Dike was evaluated and the current rock slope protection is adequate for the calculated wave splash over.

- Inundation of the Cask Storage area would be expected but would last less than 12.25 hours. Depths and velocities would not be expected to move the Casks or foundations.

- Flow patterns along the roadway/embankment that forms the west boundary of the Cask Storage area produce velocities ranging from approximately 1 foot per second (fps) to 6 fps with the greater velocities at the toe of the upstream embankment. The relatively short duration (~4 hours) of the highest range of the velocities is not anticipated to result in erosion that would impact the foundations of the Cask Storage concrete foundations.

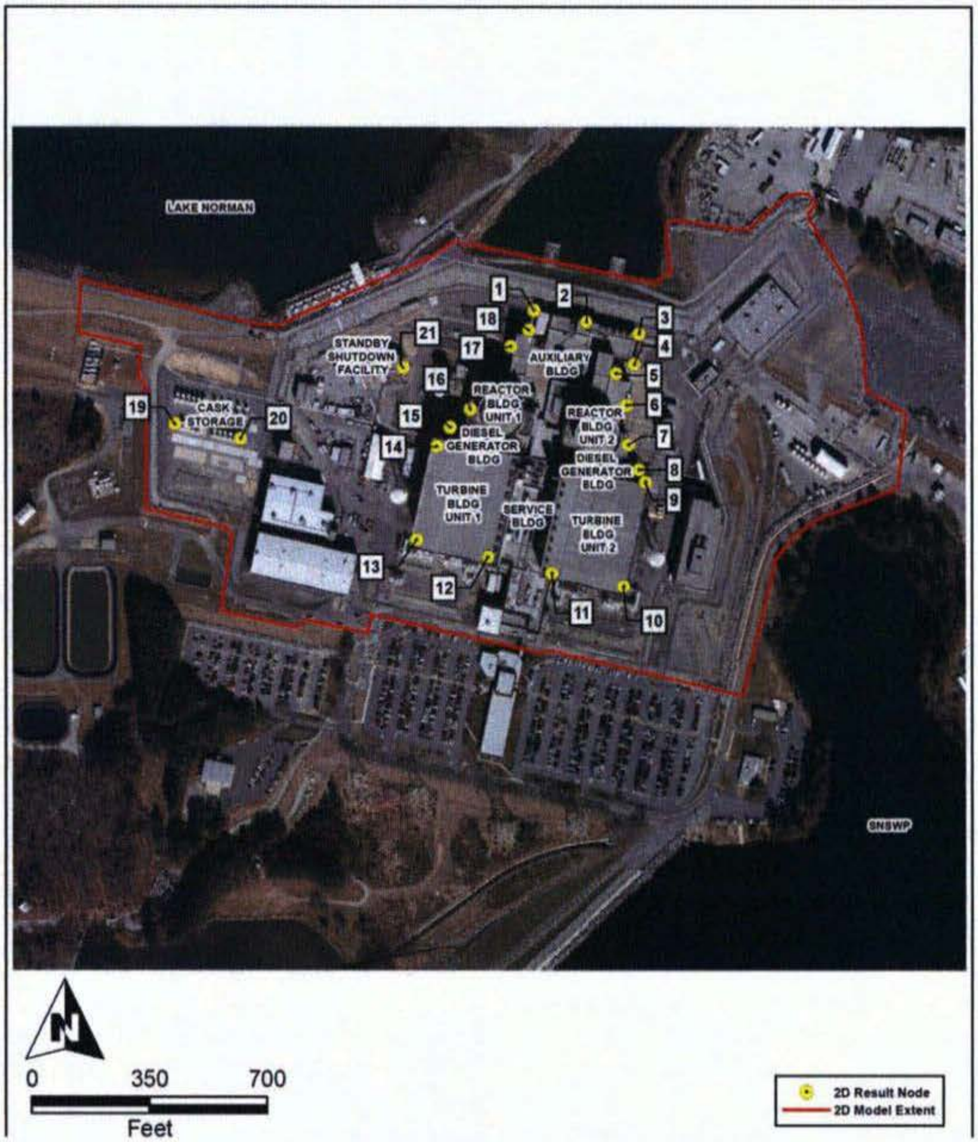
Results of the 2-D modeling are presented at 21 defined node points generated at an offset distance surrounding the perimeter of the main complex, near the Cask Storage area on the western portion of the MNS Yard, and near the Standby Shutdown Facility, in order to define locations of interest around the MNS power block as shown in Figure 2.1.4-1. Water surface elevation and depth hydrographs as well as velocity hydrographs and inundation duration estimates at each location for the combined effects simulation were exported from the ICM model. Figure 2.8.1-4 shows the location of each result node, and Table 2.8.1-1 provides the North Carolina State Plane NAD83 Coordinate System Northing and Easting in U.S. survey feet along with a brief description of the location. Water surface elevation and depth hydrographs as well as velocity hydrographs at each node location for the combined effects simulation are provided in MNS-193049-019, Rev 0 (Reference 18). 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 at the time when flood depths fall below 0.2 ft). This provides a consistent method of inundation duration calculation between all locations.

[redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) This assumption produced higher modeled Lake Norman PMF elevations due to less breach outflow area. [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) msl dike upstream (Figure 2.8.1-2) of the Cask Storage Yard (Figure 2.8.1-4) is covered with fescue grass on the downstream face and riprap on the upstream slope. [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

This recommendation is based on post-Hurricane Katrina levee overtopping erosion studies performed by Briaud for the USACE (Reference 70). [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

FIGURE 2.8.1-4

2-D BOUNDARY CONDITION LOCATION (REFERENCE 18)



HDR

McGuire Nuclear Station: 2D Result Node Locations
Calculation MNS-193049-019 (Combined Effects Model Results)
Infoworks ICM: Base Case
Hydrology: Combined Effects

TABLE 2.8.1-1

COMBINED EFFECTS 2-D MODEL RESULTS WITH LOCATION OF EACH NODE IN NORTH CAROLINA STATE PLANE NAD83 COORDINATE SYSTEM NORTHING AND EASTING ALONG WITH A BRIEF DESCRIPTION OF THE LOCATION, MAXIMUM INUNDATION DEPTH, ELEVATION, VELOCITY AND DURATION

Node	Description	Northing	Easting	Maximum Depth, ft	Maximum Elevation, ft msl	Maximum Velocity, ft per second	Inundation Duration Hrs:min
1	Auxiliary Building NW	618611.2608	1419617.426	0.0	760.7	0.0	-
2	Auxiliary Building	618576.8283	1419775.755	0.1	760.6	0.3	-
3	Auxiliary Building NE	618542.3958	1419934.085	0.0	760.5	0.1	-
4	Auxiliary Building SE	618452.2637	1419919.566	0.0	760.0	0.0	-
5	Fuel Building Unit 2	618426.4368	1419865.183	0.0	760.3	0.0	-
6	Equipment Staging Building	618335.2231	1419898.113	0.0	760.2	0.0	-
7	Unit 2 Doghouse	618220.0075	1419902.861	0.0	760.2	0.0	0:15
8	Diesel Generator Unit 2	618147.4863	1419934.587	0.0	760.1	0.0	0:15
9	Turbine Building 2 NE	618110.9513	1419953.292	0.0	760.3	0.0	0:15
10	Turbine Building 2 SE	617810.0233	1419887.182	0.0	760.4	0.0	-
11	Turbine Building 2 SW	617847.3289	1419670.766	0.0	760.1	0.0	0:15
12	Turbine Building 1 SE	617895.2459	1419479.262	0.3	760.5	0.3	4:12
13	Turbine Building 1 SW	617945.5954	1419267.17	0.1	760.5	1.0	1:49
14	Turbine Building 1 NW	618215.0141	1419325.929	0.4	760.6	0.2	3:21
15	Diesel Generator Unit 1	618271.0594	1419367.649	0.8	760.6	0.8	7:39
16	Unit 1 Doghouse	618324.0081	1419426.28	0.5	760.6	0.4	5:11
17	Fuel Building Unit 1	618504.4268	1419547.774	0.3	760.7	0.5	4:24
18	Waste Solidification Building	618553.5809	1419601.994	1.0	760.7	0.3	6:43
19	Cask Storage West	618280.3843	1418546.168	2.4	757.6	1.1	9:50
20	Cask Storage East	618238.6769	1418741.128	1.5	757.6	1.3	12:13
21	Standby Shutdown Facility	618444.3	1419227.3	0.5	760.6	0.1	6:29

Note 1: Table from calculation MNS-193049-019, Reference 18.

The upstream and downstream slopes of the [redacted] Dam and the MNS [redacted] ft msl Dike were evaluated for wave action. [redacted] The MNS [redacted] ft msl Intake and Discharge Dike, directly upstream of the power block would not be overtopped by the static PMF water levels. The upstream and downstream slopes are covered with riprap (nominal 6 to 12 inches based on photographs) (SOF 2.8.1-03). [redacted] However, there is a large concrete cable trench structure located in the crest of the [redacted] ft msl Dike that runs the length of the structure that would provide margin to minimize wave action erosion over the short period where the PMF water levels are at their peak and would not be expected to cause breaching through the upstream crest of the dike and the concrete trench structure.

Overtopping wave splash over slope velocities developed in the 2-D model were reviewed and found to be less than 1.5 fps for the stone-faced downstream slope of the MNS [redacted] ft msl Dike. Depth of water on the downstream slope due to periodic wave splash over was simulated using the 2-D model and was approximately 2 inches. The wave splash over impacts were evaluated in calculation MNS-192049-013-03 Rev. 4 and the rock slope protection was found to be adequate for protecting the slope (SOF 2.8.1-04). As noted previously, water levels in the MNS Yard were evaluated for inundation due to the splash over along the MNS [redacted] ft msl Dike and the overtopping [redacted] ft msl. The contribution from the splash over combined with the overtopping resulted in inundation levels less than determined for the LIP event for all areas except the Cask Storage Yard (Reference 18).

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

LIP floods combined with wind-driven waves (Reference 63). The peak pond elevation assuming no spillway discharge was [redacted] ft msl providing [redacted] ft of freeboard margin. Wind-driven wave run-up of [redacted] ft conservatively added directly to the peak elevation would result in (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) (SOF 2.8.2-01). 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 Mountain Island Lake and not on the MNS site.

The SNSWP Dam was evaluated for slope stability post-flood event for a rapid drawdown loading condition due to the short duration inundation of the downstream face of the dam. The factors of safety-resisting sliding failures of the downstream slope were greater than accepted standards of practice of the USACE and FERC indicating that no significant slope failure surface would be expected due to the combined effects PMF plus dam failure event. Details of the analysis are provided in calculation MNS-193049-012 Rev 0 (Reference 26).

Potential for significant surface erosion on the downstream face of the SNSWP Dam due to inundation from the combined PMF spillway discharge from (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam and breach discharges from embankment failures was reviewed for the reevaluation. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The backwater velocity approaching the MNS-SNSWP Dam from the Catawba River would be substantially less than the left overbank velocity due to the natural hillside and valley topography that exists between the river and the toe of the MNS-SNSWP Dam (Digital Terrain Model Figure 2.1.2-2 – Reference 18). (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) is not expected to result from flow velocities less than the 3 fps to 5 fps simulated for (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 riprap covered toe and grass covered embankments are expected to protect the downstream slope from significant erosion due to non-scouring flows produced from backwater inundation. The riprap and grass protected slopes should be adequate for the short duration of flooding simulated with the HEC-RAS model based on accepted flow ranges for channels (Reference 69).

Section 3

Comparison of Current Design Basis and Reevaluated Flood Causing Mechanisms

Table 3-1 below summarizes the comparison of current licensing design basis and reevaluated flood causing mechanisms, which includes wind effect for flooding in reservoirs and dam failures. The table outlines a comparison for current licensing design basis and reevaluated flood causing mechanisms for the MNS site including the dam failure mechanism that considers the postulated PMF overtopping breach failure of Cowans Ford Dam to be the bounding dam failure event.

TABLE 3-1
CURRENT LICENSING DESIGN BASIS AND REEVALUATION FLOOD ELEVATIONS

Flood Causing Mechanism	Current Licensing Design Basis Flood Elevation	Reevaluation Analysis Maximum Flood Elevation	Reevaluation Analysis Flood Delta from Current Licensing Design Basis
Local Intense Precipitation	760.375 ft msl	761.1ft msl [1]	+0.725 ft.
Flooding in Reservoirs	767.9 ft msl	777.9 [2] ft msl	+10.0 ft.
Dam Failures	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)		
Storm Surge and Seiche/Wind-Wave Run-up	774.75 ft msl	778.54 ft msl	+3.79 ft
SNSWP Flooding	746.9 ft msl	746.8 [4]	-0.1 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 MNS Yard	–[5]	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b) ft msl	(b)(3):16 U.S.C. § 824o-1 (d), (b) ft
Combined effects downstream at SNSWP Dam	(b)(3):16 U.S.C. § 824o-1(d) ft msl	ft msl	ft

Notes:

[1] Location of recorded maximum Yard elevation is based on no active catch basins at Node 2 Table 2.1.4-1.

[2] Location of recorded maximum inundation elevation is shown for Cowans Ford Dam upstream of MNS assuming no upstream dam failures – Scenario CF_ACS_PMF_1b4.

[3] Bounding dam failure scenario is (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)

[4] Assumed no discharge from SNSWP spillway (Reference 63).

[5] Licensing Design Basis did not produce a combined effects flooding of the 760 ft MNS Yard.

3.1 SNSWP Dam

There were no significant differences between the current licensing design basis and reevaluation analysis flood causing mechanism results for the SNSWP Dam for precipitation-based flooding and wind-driven waves. There are no significant changes in the drainage area upstream of the pond that impacted sources of runoff and assumed hydrologic parameters. There is adequate freeboard provided by the design assuming no discharge from the SNSWP during the PMF. Failure of the SNSWP Dam would not result in flooding of the MNS Yard due to the orientation of the dam and the topography around the site.

One area of significant difference between the current licensing design basis and the reevaluation analysis is with the determination of the Catawba River PMF and combined effects flooding. The current licensing design basis hydraulic model did not produce a PMF failure by overtopping of (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Dam. This is a significant difference in the analysis and simulated inundation level on the downstream face of the SNSWP Dam between the current licensing design basis and the Fukushima 2.1 flood hazard reevaluation.

The reevaluated inundation analysis of the downstream slope of the SNSWP Dam by the PMF combined effects event (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The SNSWP Dam was evaluated for slope stability post-flood event for a rapid drawdown loading condition due to the short duration inundation of the downstream face of the dam. The factors of safety resisting sliding of the downstream slope were greater than accepted standards of practice of the USACE and FERC.

3.2 Riverine Flooding on Lake Norman

3.2.1 Flooding on Lake Norman

As outlined in Section 2.2, there was a difference in modeling performed for the current licensing design basis compared to the reevaluation analysis. There are a number of reasons for this including the advancements in hydrologic and hydraulic modeling since the early 1970s. The reevaluation used hydrology and hydraulic methods that produced different results for the PMF. Smaller precipitation events evaluated including fair-weather upstream dam failures and the half PMF with dam failures show better correlation between modeled reservoir elevations at Cowans Ford 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 current licensing 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 current licensing design basis flooding analysis was not significantly lower than the reevaluation analysis considering all inputs and modeling capabilities.

3.2.2 PMF Bounding Event

The bounding reevaluation flooding analysis event was found to be [redacted]

(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 reevaluation analysis determined the bounding case through a series of model runs following a similar methodology as used for the current licensing design basis. However, the revaluation analysis found that (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The current licensing design basis bounding flood and the reevaluation flood hazard analysis did not exceed the inundation levels in the MNS Yard produced by the LIP.

3.2.3 Probable Maximum Flooding

NUREG 7046 requires application of HMR51/52 (HMR applicable to the MNS site) to develop the PMP for the evaluation of the PMF (Reference 58). Previous licensing basis PMP-PMF analyses used the July 1916 regional storm 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 all modeled dams. The combined effects PMP event for the Cowans Ford Dam drainage area (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) The current licensing design basis river flood analyses did not result in any upstream dam overtopping failure due to a rainfall event.

The reservoir routing method described in the MNS UFSAR appears to be based on "level pool," storage assumptions including analysis of Lake Norman reservoir levels. This method of hydraulic analysis was common standard practice at the time when the modeling was performed. Because the reservoirs upstream of Lake Norman are river-shaped with storage tributaries, "fingers," located along the length of the reservoirs, consideration of backwater impacts along these reservoir lengths is necessary when performing hydraulic 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.

3.3 Local Intense Precipitation

The current licensing design basis LIP was developed using a 6-hour duration. The 6-hour event included a peak 1-hour rainfall of 14.7 inches and a total of 30 inches over the full 6 hours. The licensing design LIP was applied to the site using the rational method for stormwater drainage design.

When comparing the Fukushima 2.1 flood hazard reevaluation 2-D modeling analysis to the MNS licensing basis rational method 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 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 MNS Yard more appropriately than the simple rational method and 1-D sheet flow approach. 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 MNS Yard near critical structures was approximate 0.75 ft higher for the reevaluation.

3.4 Dam Failures

Upstream dam failures were assumed in the current 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. Neither of the modeling analyses found fair weather failures to result in cascading failures and concluded this flooding event was not bounding.

Both analyses considered combined effects flooding assuming a seismic failure of an upstream dam during a half PMF event. The current licensing design basis analyses found [redacted] to be the MNS site bounding external flooding event. This was not due [redacted] Dam as all half PMF plus dam failure simulations performed for the current licensing design basis, and the reevaluation resulted in no more than a [redacted] ft surcharge on Lake Norman. The current licensing design basis analysis only considered a [redacted] The reevaluation analyses using the HEC-RAS unsteady flow model did not find any combination of seismic dam failures plus half PMF flooding to bound site flooding impacts at MNS.

As noted in Sections 2.2 and 2.3, the reevaluation analysis followed guidelines in ANS 2.8 Section 9 to determine the bounding external flooding impacts at MNS. Requirements of Section 9.2.2.1, Alternative I, were identified as the bounding MNS site-flooding event resulting in an upstream water elevation of [redacted] ft msl with [redacted] ft of freeboard margin for the MNS [redacted] ft msl Intake Dike.

[redacted] msl dike upstream (Figure 2.8.1-2) of the Cask Storage Yard (Figure 2.8.1-4) is covered with fescue grass on the downstream face and riprap on the upstream slope. Due to the range of velocities simulated in the 2-D model, [redacted]

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

the MNS Yard would generally be less than the LIP event except for the Cask Storage yard as previously noted.

(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)

[redacted]

3.5 Storm Surge and Seiche

Storm surge and seiche were addressed in the current 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 MNS site as described in Section 2.4. The geographic location of the Catawba River combined with the highly regulated river reaches, 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 Norman and found to produce small wave impacts less than derived using wind-driven waves outlined in Section 2.4.2.

Hurricane wind wave impacts were evaluated for the dam structures using similar approaches in the current licensing design basis and the reevaluation flood hazard analysis. The hurricane wind produced the greatest wave height; but based on the combined effects alternatives (ANS 2.8 Section 9.2) reviewed for the MNS site, the precipitation plus 2-year wind-driven wave alternative was judged to be the most significant possible MNS site flooding event. The

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

[redacted]

3.6 Tsunami

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

3.7 Ice-Induced Flooding

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

3.8 Channel Diversion

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

3.9 Combined Effects

Combined effects flooding reviewed in the current licensing design basis and reevaluation analysis include applicable sections of ANS 2.8 Section 9.2. The current licensing design basis identified the combined [redacted] The reevaluation analysis identified [redacted] The stage hydrographs were used with the MNS site 2-D model and consideration of 2-yr mean wind speed waves to simulate flooding at the MNS Site and inundation (Section 2.8).

The current licensing design basis analysis did not find a river flooding event that resulted in inundation of the MNS Yard. The reevaluation analysis found the PMF combined effects alternative to have a peak reservoir elevation approximately [redacted] ft below the crest of the MNS [redacted] ft msl Dike.

Adding the 2-year wind-driven waves to the flood hazard reevaluation analysis peak reservoir elevation produced [redacted] These depths are generally lower than the LIP depths modeled for the flood hazard reevaluation (Table 2.1.4-1).

The highest MNS Yard water level associated with water (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] Dam was found around the Cask Storage Yard (Node 19) at [redacted] ft based on the assumption that yard catch basins are not functional. (b)(3):16 U.S.C. § 824o-1(d), (b)

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

[redacted]

Section 4

Interim Actions Taken or Planned

4.1 LIP Actions

Strategies to address specific water ingress areas are included in detail in Attachment 2.

4.2 Combined Effects PMF Flooding

Strategies to address specific water ingress areas are included in detail in Attachment 2.

Section 5

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Statement of Fact Validation

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	**Statement of Fact**	**Reviewer Name**	**Basis for Concurrence**	**Date**
SOF 1.1-01	Lake Norman forms the tailwater of Lookout Shoals Dam, located 34 miles upstream from Cowans Ford, and Mountain Island Lake forms the tailwater for Cowans Ford. Mountain Island Dam is located approximately 15 miles downstream from Cowans Ford (SOF 1.1-01).	Tim Banta	MNS UFSAR Section 2.4.1.2	JCEy 1/24/14
SOF 1.1-02	The maximum modeled Cowans Ford spillway discharge tailwater elevation is 698.50 ft msl, which is 61.50 ft below the McGuire yard elevation (SOF 1.1-02).	Tim Banta	MNS UFSAR Section 2.4.1.1	JCEy 1/24/14
SOF 1.1-03	Other safety-related structures located adjacent to the Yard include the Standby Nuclear Service Water Pond (SNSWP) with crest elevation of embankment of 747 ft msl and full pond elevation of [redacted] ft msl (SOF 1.1-03).	Tim Banta	MNS Former Appendix 2G Section 6 page 2G-A-1	JCEy 1/24/14
SOF 1.2.1-01	Roof drains designed to discharge 5 inches per hour have been installed on all safety-related buildings (SOF 1.2.1-01).	Tim Banta	MNS UFSAR Section 2.4.10, page 2.4-14	JCEy 1/24/14
SOF 1.2.1-02	The roof loading due to the maximum accumulation of water does not exceed the current licensing design basis loading for any portion of the Auxiliary Building roof (SOF 1.2.1-02).	Tim Banta	MNS UFSAR Section 2.4.10, page 2.4-15	JCEy 1/24/14
SOF 1.2.1-03	The buried storm drainage system is designed to remove precipitation of up to 4 inches per hour with additional precipitation ponding in the plant yard or overflowing the plant yard perimeter by sheet flow. Considerable storage of precipitation results from the 1-foot differential between the plant yard high points	Tim Banta	MNS UFSAR Section 2.4.10, page 2.4-15	JCEy 1/24/14

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

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	Statement of Fact	Reviewer Name	Basis for Concurrence	Date
	and ridge lines at elevation 760 ft msl and the top of the catch basins at elevation 759 ft msl. This creates pockets of storage around the plant yard, which have a capacity of approximately 155,000 cubic feet (SOF 1.2.1-03).			
SOF 1.2.1-04	Runoff is routed away from the plant buildings toward the catch basins with a minimum design ground slope of 1.4 percent. Although the yard drainage system, itself, was not designed to discharge the PMP, the system has been evaluated to ensure that the inundation of water due to PMP will not endanger any safety-related facilities (SOF 1.2.1-04).	Tim Banta	MNS UFSAR Section 2.4.10, page 2.4-15	JCEy 1/24/14
SOF 1.2.1-05	The floodwater elevation due to a LIP was evaluated by applying the rational method of six 1-hour rainfall intensities. These intensities ranged from 2.4 inches per hour to 14.7 inches per hour and arranged in the following order: 2.4, 2.4, 3.6, 14.7, 4.5 and 2.4 inches for a total of 30 inches of precipitation over a 6-hr period (SOF 1.2.1-05).	Tim Banta	MNS UFSAR Section 2.4.10 , page 2.4.16 and Table 2-23	JCEy 1/24/14
SOF 1.2.1-06	The rational method was applied to route the rainfall across the site. Two methods were used to analyze the effects of excess water backup on the structures. The first method of analysis assumed that there was perimeter runoff and that the storm drainage system was operating at one-half of its total capacity. This accounted for any debris or obstacles partially blocking the drain system. Using this method of analysis, the water was estimated to pond to an elevation of 760.28 (SOF 1.2.1-06) ft msl (Reference 6).	Tim Banta	MNS 11/15/12 - 2.3 Flood Walkdown Report page 5	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	**Statement of Fact**	**Reviewer Name**	**Basis for Concurrence**	**Date**
SOF 1.2.1-07	This elevation is below the exterior doorway curbs of the safety-related structures. All exterior doorways are provided with curbs (or thresholds) at Elevation 760.5 ft msl (SOF 1.2.1-07) (Reference 6).	Tim Banta	MNS 11/15/12 - 2.3 Flood Walkdown Report page 5	JCEy 1/24/14
SOF 1.2.1-08	The second method of Yard inundation analysis assumed that the storm drainage system is completely inoperative or totally blocked and that the entire LIP runoff is discharged by sheet flow at the perimeter of the yard. The assumption was also made that the perimeter of the protected area would act as a weir for runoff to overflow the perimeter. Thus, when the quantity of flow from the PMP equaled the quantity of flow crossing the weir in a given period of time, equilibrium would be reached and the depth of ponding could be determined (level pool routing). With this method of analysis, some of the plant structures would act as obstructions to water flowing over the entire weir; therefore, the length of the weir was not assumed to be the entire distance around the plant but was divided into segments. These segments were estimated by reviewing flow paths through the Yard. Using this method, the water was estimated to pond to an elevation of 760.375 ft msl (SOF 1.2.1-08).	Tim Banta	MNS 11/15/12 - 2.3 Flood Walkdown Report page 6	JCEy 1/24/14
SOF 1.2.2.1-01	At full pond elevation 760 ft msl, Lake Norman has a surface area of approximately 32,339 acres (ac), a shoreline of approximately 603 mi, average depth of 33.5 ft and a volume of 1,093,600 acre-feet (ac-ft). Its total	Tim Banta	FERC CF STI Section 2 and hydro Relicensing documents at http://www.duke-energy.com/shoreline-management/catawba-wateree/lake-norman.asp.	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	**Statement of Fact**	**Reviewer Name**	**Basis for Concurrence**	**Date**
	watershed is approximately 1,790 square miles (sq mi) (SOF 1.2.2.1-01)) (References 32 and 67).		Reference #32 and duke website (comment in 2.1 CF/STI Section 2).	
(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted]		Tim Banta	Ref. 12 FERC relicensing Application Documents reporting usuable storage between 100 (full pond) and critical elevation Supporting file CWvolsummary11.xls	JCEy 1/24/14
SOF 1.2.2.1-03	On July 17, 1916, the Catawba River near the Cowans Ford Dam location reached an estimated flood discharge of 199,500 cfs (SOF 1.2.2.1-03).	Tim Banta	UFSAR 2.4.1.2	JCEy 1/24/14
SOF 1.2.2.1-04	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 current licensing design basis study, an infiltration rate of 0.10 inch per hour was	Tim Banta	MNS UFSAR 2.4.3.2	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	**Statement of Fact**	**Reviewer Name**	**Basis for Concurrence**	**Date**
	selected (SOF 1.2.2.1-04)			
(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted]		Tim Banta	Email from Brad Keaton, Duke Energy Hydro Operations Engineer, 12/11/2013.	JCEy 1/24/14
SOF 1.2.2.1-06	Considering the plant layout, the MNS site can be characterized as a "flood-dry site," as described in Section 5.1.3 of the American National Standard Report, *Determining Design Basis Flooding at Power Reactor Sites*, because the safety-related structures of the existing MNS are above spillway discharge flooding elevations (Reference 11). The Yard is nominally 760 ft msl and during the discharge of the licensing basis PMF through the Cowans Ford spillway, discharged water is not expected to backup significantly over the river elevation of approximately 698.5 ft msl (SOF 1.2.2.1-06). This meets the intent of the definition of a "flood-dry site."	Tim Banta	MNS UFSAR 2.4.1.1	JCEy 1/24/14
SOF 1.2.2.2-01	The Standby Nuclear Service Water Pond (SNSWP) is a nuclear safety-related impoundment constructed by placing a dam across a small tributary immediately south of the MNS Yard (Figure 1.2.2.1-2). Table 1.2.2.2-1 (SOF 1.2.2.2-01) provides pertinent	Tim Banta	FSAR Appendix 2G, Section 2G.2	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	**Statement of Fact**	**Reviewer Name**	**Basis for Concurrence**	**Date**
	information about the pond and dam.			
SOF 1.2.2.2-02	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*. Due to the small drainage area, the PMP (30.0 inches) (SOF 1.2.2.2-02) for a 10-sq-mi area and a 6-hr duration was used.	Tim Banta	FSAR Appendix 2G, Section 2G.3.1	JCEy 1/24/14
SOF 1.2.2.2-03	The 6-hr PMP was divided into an hourly temporal sequence which produced the greatest PMF for the basin. The first 0.5 inch of rainfall and 0.1 inch per hour were subtracted to provide for interception and infiltration based on USACE-EM 1110-2-1411, 1952, Plate 19. The first 6-hr incremental runoff values are presented in Table 1.2.2.2-2 (SOF 1.2.2.2-03).	Tim Banta	FSAR Appendix 2G, Section 2G.3.1	JCEy 1/24/14
SOF 1.2.2.2-04	A total of 48 hours was used for the analysis with the hourly values estimated using the graph in Figure 2, zone 6 (*Design of Small Dams*) multiplied by the PMP for 6 hours to determine the rainfall for 12, 24, and 48 hours. This produced a rainfall of 0.35 inch per hour for hours 7 through 12 and 0.1 inch per hour for hours 13 through 24 and 0.01 inch per hour for hours 25 through 48 (SOF 1.2.2.2-04).	Tim Banta	FSAR Appendix 2G, Section 2G.3.1	JCEy 1/24/14
SOF 1.2.2.2-05	A spillway rating curve was developed for the discharge from the weir structure and outlet pipe using methods described in the *Handbook of Hydraulics* (Reference 68). The discharge calculations considered control at the inlet weir discharge up to elevation 742.5 ft msl (320 cfs) and the discharge through the outlet pipe controlling above SNSWP elevation 742.5 ft	Tim Banta	FSAR Appendix 2G, Section 2G.3.1	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	Statement of Fact	Reviewer Name	Basis for Concurrence	Date
	msl (SOF 1.2.2.2-05).			
SOF 1.2.2.3-01	Due to the location of the MNS site on natural ground directly downstream of the Cowans Ford East Embankment, preconstruction groundwater levels were approximately 10 to 35 ft below plant yard grade of Elevation 760 ft msl (SOF 1.2.2.3-01).	Tim Banta	MNS UFSAR 2.4.13.5	JCEy 1/24/14
SOF 1.2.2.3-02	Since the zoned filter wall drain system is confined by building walls and the compacted earth backfill (or rock excavation at the foundation level), the wall drain system will remain passive during an earthquake as will the underdrain system. Since the top of the zoned wall filter is 5 ft below plant yard grade, there is no credible flood that will affect the underdrain system (SOF 1.2.2.3-02).	Tim Banta	MNS UFSAR 2.4.13.5	JCEy 1/24/14
SOF 1.2.2.3-03	The Nuclear Service Water System (NSWS) is a moderate energy fluid system and has been evaluated according to NRC Branch Technical Positions MEB 3-1 and APCSB 3-1. A through wall leakage crack, one-half the pipe diameter by one-half the wall thickness, would result in a flow of 666 gpm to the underdrain system. This flow plus the calculated groundwater seepage would result in a total flow of 696 gpm. Since six 250-gpm pumps are available to discharge groundwater, the postulated failure of the Nuclear Service Water pipe will not flood the underdrain system (SOF 1.2.2.3-03).	Tim Banta	MNS UFSAR 2.4.13.5	JCEy 1/24/14
SOF 1.2.3-01	Table 1.2.3-2 provides a list of the upstream	Tim Banta	CNS UFSAR, Table 2-51	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	**Statement of Fact**	**Reviewer Name**	**Basis for Concurrence**	**Date**
	dams and drainage areas in sq-mi for each reservoir (SOF 1.2.3-01).			
SOF 1.2.3-02	The seismic failure for each upstream dam was timed to coincide with the SPF Storm centered over its drainage area. At the hour in 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 MNS UFSAR Sections 2.4.4.2, 2.4.4.3, and 2.4.10 (Reference 11). The results of the test scenarios showing maximum reservoir elevations at each upstream reservoir are shown in Table 1.2.3-3 (SOF 1.2.3-02).	Tim Banta	CNS UFSAR, Table 2-64, MNS FSAR, Appendix 2F, Plate VI	JCEy 1/24/14
SOF 1.2.3-03	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) which is less than the peak PMF elevation of 767.9 ft msl; therefore, it was not the bounding case.	Tim Banta	MNS 2.3 Walkdown Report Rev 0 11/15/12 Reference 6	JCEy 1/24/14
SOF 1.2.3-04	The MNS yard and associated safety-related facilities are located at Elevation 760 ft msl which is above the maximum (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) msl (SOF 1.2.3-04) and river stage and, therefore, are protected from inundation damage as a result of the dam breach.	Tim Banta	MNS UFSAR 2.4.10	JCEy 1/24/14
SOF 1.2.3-05	The rising stage of the Catawba River will cause water to backup onto the downstream slope of the dam, but the expected backwater velocity is below the maximum permissible velocity for the existing in-place riprap and	Tim Banta	MNS UFSAR 2.4.10	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	**Statement of Fact**	**Reviewer Name**	**Basis for Concurrence**	**Date**
	grass. Fluctuation of the water surface will, therefore, not adversely affect the integrity of the SNSWP dam (SOF 1.2.3-05).			
SOF 1.2.4-01	The run-up associated with the breaking of significant and maximum waves caused by PMH is 9.41 ft and 11.92 ft, respectively, as shown in UFSAR Table 2-20 and Table 2-21. The most severe combination of surge (assumed to be setup), seiche, and wave run-up with Lake Norman at full pond (Elevation 760) results in water elevation of 774.75 ft msl for maximum waves and 772.24 ft msl for significant waves. [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Therefore, wave run-up presents no problems to any safety-related facilities (SOF 1.2.4-01).	Tim Banta	MNS UFSAR 2.4.5.6	JCEy 1/24/14
SOF 1.2.5-01	Tsunamis were never postulated to affect the site, and no flood elevation is given in the current licensing/design basis case basis of the plant. MNS 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.	Tim Banta	MNS UFSAR 2.4.6	JCEy 1/24/14
SOF 1.2.6-01	Ice-induced flooding was never postulated to affect the site, and no flood elevation is given in the current licensing/design basis case basis of the plant. The climate in the Catawba River	Tim Banta	MNS UFSAR 2.4.7	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	Statement of Fact	Reviewer Name	Basis for Concurrence	Date
	basin is moderate (minimum monthly mean water temperature for Lake Norman is in the low 40's) (SOF 1.2.6-01) and there has not been any recorded ice formation on the reservoirs in the river system.			
SOF 1.2.8-01	Combined flooding effects (PMP, PMF, dam failure and/or wind-driven waves) were reviewed for impacts at the MNS site. Maximum upstream water level elevation at the station occurs with the Cowans Ford Dam PMF as reported in Section 1.2.2.1. The maximum water surface elevations when combined with wind-driven waves are shown in Table 1.2.8-1 (SOF 1.2.8-01).	Tim Banta	MNS UFSAR 2.4.5, Table 2-20	JCEy 1/24/14
SOF 2.1.1-01	Using the PMP chart and the site location, the 1-hour, 1-mi^2 PMP estimate was determined to be 18.8 inches per hour (in/hr) (Reference 17) as illustrated in Figure 2.1.1-2 (SOF 2.1.1-01)	Tim Banta	MNS-193049-017 Rev 0	JCEy 1/24/14
SOF 2.1.1-02	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 18.8 in/hr. (Reference 17) (SOF 2.1.1-02)	Tim Banta	MNS-193049-017 Rev 0	JCEy 1/24/14
SOF 2.1.1-03	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).	Tim Banta	MNS-193049-017 Rev 0	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	Statement of Fact	Reviewer Name	Basis for Concurrence	Date
SOF 2.1.2-01	Roughness values for each material type are provided in Table 2.1.2-1 (SOF 2.1.2-01).	Tim Banta	MNS-193049-017 Rev 0	JCEy 1/24/14
SOF 2.1.2-02	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).	Tim Banta	MNS-193049-017 Rev 0	JCEy 1/24/14
SOF 2.1.2-03	Table 2.1.2-2 provides the associated Manning's *n* values used for each roughness zone (SOF 2.1.2-03).	Tim Banta	MNS-193049-017 Rev 0	JCEy 1/24/14
SOF 2.1.2-04	Figure 2.1.2-4 shows the model boundaries of each roughness zone (SOF 2.1.2-04).	Tim Banta		
SOF 2.1.4-01	A representative maximum water surface elevation level in the MNS Yard around the main complex (i.e., Auxiliary, Reactor, and Turbine Buildings) is approximately 761.1 ft msl (SOF 2.1.4-01).	Tim Banta	MNS-193049-017 Rev 0	JCEy 1/24/14
SOF 2.1.4-02	A representative average maximum water surface elevation level of inundation in the MNS Yard around the Cask Storage area is approximately 757.1 ft msl (SOF 2.1.4-02).	Tim Banta	MNS-193049-017 Rev 0	JCEy 1/24/14
SOF 2.1.4-03	A representative average maximum water surface elevation level of inundation near the Standby Shutdown Facility is approximately 761.0 ft msl (SOF 2.1.4-03).	Tim Banta	MNS-193049-017 Rev 0	JCEy 2/13/14
SOF 2.2-01	The Catawba River Model (HEC-1 and HEC-RAS) was used to verify the ability of the model unit hydrographs and routing parameters	Tim Banta	MNS-193049-018 Rev 0	JCEy 1/24/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	Statement of Fact	Reviewer Name	Basis for Concurrence	Date
	(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 2.2-1 were reconstructed from historic precipitation and runoff records. (SOF-2.2-01.)			
SOF 2.2-02	Storage and spillway capacity at each dam is adequate to discharge the upstream dam breach flow without causing overtopping at the downstream dams (Reference 19) (SOF 2.2-02).	Tim Banta	MNS-193049-018 Rev 0	JCEy 1/24/14
SOF 2.2-03	Experience with existing FERC Catawba-Wateree PMF models was used in the HHA evaluation of flooding from upstream reservoirs through evaluation of the insignificant contribution of the [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) (Reference 23) (SOF 2.2-03).	Tim Banta	MNS-193049-015 Rev 0	JCEy 1/24/14
SOF 2.2.1-01	The non-failure scenario that represented the most significant potential for combined PMF plus dam failure consequences at MNS was determined through modeling to be CF_ACS_PMF_1B4 defined as follows (SOF 2.2.1-01).	Tim Banta	MNS-193049-018 Rev 0	JCEy 1/24/14
SOF 2.4.2-01	Impacts of wave action on the upstream slopes for Normal (760 ft msl) and PMF [redacted] ft msl) water levels were reviewed using current state-of-the-practice methodologies outlined in Reference 22. Upstream rip-rap sizes for the	Tim Banta	MNS-193049-013-03 Rev 4	JCEy 2/13/14

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

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	**Statement of Fact**	**Reviewer Name**	**Basis for Concurrence**	**Date**
	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] The MNS [redacted] ft msl Intake Dike and MNS [redacted] ft msl Discharge Dike are protected at normal reservoir level plus hurricane wind by land and structural features therefore were not evaluated for that wind-driven wave effect. (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted] msl was found to have adequate upstream slope protection for both the wind wave cases. Also, the SNSWP Dam was found to have adequate sized riprap for both applicable wind wave cases (SOF 2.4.2-01).			
SOF 2.6.2-01	There are no recorded ice jam events in the upper reach of the Catawba River based on a	Tim Banta	Reference 49	JCEy 1/24/14

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

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	Statement of Fact	Reviewer Name	Basis for Concurrence	Date
	search of the USACE's *Ice Jam Database* (SOF 2.6.2-01). Water temperatures in this area of the southeast United States consistently remain above freezing (Reference 49).			
SOF 2.8-01	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 (SOF 2.8-01).	Tim Banta	MNS-193049-018 Rev 0	JCEy 1/24/14
SOF 2.8.1-01	This evaluation was performed for the critical combination of the [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) ft msl (SOF 2.8.1-01).	Tim Banta	MNS-193049-018 Rev 0	JCEy 1/24/14
SOF 2.8.1-02	Figure 2.3.2-1 shows the upstream boundary condition stage hydrograph from the Catawba River Model using scenario CF ACS PMF 8j4 (SOF 2.8.1-02).	Tim Banta	MNS-193049-018 Rev 0	JCEy 1/24/14
SOF 2.8.1-03	The MNS [redacted] ft msl Intake and Discharge Dike, directly upstream of the power block would not be overtopped by the static PMF water levels. The upstream and downstream slopes are covered with riprap (nominal 6 to 12 inches based on photographs) (SOF 2.8.1-03).	Tim Banta	MC-1022-13 Rev. 3 Photographs taken by M Hunt on 1/31/2014.	JCEy 1/24/14
SOF 2.8.1-04	The wave splash over impacts were evaluated in calculation MNS-192049-013-03 Rev. 4 and the rock slope protection was found to be adequate for protecting the slope (SOF 2.8.1-04).	Tim Banta	MNS-193049-013-03, Rev 4 Table 2	JCEy 2/13/14

Site: McGuire Nuclear Station		Section No.:		Author: HDR
SOF #	Statement of Fact	Reviewer Name	Basis for Concurrence	Date
2.8.2-01	Wind-driven wave run-up of 0.9 ft conservatively added directly to the peak elevation would result in 1.7 ft of freeboard margin (SOF 2.8.2-01).	Tim Banta	MNS-193049-013-01 Rev 2	JCEy 2/13/14

ATTACHMENT 2

McGuire Nuclear Station Flooding Interim Actions Taken or Planned

1.0 Introduction

As stated in Reference 1, Enclosure 2, Required Response Item 2, "*In accordance with the NRC's prioritization plan, within 1-to 3-years from the date of this information request, submit the Hazard Reevaluation Report. Include the interim action plan requested in item 1.d, if appropriate.*" Reference 1, Enclosure 2, item 1.d states that the licensee's Flooding Hazard Reevaluation Report (HRR) should contain the "*Interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment described below, if necessary.*" Duke Energy has determined that some flood levels included in the McGuire Nuclear Station (MNS) Flooding HRR contained in Attachment 1 of this submittal are not bounded by the Current Licensing Basis (CLB) flood levels for MNS Units 1 and 2. Therefore, in accordance with the above requirements, Attachment 2 of this submittal provides the MNS interim evaluation and actions taken or planned to address these non-bounded hazards relative to the MNS CLB.

The reevaluated non-bounded hazards in the MNS Flooding HRR have been entered into the MNS Corrective Action Program (CAP). These non-bounded hazards 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 non-bounded hazards do not represent errors in the current MNS flooding design or licensing basis. Therefore, consistent with Reference 5, the MNS flood hazard reevaluation results and the reevaluated non-bounded hazards do not call into question the operability or functionality of a MNS Structure, System, or Component (SSC) and they are not reportable pursuant to 10 CFR 50.72 and 10 CFR 50.73.

2.0 Local Intense Precipitation

The MNS reevaluated Local Intense Precipitation (LIP) Event is described in Section 2 of the MNS Flooding HRR contained in Attachment 1 of this submittal. Section 3 of the HRR provides a comparison of the MNS LIP CLB and the reevaluated LIP flood causing mechanism. As a result of this comparison and the HRR conclusions and supporting calculations, MNS has identified reevaluated LIP related flooding hazards which are not bounded by the CLB. These non-bounded flooding hazards are summarized below along with interim actions which provide a reasonable level of confidence these non-bounded hazards do not pose an imminent risk to the units and public health and safety until the total plant response to these hazards is determined by an integrated assessment.

LIP Related MNS Power Block Yard Non-Bounded Hazard

The maximum reevaluated LIP flood elevation in the MNS power block yard is 761.1 ft msl. This would exceed the existing yard curbs or door entrances elevation of 760.5 ft msl for approximately 2 hours. The MNS Auxiliary Building and the safety related systems and equipment within that building are the only MNS SSCs where LIP related yard water levels in excess of 760.5 ft msl could potentially have an adverse impact on the units and public health and safety.

LIP related water flow into the MNS Auxiliary Building from the power block yard would enter cracks in various door and doorway openings for a peak time period of about 2 hours and flow into one of the three interconnected Groundwater Drainage System sumps in the Auxiliary Building. This water flow could exceed the combined capacity of the pumps which remove water from these sumps. Since the Auxiliary Building contains safety related equipment, this water flow could have an adverse impact on equipment needed to ensure the safety of the units and the health and safety of the public. Therefore, an interim action is needed to limit power block yard flood water from entering the Auxiliary Building to ensure this reevaluated hazard does not pose an imminent risk to the units and public health and safety.

LIP related water flow into the Unit 1 and Unit 2 Turbine Buildings from the power block yard would not adversely affect the safety related Auxiliary Building due to QA-1 sealed flood walls between the buildings. These walls are periodically inspected to ensure they retain the ability to protect the Auxiliary Building from in-leakage of water from the Turbine Buildings. In addition, LIP related water flow into the Turbine Buildings from the power block yard would not adversely affect the safety related Emergency Diesel Generator Rooms due to QA-1 sealed flood walls and QA-1 flood doors between the Turbine Buildings and these rooms. These walls and doors are periodically inspected to ensure they retain the ability to protect the Emergency Diesel Generator Rooms from in-leakage of water from the Turbine Buildings.

LIP Related MNS Power Block Yard Interim Action:

LIP-1 > If at any time a meteorological forecast indicates rainfall for the MNS site may approach the amounts associated with a LIP event, actions will be initiated to place sand bags as needed at Auxiliary Building doors and doorway openings prior to a LIP event to limit LIP related power block yard flood water from entering the Auxiliary Building and adversely impacting SSCs within the building. The meteorological forecast trigger conditions for placing the sandbags will ensure the sandbags are placed at Auxiliary Building doors and doorway openings prior to a LIP event. These sand bags will be stacked to height above 761.1 ft msl, the maximum reevaluated LIP flood elevation in the MNS power block yard.

The quantity of sand bags needed to protect the Auxiliary Building has been purchased and are stored on site ready for use if needed. A site procedure has been approved and issued which:

- Describes the triggering conditions for taking actions to place sandbags at the Auxiliary Building doors and door openings as needed prior to a LIP event.
- Provides steps for placing sandbags at the Auxiliary Building doors and door openings as needed prior to a LIP event.

Any training and/or simulation needed to ensure the above LIP related MNS Power Block Yard Interim Action can be successfully implemented when needed has been completed as applicable.

Reference Attachment 3 of this submittal for a regulatory commitment related to the above LIP related MNS Power Block Yard Interim Action. Upon completion of the below LIP-2 MNS Power Block Yard Planned Action, sandbags will no longer be needed to address the above LIP related MNS Power Block Yard Non-Bounded Hazard. At that point, LIP-1 will no longer represent a regulatory commitment.

As a beyond design basis event and given the conservatisms in the approaches used to develop the reevaluated hazards in the MNS Flooding HRR, the extreme reevaluated LIP related power block yard flooding hazard is an unlikely event. Therefore, until the below longer term permanent LIP Related MNS Power Block Yard Planned Action is implemented and the total plant response to this reevaluated hazard is determined by an integrated assessment, the above LIP related MNS Power Block Yard Interim Action provides a reasonable level of confidence the above LIP related MNS Power Block Yard Non-Bounded Hazard does not pose an imminent risk to the units and public health and safety.

LIP Related MNS Power Block Yard Planned Action:

Until the following longer term permanent planned action is implemented and total plant response to the reevaluated hazard is determined by an integrated assessment, the LIP related MNS Power Block Yard Interim Action described above will provide a reasonable level of confidence the above LIP related MNS Power Block Yard Non-Bounded Hazard does not pose an imminent risk to the units and public health and safety.

LIP-2 > Frames will be installed in vulnerable doorways in the MNS Auxiliary Building. Temporary flood doors will be installed in the frames to limit any power block yard flood water from entering the Auxiliary Building during a LIP flooding event. Upon receipt of the triggering condition, plant procedures will direct the station personnel to install the temporary doors prior to a LIP event. The trigger conditions for installing the temporary flood doors will ensure they are installed prior to a LIP event. Simulations and/or training will be performed as needed to ensure the station personnel who will perform these actions are capable of installing the temporary flood doors in a timeframe which would protect equipment in the Auxiliary Building.

Based upon the projected time needed to perform design work, procure materials, and implement the above planned action, it is reasonable to expect this planned action could be completed by June 30, 2015. Therefore, MNS shall implement this planned action by that date. Reference Attachment 3 of this submittal for a regulatory commitment related to the above LIP related MNS Power Block Yard Planned Action. As a beyond design basis event and given the conservatisms in the approaches used to develop the reevaluated hazards in the MNS Flooding HRR, the extreme reevaluated LIP related power block yard flooding hazard is an unlikely event. Given this and the above LIP related MNS Power Block Yard Interim Action, an implementation date of June 30, 2015 does not pose an imminent risk to the units and public health and safety.

<u>LIP Related MNS Site Roofing Non-Bounded Hazard</u>

Given the rainfall associated with the reevaluated LIP event, sufficient quantities of water could collect on site roofs to cause the failure of non-safety building roofing. Due to the proximity of this failed roofing to the Auxiliary Building, water from the failed roofing could make its way into the Auxiliary Building. This water would flow into one of the three interconnected Groundwater Drainage System sumps in the Auxiliary Building. This water flow could exceed the combined capacity of the pumps which remove water from these sumps. Since the Auxiliary Building contains safety related equipment, this water flow could have an adverse impact on equipment needed to ensure safety of the units. Therefore, an interim action is needed to prevent this roof related flood water from entering the Auxiliary Building to ensure this reevaluated hazard does not pose an imminent risk to the units and public health and safety.

The MNS Auxiliary Building and the safety related systems and equipment within that building are the only MNS SSCs where roof related flood water could potentially have an adverse impact on unit safety.

LIP Related MNS Site Roofing Interim Action:

LIP-3 > If at any time a meteorological forecast indicates rainfall for the MNS site may approach the amounts associated with a LIP event, actions will be initiated to ensure, prior to a LIP event, holes will be cut in the roofing of non-safety buildings as needed to prevent LIP related site roofing flood water from adversely impacting SSCs within the Auxiliary Building. The meteorological forecast trigger conditions for cutting holes in the roofing will ensure the holes are cut prior to a LIP event. These holes will be of sufficient size and quantity to ensure water collection on this roofing will not cause failure of these roofs and the subsequent influx of roof water into the Auxiliary Building. Water will flow through these holes into the non-safety buildings. There are no SSCs within these non-safety buildings that could be impacted by this influx of water such that there would be an adverse impact on plant safety.

The equipment needed to cut the holes in the non-safety building roofs and protect the Auxiliary Building will be staged and ready for use if needed. A site procedure has been approved and issued which:

- Describes the triggering conditions for taking actions to cut the holes in the non-safety building roofs as needed prior to a LIP event to protect the Auxiliary Building.
- Provides steps for cutting the holes in the non-safety building roofs as needed prior to a LIP event to protect the Auxiliary Building.

Any training and/or simulation needed to ensure the above LIP related MNS Site Roofing Interim Action can be successfully implemented when needed has been completed as applicable.

Reference Attachment 3 of this submittal for a regulatory commitment related to the above LIP related MNS Site Roofing Interim Action. Upon completion of the below LIP-4 MNS Site Roofing Planned Action, cutting holes in the roofing of non-safety buildings will no longer be needed to address the above LIP related MNS Site Roofing Non-Bounded Hazard. At that point, LIP-3 will no longer represent a regulatory commitment.

As a beyond design basis event and given the conservatisms in the approaches used to develop the reevaluated hazards in the MNS Flooding HRR, the extreme reevaluated LIP related site roofing flooding hazard is an unlikely event. Therefore, until the below longer term permanent LIP Related MNS Site Roofing Planned Action is implemented and the total plant response to this reevaluated hazard is determined by an integrated assessment, the above LIP related MNS Site Roofing Interim Action provides a reasonable level of confidence the above LIP related MNS Site Roofing Non-Bounded Hazard does not pose an imminent risk to the units and public health and safety.

LIP Related MNS Site Roofing Planned Action:

Until the following longer term permanent planned action is implemented and total plant response to the reevaluated hazard is determined by an integrated assessment, the LIP related MNS Site Roofing Interim Action described above will provide a reasonable level of confidence the above LIP related MNS Site Roofing Non-Bounded Hazard does not pose an imminent risk to the units and public health and safety.

LIP-4 > New scuppers will be installed in the parapet walls of site roofing as needed to limit the quantity of LIP related flood water that collects on site roofing thereby preventing a roofing failure and the subsequent influx of roof water into the Auxiliary Building. Based upon the projected time needed to perform design work, procure materials, and implement the above planned action, it is reasonable to expect this planned action could be completed by June 30, 2015. Therefore, MNS shall implement this planned action by that date. Reference Attachment 3 of this submittal for a regulatory commitment related to the above LIP related MNS Site Roofing Planned Action. As a beyond design basis event and given the conservatisms in the approaches used to develop the reevaluated hazards in the MNS Flooding HRR, the extreme reevaluated LIP related site roofing flooding hazard is an unlikely event. Given this and the above LIP related MNS Site Roofing Interim Action, an implementation date of June 30, 2015 does not pose an imminent risk to the units and public health and safety.

3.0 Combined Effects Probable Maximum Flood

The MNS reevaluated Combined Effects Probable Maximum Flood (Combined Effects/PMF) Event is described in Section 2 of the MNS Flooding HRR contained in Attachment 1 of this submittal. Section 3 of the HRR provides a comparison of the MNS Combined Effects/PMF CLB and the reevaluated Combined Effects/PMF flood causing mechanism. As a result of this comparison and the HRR conclusions and supporting calculations, MNS has identified reevaluated Combined Effects/PMF related flooding hazards which are not bounded by the CLB. These non-bounded flooding hazards are summarized below along with interim actions which provide a reasonable level of confidence these non-bounded hazards do not pose an imminent risk to the units and public health and safety until the total plant response to these hazards is determined by an integrated assessment.

Combined Effects/PMF Related MNS Non-Bounded Hazards

As described in the HRR, MNS is protected from upstream flooding by [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) Lake Norman which is impounded by Cowans Ford Dam is immediately north of the site.

The MNS Flooding HRR determined the reevaluated Combined Effects/PMF flood resulted in Lake Norman water levels at Cowans Ford Dam that:

- [redacted]

  [redacted]

  Note, with the exception of the spent fuel dry cask storage area of the yard, water elevations in the MNS power block yard associated with this re-evaluated hazard would be bounded by the yard water levels associated with the reevaluated LIP event hazard. Similar to the LIP event, interim action LIP-1 and planned action LIP-2 will be implemented as applicable to protect the MNS Auxiliary Building from power block yard flood water to ensure this revaluated hazard does not pose an imminent risk to the units and public health and safety.

  [redacted]

  Note, given the design of the dry casks, any accumulation of flood water in the spent fuel dry cask storage area of the MNS power block yard could not enter the sealed inner casks and come in contact with spent fuel.

- Produced wave run-up spill over of the [redacted]

  Wave run-up spill over of the [redacted] However, it would result in the accumulation of flood water in the MNS power block yard. The accumulation of flood water in the power block yard would be bounded by the flood water levels associated with the reevaluated LIP event hazard. Similar to the LIP event, interim action LIP-1 and planned action LIP-2 will be implemented as applicable to protect the MNS Auxiliary Building from power block yard flood water to ensure this revaluated hazard does not pose an imminent risk to the units and public health and safety.

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

As described in Section 3 of the MNS Flooding HRR. [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) would result in an accumulation of flood water in the MNS power block yard. This yard water level would not be bounded by yard water levels associated with the reevaluated LIP event hazard. Therefore, an interim action is needed [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

Combined Effects/PMF Related MNS Interim Actions:

CE/PMF-1 > [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

[redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) a beyond design basis event and given the conservatisms in the approaches used to develop the reevaluated hazards in the MNS Flooding HRR, the extreme reevaluated Combined Effects/PMF event is an unlikely event. [redacted] (b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F)

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

Reference Attachment 3 of this submittal for a regulatory commitment related to the above Combined Effects/PMF Related MNS Interim Action.

CE/PMF-2 > If at any time a meteorological forecast indicates rainfall for the MNS site may approach the amounts associated with a Combined Effects/PMF event, the following actions will be initiated prior to a Combined Effects/PMF event to ensure the above Combined Effects/PMF Related MNS Non-Bounded Hazards do not pose an imminent risk to the units and public health and safety. The meteorological forecast trigger conditions for initiating the below actions will ensure these actions are completed prior to a Combined Effects/PMF event:

- Interim action LIP-1 and planned action LIP-2 will be implemented as applicable to protect the MNS Auxiliary Building from power block yard flood water. Upon completion of planned action LIP-2, interim action LIP-1 will no longer be needed to address the above Combined Effects/PMF Related MNS Non-Bounded Hazards. At that point, LIP-1 will no longer represent a regulatory commitment.
- Additional plant staffing will be staged onsite as needed to ensure the required plant staffing levels and capabilities are maintained.
- A diesel powered pump(s) and supporting equipment will be staged as needed to maintain flood water level in the spent fuel dry cask yard area below the level of the dry cask lower cooling air inlets. This pump and equipment are onsite and ready for use if needed.

A site procedure has been approved and issued which:

- Describes the triggering conditions for taking actions to place sandbags at the Auxiliary Building doors and door openings as needed prior to a Combined Effects/PMF event.
- Provides steps for placing sandbags at the Auxiliary Building doors and door openings as needed prior to a Combined Effects/PMF event.
- Describes the triggering conditions for taking actions to stage additional plant staffing onsite as needed prior to a Combined Effects/PMF event.
- Provides steps for staging additional plant staffing onsite as needed prior to a Combined Effects/PMF event to ensure the required plant staffing levels and capabilities are maintained.
- Describes the triggering conditions for taking actions to stage a diesel powered pump(s) and supporting equipment as needed prior to a Combined Effects/PMF event.
- Provides steps for staging and operating a diesel powered pump(s) and supporting equipment as needed prior to a Combined Effects/PMF event.

Any training and/or simulation needed to ensure the above Combined Effects/PMF Related MNS Interim Actions can be successfully implemented when needed has been completed as applicable.

Reference Attachment 3 of this submittal for a regulatory commitment related to the above Combined Effects/PMF Related MNS Interim Action.

As a beyond design basis event and given the conservatisms in the approaches used to develop the reevaluated hazards in the MNS Flooding HRR, the extreme reevaluated Combined Effects/PMF event is an unlikely event. Therefore, until the total plant response to this reevaluated hazard is determined by an integrated assessment, the above Combined Effects/PMF Related MNS Interim Actions provide a reasonable level of confidence the above Combined Effects/PMF Related MNS Non-Bounded Hazards do not pose an imminent risk to the units and public health and safety.

ATTACHMENT 3
McGuire Nuclear Station Reevaluation Report Regulatory
Commitments

The following table documents the interim and planned actions described in Attachment 2 which represent regulatory commitments. In addition to the below, Duke Energy commits to not modify any of these commitments and associated completion dates without notifying the NRC in advance.

Interim and Planned Actions Which Represent Regulatory Commitments	**Regulatory Commitment Completion Date**
LIP-1 > Approved site procedure(s) will be in place that will ensure sandbags are placed as needed at Auxiliary Building doors and doorway openings prior to a LIP event. The sandbags will be on site and any training and/or simulation needed to ensure the above can be successfully implemented will be completed as applicable. Upon completion of the below LIP-2 regulatory commitment, LIP-1 will no longer represent a regulatory commitment.	By March 12, 2014
LIP-2 > Frames will be installed in vulnerable doorways in the MNS Auxiliary Building. Approved site procedure(s) will be in place that will ensure temporary flood doors will be installed in the frames prior to a LIP event. The temporary flood doors will be on site and any training and/or simulation needed to ensure the above can be successfully implemented will be completed as applicable.	By June 30, 2015
LIP-3 > Approved site procedure(s) will be in place that will ensure, prior to a LIP event, holes are cut in the roofing of non-safety buildings as needed to prevent LIP related site roofing flood water from adversely impacting SSCs within the Auxiliary Building. Any training and/or simulation needed to ensure the above can be successfully implemented will be completed as applicable. Upon completion of the below LIP-4 regulatory commitment, LIP-3 will no longer represent a regulatory commitment.	By March 12, 2014
LIP-4 > New scuppers will be installed in the parapet walls of site roofing as needed to limit the quantity of LIP related flood water that collects on site roofing thereby preventing a roofing failure and the subsequent influx of roof water into the Auxiliary Building.	By June 30, 2015
CE/PMF-1 > [redacted] (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) [redacted]	(b)(3):16 U.S.C. § 824o-1(d), (b)(4), (b)(7)(F) [redacted]

Interim and Planned Actions Which Represent Regulatory Commitments (continued)	**Regulatory Commitment Completion Date**
CE/PMF-2 > Approved site procedure(s) will be in place that will ensure, prior to a Combined Effects/PMF event, the above LIP-1 and LIP-2 regulatory commitments will be implemented as applicable, additional plant staffing will be staged onsite as needed to ensure the required plant staffing levels and capabilities are maintained, a diesel powered pump(s) and supporting equipment will be staged as needed to maintain flood water level in the spent fuel dry cask yard area below the level of the dry cask lower cooling air inlets. The diesel powered pump(s) and supporting equipment will be on site and any training and/or simulation needed to ensure the above can be successfully implemented will be completed as applicable.	By March 12, 2014
Duke Energy commits to not modify any of the above regulatory commitments and associated completion dates without notifying the NRC in advance.	N/A