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March 11, 2014
L-14-104

10 CFR 50.54(f)

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

SUBJECT:
Davis-Besse Nuclear Power Station
Docket No. 50-346, License No. NPF-3
FirstEnergy Nuclear Operating Company (FENOC) Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding the Flooding Aspects of Recommendation 2.1 of the Near-Term Task Force (NTTF) Review of Insights from the Fukushima Dai-ichi Accident

On March 12, 2012, the Nuclear Regulatory Commission (NRC) issued a letter titled, "Request for Information Pursuant to Title 10 of the *Code of Federal Regulations* 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," to all power reactor licensees and holders of construction permits in active or deferred status. Enclosure 2 of the 10 CFR 50.54(f) letter addresses NTTF Recommendation 2.1 for flooding. One of the required responses is for licensees to submit a Hazard Reevaluation Report (HRR) in accordance with the NRC's prioritization plan. By letter dated May 11, 2012, the NRC placed the Davis-Besse Nuclear Power Station (DBNPS) in Category 2 requiring a response by March 12, 2014. The Flood HRR for DBNPS is enclosed.

As discussed in the enclosed report, two flood levels (local intense precipitation and probable maximum storm surge) determined during the hazard reevaluation exceed the current licensing basis (CLB) flood levels. The increased levels are the result of newer methodologies and not the result of errors within the CLB evaluations. Current plant procedures addressing flooding at the site provide actions to be taken in the event flooding is imminent or has occurred at or near the DBNPS site. No additional actions beyond those currently in place are necessary at this time.

In accordance with the guidance provided by NRC letter dated December 3, 2012, titled "Trigger Conditions for Performing an Integrated Assessment and Due Date for Response," an integrated assessment is required if flood levels determined during the hazard reevaluation are not bounded by the CLB flood levels. The 10 CFR 50.54(f) specifies that the integrated assessment be completed and a report submitted within two years of submitting the HRR. Therefore, FENOC intends to submit an Integrated Assessment Report for DBNPS prior to March 12, 2016.

There are no regulatory commitments contained in this letter. If there are any questions or if additional information is required, please contact Mr. Thomas A. Lentz, Manager – Fleet Licensing, at 330-315-6810.

I declare under penalty of perjury that the foregoing is true and correct. Executed on March 11, 2014.

Respectfully,

Raymond A. Lieb

Enclosure:
Flood Hazard Reevaluation Report

cc: Director, Office of Nuclear Reactor Regulation (NRR)
NRC Region III Administrator
NRC Resident Inspector
NRR Project Manager
Utility Radiological Safety Board

Enclosure
L-14-104

Flood Hazard Reevaluation Report
(33 pages follow)

FLOOD HAZARD REEVALUATION REPORT

IN RESPONSE TO THE 50.54(f) INFORMATION REQUEST REGARDING NEAR-TERM TASK FORCE RECOMMENDATION 2.1: FLOODING

for the

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Revision 1

Submitted to FENOC: March 06, 2014

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Table of Contents

1. PURPOSE

1.1. Background

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

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

1. NTTF Recommendation 2.1: Seismic
2. NTTF Recommendation 2.1: Flooding
3. NTTF Recommendation 2.3: Seismic
4. NTTF Recommendation 2.3: Flooding
5. NTTF Recommendation 9.3: EP
6. Licensees and Holders of Construction Permits

In Enclosure 2 of the NRC-issued information request (Reference NRC March 2012), the NRC requested that licensees reevaluate the flooding hazards at their sites against present-day regulatory guidance and methodologies being used for early site permits (ESP) and combined operating license reviews.

On behalf of First Energy Corporation (FENOC) for the Davis-Besse Nuclear Power Station (DBNPS), this Flood Hazard Reevaluation Report (Report) provides the information requested in the March 12, 2012 50.54(f) letter; specifically, the information listed under the "Requested Information" section of Enclosure 2, paragraph 1 ('a' through 'e'). The "Requested Information" section of Enclosure 2, paragraph 2 ('a' through 'd'), Integrated Assessment Report, will be addressed separately if the current design basis floods do not bound the reevaluated hazard for all flood-causing mechanisms.

1.2. Requested Actions

Per Enclosure 2 of the NRC-issued information request, 50.54(f) letter, FENOC is requested to perform a reevaluation of all appropriate external flooding sources for DBNPS, including the effects from local intense precipitation (LIP) on the site, the probable maximum flood (PMF) on streams and rivers, lake flooding from storm surges, seiches and tsunamis, and dam failures. It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for ESPs, and calculation reviews including current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The requested information will be gathered in Phase 1 of the NRC staff's two-phase process to implement Recommendation 2.1, and will be used to identify potential "vulnerabilities" (see definition below).

For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan documenting planned actions or measures implemented to address the reevaluated hazards.

Subsequently, addressees shall perform an integrated assessment of the plant to fully identify vulnerabilities and detail actions to address them. The scope of the integrated assessment report will include full power operations and other plant configurations that could be susceptible due to the status of the flood protection features. The scope also includes those features of the ultimate heat sink (UHS) that could be adversely affected by flood conditions (the loss of UHS from non-flood associated causes is not included). It is also requested that the integrated assessment address the entire duration of the flood conditions.

A definition of vulnerability in the context of Enclosure 2 is as follows: Plant-specific vulnerabilities are those features important to safety that when subject to an increased demand due to the newly calculated hazard evaluation have not been shown to be capable of performing their intended functions.

1.3. Requested Information

Per Enclosure 2 of the NRC-issued information request 50.54(f) letter, the Report should provide documented results, as well as pertinent DBNPS information and detailed analysis, and include the following:

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

    1. Detailed site information (both designed and as-built), including present-day site layout, elevation of pertinent SSCs important to safety, site topography, and pertinent spatial and temporal data sets;
    2. Current design basis flood elevations for all flood-causing mechanisms;
    3. Flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance;
    4. Changes to the watershed and local area since license issuance;
    5. Current licensing basis flood protection and pertinent flood mitigation features at the site; and
    6. Additional site details, as necessary, to assess the flood hazard (e.g., bathymetry and walkdown results).

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

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

4. 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.

5. Additional actions beyond requested information item 1.d taken or planned to address flooding hazards, if any.

2. SITE INFORMATION

DBNPS is located on the shore of Lake Erie in Oak Harbor, Ohio. The major hydrological features of the terrain are the broad expanse of Lake Erie to the north and east, and the Toussaint River, which flows east into the lake along the south side of DBNPS. DBNPS is approximately 3,000 feet (ft) from the Lake Erie shoreline (USAR, Section 1.2.1.1) and approximately 2,000 ft from the Toussaint River. Site areas surrounding the station structures have been built up from 6 to 14 feet above the existing grade elevation to an elevation of 584 ft International Great Lakes Datum of 1955 (IGLD55) or 15.4 ft above the Lake Erie Low Water Datum of 568.6 ft-IGLD55. Topography at and around DBNPS is relatively flat, with a mean station elevation of approximately 584 ft-IGLD55. The site safety-related structures are protected against high water levels up to an elevation of 585 ft-IGLD55. A Lake Erie dike, which is located along the shore of Lake Erie, protects the site from lake surges. Additionally, a wave protection dike is situated along the northern, eastern, and a small portion of southern sides of DBNPS. The elevation at the top of the wave protection dike is 591 ft-IGLD55. Present-Day Site Layout is shown in Figures 2.0.1.

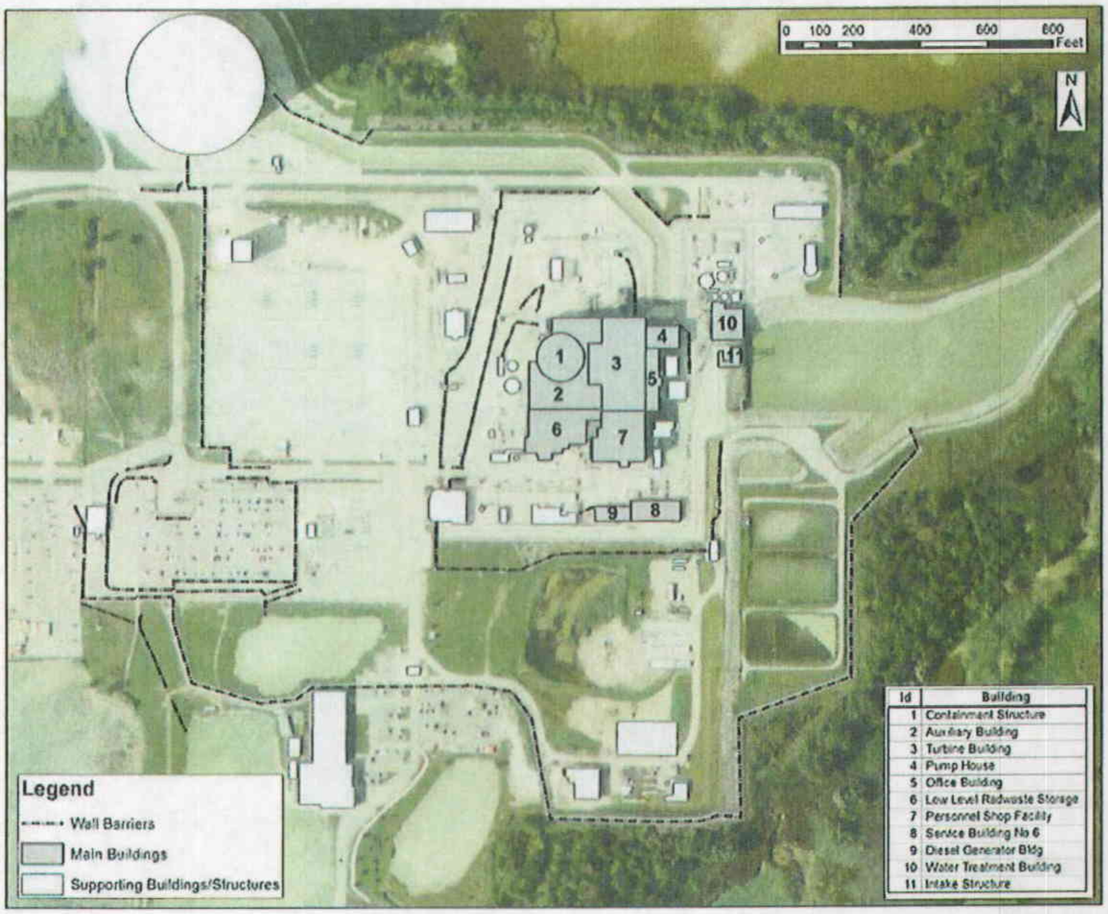

Figure 2.0.1 – Present-Day Site Layout

2.1. Current Design Basis

The current design basis is defined in the DBNPS Updated Safety Analysis Report (USAR). The following is a list of flood-causing mechanisms and their associated water surface elevations that were considered for the DBNPS current design basis.

2.1.1. LIP

The USAR indicates that the precipitation value of 24.5-inches over a 6 hour period is utilized for the LIP analysis. As indicated in the USAR, the average invert elevation of manholes and catch basins is 582 ft-IGLD55; with 24.5 inches of estimated accumulation, water could build up to 584.5 ft-IGLD55 (USAR, Section 2.4.2.3).

2.1.2. Flooding in Streams and Rivers

The USAR indicates that a flow rate of 78,500 cubic feet per second (cfs) in the Toussaint River at DBNPS (USAR, Section 2.4.3) would result in a maximum water surface elevation of 579 ft-IGLD55. As indicated in the USAR, the elevation of 579 ft-IGLD55 was derived using the conservative assumption that none of the water is discharged to Lake Erie, assuming that the PMF flow is hypothetically dammed up at that point (USAR, Section 2.4.3.5).

2.1.3. Dam Breaches and Failures

The USAR indicates that there are no dams or other regulating hydraulic structures on the Toussaint River which would affect the flow hydrograph at DBNPS (USAR, Section 2.4.3).

2.1.4. Storm Surge & Seiche

The probable maximum meteorological event in Lake Erie results in a maximum water surface elevation of 583.7 ft-IGLD55. This meteorological event is caused by a maximum east-northeast wind at any location of 100 miles per hour for a 10-minute duration, and a wind speed of 70 miles per hour during the six-hour period both before and after the maximum wind speed (USAR, Section 2.4.5).

2.1.5. Low Water

No water is taken from the Toussaint River for plant cooling water requirements. Therefore, low flows in the Toussaint River will not affect DBNPS operation. The probable maximum meteorological event in Lake Erie results in the probable extreme low water level of 556.8 ft-IGLD55 (USAR, Section 2.4.11).

2.1.6. Ice-Induced Flooding

Flooding of the safety-related structures and equipment at DBNPS due to ice jams in the Toussaint River is not credible. The USAR indicates that the elevation of the plant structures is above the level of normal lake ice formations. Category 1 wave protection dikes are designed to withstand the impact of ice (USAR, Section 2.4.7).

2.1.7. Channel Migration or Diversion

As indicated in the USAR, the mean lake level is not subject to variations due to diversions or source cutoff (USAR, Section 2.4.9).

2.1.8. Combined Effect Flood (including Wind-Generated Waves)

Wind-wave activity, including runup, was evaluated for its effect on the wave protection dikes on the north, east, and south sides of DBNPS. As indicated in the USAR, the maximum wave run-up on the dike is 6.6 ft above the probable maximum water surface elevation of 583.7 ft-IGLD55. The resulting maximum wave runup elevation is 590.3 ft-IGLD55, which is below the top of the dike (USAR, Section 2.4.2.2.1).

2.2. Flood-Related Changes to the License Basis

There were no changes to the license basis since the initial license issuance with regard to flooding.

2.3. Changes to the Watershed and Local Area since License Issuance

The watershed contributory to the Toussaint River upstream of DBNPS is approximately 139.0 square miles (Reference DBNPS 2013c). Based on aerial images of the watershed, the changes to the watershed include commercial development within the watershed area, which is a very small percentage of the overall watershed area. The changes to the local area sub-watershed for DBNPS include buildings, parking lots, and security barrier upgrades that have been added to the site since license issuance.

2.4. Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

The maximum flood level in the design basis is below the site finish floor elevation of 585 ft-IGLD55. Therefore, there were no mitigation actions initiated or taken for flooding at the site.

3. SUMMARY OF FLOOD HAZARD REEVALUATION

NUREG/CR-7046, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America* (Reference NUREG/CR-7046), by reference to the American Nuclear Society (ANS), states that a single flood-causing event is inadequate as a design basis for power reactors and recommends that combinations should be evaluated to determine the highest flood water elevation at the site. For DBNPS, the combination that produces the highest flood water elevation at the site is the probable maximum surge and seiche on Lake Erie with the effects of coincident wind wave activity.

The USAR reports elevations corresponding to IGLD55 vertical datum. The recent site survey, United States Geological Survey (USGS) topographic maps, and other reference documents report elevation in North American Vertical Datum of 1988 (NAVD88). In order to compare the reevaluated flood elevations with the existing design basis reported in USAR, final pertinent elevations have been converted to IGLD55 datum. The conversion between IGLD55 and NAVD88 at DBNPS is represented as-- ft-IGLD55 = ft-NAVD88 – 1.07 ft.

Calculation C-CSS-020.13-017 (Reference DBNPS 2013i) defines the maximum water surface elevation of 585.81 ft-IGLD55 at DBNPS adjacent to the power block. This elevation is due to a probable maximum storm surge (PMSS) during a probable maximum wind storm (PMWS) event. The revised maximum water surface elevation is above the site finish floor elevation of 585 ft-IGLD55.

Calculation C-CSS-020.13-022 (Reference DBNPS 2013n) defines the coincident wind wave runup. The maximum wave runup elevation of the PMSS coincident with wind wave activity is determined by adding the wind wave runup to the water surface flood elevation due to the PMSS. The maximum runup on the wave protection dike is 589.88 ft-IGLD55, which is below the top of the wave protection dike elevation of 591 ft-IGLD55. The maximum wave runup elevation in the vicinity of the power block is 585.90 feet-IGLD55. The wave runup elevations in the vicinity of the power block are above the site finish floor elevation of 585 ft-IGLD55.

Calculation C-CSS-020.13-014 (Reference DBNPS 2013f) defines the maximum water surface elevation resulting from the LIP event. The water surface elevation due to the LIP event varies from 585.17 ft-IGLD55 to 585.44 ft-IGLD55. The LIP maximum water surface elevations are above the site finish floor elevation of 585ft-IGLD55

The methodology used in the flooding reevaluation for DBNPS is consistent with the following standards and guidance documents:

- NRC Standard Review Plan, NUREG-0800, revised March 2007 (Reference NUREG-0800)
- NRC Office of Standards Development, Regulatory Guides, RG 1.102 – "Flood Protection for Nuclear Power Plants", Revision 1, dated September 1976 (Reference NRC RG 1.102) and RG 1.59-"Design Basis Floods for Nuclear Power Plants", Revision 2, dated August 1977 (Reference NRC RG 1.59)

- NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," dated November 2011 (Reference NUREG/CR-7046)
- NUREG/CR-6966, "Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America", dated March 2009 (Reference NUREG/CR-6966)
- "American National Standard for Determining Design Basis Flooding at Power Reactor Sites", dated July 28, 1992 (Reference ANSI/ANS-2.8-1992)
- NEI Report 12-08, "Overview of External Flooding Reevaluations" (Reference NEI August 2012)
- NRC JLD-ISG-2012-06, "Guidance for Performing a Tsunami, Surge or Seiche Flooding Hazard Assessment", Revision 0, dated January 4, 2013 (Reference JLD-ISG-2012-06)
- NRC JLD-ISG-2013-01, "Guidance for Assessment of Flooding Hazards due to Dam Failure", Revision 0, dated July 29, 2013 (Reference JLD-ISG-2013-01)

The following provides the flood-causing mechanisms and their associated water surface elevations that are considered in the DBNPS flood hazard reevaluation study:

3.1. Flooding in Streams and Rivers (Reference DBNPS 2013a, DBNPS 2013b, and DBNPS 2013c)

The PMF in rivers and streams adjoining the site is determined by applying the probable maximum precipitation (PMP) to the drainage basin in which the site is located. The PMF is based on a translation of PMP rainfall in the watershed to flood flow. The PMP is a deterministic estimate of the theoretical maximum depth of precipitation that can occur at a time of year for a specified area. A rainfall-to-runoff transformation function, as well as runoff characteristics, based on the topographic and drainage system network characteristics and watershed properties, are needed to appropriately develop the PMF hydrograph. The PMF hydrograph is a time history of the discharge and serves as the input parameter for other hydraulic models which develop the flow characteristics, including flood flow and elevation.

The PMF is a function of the combined events defined in NUREG/CR-7046 for floods caused by precipitation events.

Alternative 1 – Combination of:

- Mean monthly base flow
- Median soil moisture
- Antecedent rain: lesser of (1) rainfall equal to 40 percent of the PMP, or (2) a 500-year rainfall
- The All-Season PMP
- Waves induced by 2-year wind speed applied along the critical direction

Alternative 2 – Combination of:

- Mean monthly base flow

- Snowmelt from the Probable maximum snowpack
- A 100-year, cool-season rainfall
- Waves induced by 2-year wind speed applied along the critical direction

Alternative 3 – Combination of:

- Mean monthly base flow
- Snowmelt from a 100-year snowpack
- The cool-season PMP
- Waves induced by 2-year wind speed applied along the critical direction

3.1.1. Basis of Inputs:

The inputs used in the PMP, snowmelt, and PMF analyses are based on the following:

All-Season PMP Analysis and Cool-Season PMP with Snowmelt Analysis

- DBNPS and Toussaint River watershed locations, areas, boundaries and configurations;
- Historic flow rate data collected by USGS at gage 04195820 on the Portage River;
- HMR-52-standard isohyetal patterns, storm orientation, percentage of 6-hour increment of PMP, and standard isohyetal geometry information;
- HMR-53-seasonal PMP values;
- The 100-year all-season point rainfall estimates from the National Oceanic and Atmospheric Administration (NOAA) Precipitation Frequency Data Server;
- Median and extreme daily snow cover by month for Ohio, data is downloaded from NOAA; and
- Snowmelt rate (energy budget) equations and constants are based on U.S. Army Corps of Engineers (USACE) Engineering Manual EM-1110-2-1406.

PMF analysis-Hydrologic & Hydraulic Analysis

- Digital elevation model (DEM): The DEM used for the PMF calculation is obtained from the U.S. Department of Agriculture (USDA) National Resource Conservation Services (NRCS), national elevation data;
- Probable maximum precipitation (PMP): 72-hour PMP and associated snowmelt, as applicable, for the subject watershed area;
- Baseflow: Historic flow rate data collected by USGS at gage 04195820 on the Portage River, which is used as the baseflow for the Toussaint River;
- Soil Type: The soil types within the project watershed are developed using USDA NRCS soil information;
- Land Use: The land use information for the watershed is obtained from USDA NRCS soil survey geographic database;

- Manning's roughness coefficients are based on a visual assessment of aerial photography and selected using standard applicable engineering guidance references;
- Bridge information obtained from the Ohio Department of Transportation (ODOT); and
- Supporting Geographic Information System (GIS) input (Reference DBNPS 2013k).

3.1.2. Computer Software Programs

PMP and Snowmelt analysis

- AutoCAD Civil 3D 2012
- ArcGIS Desktop 10.1
- HMR-52 software
- Microsoft Excel

PMF analysis

- ArcGIS Desktop 10.1
- HEC-HMS 3.5
- HEC-RAS 4.1
- HEC-GeoRAS 10.1
- IGLD85 Height Conversion Online Tool
- Microsoft Excel

3.1.3. Methodology

The PMF analysis included the following steps:

- Delineate watershed and sub-watersheds, then calculate sub-watershed areas for input into the USACE HEC-HMS rainfall-runoff hydrologic computer model.
- Determine rainfall.
- Estimate HEC-HMS rainfall-runoff model initial input parameters: Snyder unit hydrograph method, peaking and lag coefficient.
- Calculate HEC-HMS model loss input parameters: initial loss and constant loss rate.
- Calculate HEC-HMS river reach routing model initial input parameters: Muskingum-Cunge.
- Method: 8-point cross-section, reach length, slope, and Manning's roughness coefficient.
- Perform PMF simulation with PMP input using HEC-HMS model.
- Estimate water surface elevation using HEC-RAS unsteady-state model by using runoff from the HEC-HMS model as an input.

Watershed Delineation

For the purposes of the hydrologic modeling effort, the Toussaint River watershed is subdivided into three (3) sub-watersheds (Packer Creek, Upper Toussaint Creek, and Lower Toussaint Creek) based on the hydrologic unit code (HUC) boundaries.

Rainfall & Snowmelt

Each alternative contains rainfall defined either by the all-season PMP (Alternative 1), the 100-year, cool-season rainfall (Alternative 2), or the cool-season PMP (Alternative 3). Each rainfall event is considered to be a 72-hour duration event. Note that an antecedent rainfall occurs prior to the all-season PMP. An antecedent storm equivalent to 40 percent of the all-season PMP is applied to the HEC-HMS model with a 72-hour dry period between the antecedent storm and the PMP event.

Snowmelt is included in the two cool season alternatives. For rain-on-snow conditions, the air temperature, dew point temperature, and average maximum daily wind speed are obtained from representative weather stations. The basin wind coefficient is determined based on the density of forest stands in each sub-basin. It is conservatively assumed that the Toussaint River watershed is unforested plain to maximize snowmelt. For rain-free conditions, the snowmelt parameters are selected based on the USACE guidance.

The snowpack is assumed to be at its maximum at the onset of rainfall events and cover the entire watershed. Soil is assumed to be frozen with no losses during the months of October through April. For the probable maximum snowpack, snowpack depth is assumed to be available for the duration of the coincident rainfall event.

Alternative 1 – All-Season PMP

The location of the DBNPS watershed is within the domain of the HMR-51 and HMR-52 guidance. The all-season PMP is determined by using the generalized PMP estimates defined by the HMR-51 and HMR-52 guidance. Different storm centers throughout the watershed are examined to determine the critical storm center that maximizes runoff. HMR-52 software is used to optimize the storm and define the PMP estimates for each sub-basin.

HMR-52 software is based on a standard temporal distribution. The HMR guidance indicates the greatest precipitation may occur at other times throughout the duration of the storm. The temporal distribution of the PMP is calculated in accordance with recommendations in HMR-52, wherein individual rainfall increments decrease progressively to either side of the greatest rainfall increment. Various temporal distributions for each rainfall scenario are then evaluated to further maximize the runoff. Front, one-third, center, two-third, and end-loading temporal distributions are considered in an effort to capture the distribution that maximizes runoff.

Alternative 2 – Probable Maximum Snowpack and 100-Year Cool-Season Rainfall

The probable maximum snowpack is assumed to be equal to an unlimited snowpack during the entire coincident rainfall. While the snowpack can be determined directly from the snow depth, there is not adequate data to extrapolate any historical observations up to the magnitude of the probable maximum event.

The 100-year, cool-season rainfall is determined for the watershed location using precipitation frequency estimates defined by NOAA Atlas 14 guidance and applying regional seasonal guidance. NOAA Atlas 14 provides all-season point precipitation rainfall estimates via the NOAA precipitation frequency data server. As NOAA Atlas 14 values are point precipitation values, the estimates are adjusted using area-depth reduction factors. NOAA Atlas 14 values are also adjusted to reflect cool-season rainfall rather than all-season rainfall.

The 3-day (72-hour) snowmelt duration is assumed in order to correspond to the precipitation events (PMP and 100-year rainfall). A 100-year, cool-season rainfall is equivalent for all the cool-season months. However, the snowmelt is expected to be different for each cool-season month because it is highly dependent on temperature, which varies significantly from month to month. Therefore, the month with the highest expected snowmelt is identified and used for the calculations of the snowmelt rates

Alternative 3 – 100-Year Snowpack and Cool-Season PMP

A 100-year snow depth is calculated by performing a statistical analysis based on the historical data obtained from the NOAA Annual Observation Data website. A Fisher-Tippett Type I (FT-I) distribution frequency analysis is performed to determine the maximum snow depth with an annual exceedance probability of 1 percent (i.e. 100-year snow depth). The FT-I distribution is applicable for long-term statistical analyses and specifically for extreme value calculations. The cool-season PMP is determined by applying seasonal HMR-53 guidance to the all-season PMP estimates.

Hydrologic Model (HEC-HMS)

The PMF is the flood resulting from the PMP. The temporal distribution of the PMP is calculated in accordance with the recommendations in HMR-52, wherein individual increments decrease progressively to either side of the greatest increment. For each sub-watershed, a 9-day PMP hyetograph is constructed using a rainfall equivalent to 40 percent of the PMP during the first 72 hours, followed by a dry 72-hour period, and finally followed by the full 72-hour PMP storm.

USACE HEC-HMS hydrologic software is used to convert rainfall to runoff. A rainfall hyetograph is applied to each sub-watershed and transformed to runoff using unit hydrograph methodology. Generally a unit hydrograph is developed using historical data obtained from various rain and stream gages in the watershed. The DBNPS watershed is ungaged. Thus, there are no historical observations available to use as a basis to create a unit hydrograph. Therefore, a synthetic unit hydrograph is developed. The Snyder unit hydrograph methodology is used for rainfall-to-runoff transformation.

Routing accounts for change in the flow hydrograph as a flood wave passes downstream and accounts for storage and attenuation during a flooding event. The Muskingum-Cunge routing method is utilized in the HEC-HMS model, with the streams represented by 8-point cross sections.

ANSI/ANS-2.8-1992 suggests that baseflow should be based on mean monthly flow. As mean monthly flow is not available for the Toussaint River, the baseflow is approximated based on the mean monthly flow in an adjacent watershed. The only gage station available in the same hydrologic unit is on the Portage River near Elmore, OH. The watersheds of the Toussaint River and Portage River are located in the same HUC and have similar watershed characteristics. Therefore, it is an acceptable approach to use the base flow information for the Portage River as the basis for estimation of the base flow at Toussaint River.

Initial losses are ignored. Infiltration, or constant losses, is determined based on the hydrologic characteristics of the soils within each basin. Constant losses are not applied to impervious areas. Additionally, constant losses are ignored for the cool-season PMF alternatives due to the assumption that ground is frozen.

The unit hydrographs for each sub-watershed are modified to account for the effects of nonlinear basin response in accordance with NUREG/CR-7046. The peak of each unit hydrograph is increased by one-fifth and the time-to-peak is reduced by one-third. The remaining hydrograph ordinates are adjusted to preserve the runoff volume to a unit depth over the drainage area.

Hydraulic Model (HEC-RAS)

The unsteady flow module within the USACE HEC-RAS software is used to transform the resulting flow hydrographs from the controlling alternative into a water surface elevation hydrograph under unsteady flow conditions. For reference and comparison, all three alternatives are evaluated with the HEC-RAS model.

Channel and floodplain geometry for the Toussaint River is modeled by developing cross sections of the stream. The cross sections are placed at locations that define geometric characteristics of the river valley and overbanks. Cross sections are also placed at representative locations where changes occur in discharge, slope, shape, and roughness, as well as at hydraulic structures (e.g. bridges). River banks, blocked obstructions, and ineffective flow areas are also incorporated into the HEC-RAS model.

Two bridges are modeled (the CR19 Bridge and the CR2 Bridge) using information received from ODOT. There are two additional bridges over the modeled portion of the Toussaint River: the N. Benton-Carroll Rd Bridge and the CR590 Bridge. These two bridges are located farther away from DBNPS (approximately 7 and 10 river miles respectively). It is not expected that these two bridges would have a measurable effect on the computational results because of the distance. Any possible effect produced by the bridges upstream of the river will be lost due to attenuation in the stream. Therefore, they are not included in the model.

The PMF flow hydrographs obtained from the HEC-HMS model are entered into the HEC-RAS model. The highest observed water level in Lake Erie is used as a downstream boundary condition in the HEC-RAS software program.

The HEC-RAS model is evaluated for both all-season PMF (Alternative 1) and cool-season PMF alternatives (Alternatives 2 and 3).

3.1.4. Results

The Alternative 1 PMF is the controlling combination and is a result of the all-season PMP. The maximum water surface elevation at DBNPS is 575.96 ft-IGLD55 (576.93 ft-NAVD88), with a maximum flow of 100,436 cfs.

For Alternative 2, the maximum water surface elevation is 574.15 ft-IGLD55 (575.12 ft-NAVD88) with a maximum flow of 31,747 cfs.

For Alternative 3, the maximum water surface elevation is 575.06 ft-IGLD55 (576.03 ft-NAVD88) with a maximum flow of 61,943 cfs.

The all-season PMF is determined to be the controlling PMF scenario and an additional combined event analysis is performed in Calculation C-CSS-020.13-022.

3.2. Dam Assessment (Reference DBNPS 2013d)

3.2.1. Basis of Inputs

Inputs used for the dam assessment evaluation include:

- HEC-RAS model developed in the PMF analysis.
- Dam information: The National Inventory of Dams (NID) is used to identify the watershed dams.

3.2.2. Computer Software Programs

- ArcGIS Desktop 10.1
- HEC-RAS 4.1

3.2.3. Methodology

The criteria for dam assessment is provided in the NRC *Guidance for Assessment of Flooding Hazards due to Dam Failure*, JLD-ISG-2013-01. Only two dams reported by NID are located in the DBNPS watershed – the Genoa UG Sewage Disposal Lagoons and the Graymont Sludge Lagoons.

Effects of the failure of the two dams are analyzed using a simplified approach as outlined in JLD-ISG-2013-01. The peak outflow without attenuation method is based on summing estimated discharges from simultaneous failures of upstream dams arriving at the site without attenuation.

The peak discharge using the simplified equations was calculated to be 4,602 cfs and 13,651 cfs for Genoa UG Sewage Disposal Lagoons and Graymont Sludge Lagoons respectively. A cumulative peak breach discharge equal to 18,253 cfs from both dams is included in the HEC-RAS model as additional lateral inflow at the cross section immediately upstream of the DBNPS site. Conservatively, the peak breach flow is applied to the HEC-RAS model at the time corresponding to the PMF peak discharge determined in the PMF analysis.

3.2.4. Results

The maximum water surface elevation at the site resulting from the PMF event combined with the cumulative upstream dam failures is 577.09 ft-NAVD88. Compared to the PMF results, the water surface elevation increase due to the additional dam breach flow is equal to 0.16 ft (577.09 ft – 576.93 ft = 0.16 ft). The maximum PMF water surface elevation at DBNPS is 8.04 ft below the site grade elevation of 584.0 ft-IGLD55. Consequently, the increase due to the dam failure results in a water surface elevation that is 7.88 ft below site grade (8.04 ft – 0.16 ft = 7.88 ft).

The maximum water surface elevation at the site resulting from the PMF event combined with the cumulative upstream dam failures is well below the plant site grade elevation. Therefore, the upstream dams are determined to be noncritical dams as referred to in the JLD-ISG-2013-01. No further dam failure analysis is required.

There are no dams downstream of DBNPS on the Toussaint River.

3.3. Ice-Induced Flooding (Reference DBNPS 2013d)

As identified by NUREG/CR-7046, ice jams and ice dams can form in rivers and streams adjacent to a site, and may lead to flooding by two mechanisms:

- Collapse of an ice jam or an ice dam upstream of the site can result in a dam breach-like flood wave that may propagate to the site; and
- An ice jam or an ice dam downstream of a site may impound water upstream of itself, thus causing a flood via backwater effects.

3.3.1. Basis of Inputs

- USACE ice jam database.
- Bridge geometry (upstream and downstream of DBNPS) - Information relative to the bridge structures provided by ODOT.
- DBNPS HEC-RAS model developed in the PMF analysis.

3.3.2. Computer Software Programs

- HEC-RAS 4.1

3.3.3. Methodology

Per NUREG/CR-7046, ice-induced flooding is assessed by reviewing the USACE ice jam database to determine the most severe historical events that have occurred. There are no historical records available for the Toussaint River. The nonexistence of ice jam records is explained by the absence of stream monitoring stations on the Toussaint River. Based on ice jam occurrence within adjacent streams in the same hydrologic unit code (HUC), it is determined that ice jam events are possible in the Toussaint River.

The maximum ice jam is determined by selecting the historic event that produced the maximum flood stage relative to the normal water surface elevation at that location. Regardless of specific conditions that produced the historic flood stage at a specific location, the full height is conservatively assumed to represent the ice jam.

Historical ice jam data for Portage River, Rock Creek, Bayou Ditch, and Lacarpe Creek are considered as they are located in the same HUC area. The maximum recorded stage due to an ice jam is used.

The peak water surface elevation at DBNPS as a result of an upstream ice jam breach (i.e., failure of ice dam) is estimated. A hypothetical ice jam is incorporated into the HEC-RAS model at the location of the first bridge upstream of DBNPS, the CR2 Bridge. Ice dam breach parameters are selected so the entire ice jam within the main channel would breach when the water level behind the ice jam is at its maximum elevation.

The recorded ice jams have a maximum reported stage of approximately 13 ft; however, there are no records for the height of the ice dams themselves. The clearance between the bottom of the Toussaint River and the low point of the bridge is approximately 8 ft, which is less than the maximum reported stage. Therefore, conservatively, the postulated ice dam could completely block the bridge clearance.

Per NUREG/CR-7046, flooding due to an ice jam is not required to be combined with other extreme flooding events. However, to represent normal flow in the Toussaint River during the cool-season month, a smaller cool-season PMF alternative is utilized (Alternative 2 PMF). The Alternative 2 PMF is a combination of the snowmelt from a probable maximum snowpack and a 100-year, cool-season rainfall. The inflow hydrographs representing the Alternative 2 PMF event are used in the HEC-RAS model evaluating the effect of a postulated ice dam failure.

3.3.4. Results

The maximum water surface elevation at DBNPS resulting from the upstream ice jam breaching was calculated to be 574.05 ft-IGLD55 (575.12 ft-NAVD88). There are no bridges or structures downstream of DBNPS on Toussaint River that could create an ice dam or ice jam.

The water surface elevation at DBNPS due to the PMF is equal to 575.96 ft-IGLD55. Therefore, the ice-induced flooding at DBNPS is bounded by the PMF, and no further consideration is required.

3.4. Channel Migration or Diversion (Reference DBNPS 2013d)

NUREG/CR-7046 indicates historical records and hydrogeomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander towards the site.

3.4.1. Basis of Inputs

- USGS topographic maps
- Aerial images

3.4.2. Computer Software Programs

- Arc GIS 10.1

3.4.3. Methodology

Historic and current topographic maps and aerial images are reviewed to examine the condition and alignment of rivers and streams over time.

Historical maps for the years of 1900, 1952, 1967, 1986, and 2011 were reviewed to assess historic channel migration of the Toussaint River. Toussaint River is approximately 2,000 ft south of the DBNPS. The locations of the river and lake shorelines shown on the historical maps are compared to the present-day locations (2011).

3.4.4. Results

From the comparison of the historical maps, the most significant discrepancies between the present-day and historical stream bank and shoreline locations are observed on the USGS map for 1900. More recent USGS maps show both stream and shoreline locations approximately the same as the current location (within +/- 0.1 mile difference).

Based on the comparison between the current location of the Toussaint River and the river location as shown on the historical maps which cover a period of approximately 110 years, it is determined that channel diversion towards the site is not probable. The same comparison is performed for the lake shoreline and similarly, Lake Erie shoreline diversion towards the site is not probable.

3.5. Storm Surge (Reference DBNPS 2013g, DBNPS 2013h, DBNPS 2013i and DBNPS 2013m)

Probable Maximum Storm Surge (PMSS)

In accordance with JLD-ISG-2012-06, all coastal nuclear power plant sites and nuclear power plant sites adjacent to cooling ponds or reservoirs subject to potential hurricanes, windstorms and squall lines must consider the potential for inundation from storm surge and waves. JLD-ISG-2012-06 also suggests that for the storm surge hazard assessment, historical storm events in the region should be augmented by a synthetic storm parameterized to account for conditions more severe than those in the historical records and considered reasonably possible on the basis of technical reasoning.

3.5.1. Basis of Inputs

The inputs used in PMSS analysis are based on the following:

- Historical wind and pressure field data from NOAA for the Great Lake Region
- Probable maximum windstorm (PMWS)
- Lake Erie bathymetry from the NOAA geophysical database
- Supporting GIS data (Reference DBNPS 2013o)

3.5.2. Computer Software Programs

- ArcMap 10.1
- Deft3D software suite (Delft3D-FLOW, Delft3D-WAVE, Delft3D-RGFGRID, and Delft3D-QUICKIN)

3.5.3. Methodology

Several physical processes contribute to the generation of a storm surge. The contribution of wind to a storm surge is often called wind setup. Wind blowing over the water causes a shear stress that is exerted on the surface of the water, pushing water in the direction of the wind. Atmospheric pressure gradients are another forcing mechanism that contributes to changes in water level, as water is forced from regions of high atmospheric pressure toward regions of low pressure.

The following describes the methodologies used in the PMSS calculation:

Development of the PMWS

The PMWS storm-based approach is specific to the characteristics of the site. Past extreme events in a region are analyzed and considered transpositionable. As part of the PMWS, different storm types (such as synoptic, squall line, and hybrid) that impacted the Great Lakes region are considered in order to determine the storm event that will generate the maximum surge and seiche. Each storm's input parameters are quantified and plotted based on the location of low/high pressure centers, concurrent wind/pressure fields, and how they evolve through time and space.

Most of the synoptic storms occur in association with deep areas of low pressure which move through the region from southwest to northeast. The general synoptic pattern is one in which the deep area of low pressure results in a very strong pressure gradient force between its low pressure center and a corresponding region of higher pressure to the north or west. The larger the gradient between the two systems over a given distance is, the stronger the resulting winds.

Squall line (or derecho) events create a widespread straight-line windstorm that is associated with a fast-moving band of severe thunderstorms. These winds have produced some of the highest instantaneous gusts on record, but last for only a short time (less than 30 minutes) at a given location. The short duration of these events, as they quickly traverse a given location, mean they will not control the PMSS. Further, these events do not occur within deep low pressure systems or remnant tropical systems. Therefore, their wind and pressure data are not combined with the other storm types in this analysis, as this would result in a PMWS that is not physically possible.

Although deep low pressure systems often produce the longest duration large-scale winds, other storm types also produce strong winds over the region. In rare cases, land-falling tropical systems along the Gulf Coast or Atlantic Seaboard move inland across the Appalachians or up the Mississippi and Ohio River valleys. By the time these storms reach the Great Lakes region, they are no longer tropical systems, but instead have transitioned into extra-tropical cyclones. Their general circulation and center of deep low pressure persists. Much like the deep low pressure scenario previously discussed, strong and persistent winds can result. The remnants of Hurricane Hazel (October 1954) and Hurricane Sandy (October 2012) are classic examples of this storm type. This storm scenario provided some of the strongest winds from the northwest through the northeast directions over Lake Erie (with durations of 12 hours or more).

Delft3D Calibration

The Delft3D hydrodynamic model is set up based on the Delft3D software suite. The wave setup contribution to the total storm surge values are modeled by coupling the Delft3D-WAVE and Delft3D-FLOW surge models. The general approach to storm surge modeling using coupled Delft3D-FLOW and Delft3D-WAVE models consists of the following steps:

- Developing the bathymetric dataset and model grid mesh for the lake system;
- Assembling input files for atmospheric forcing (wind and pressure fields);
- Assembling input files for initial water level, boundary conditions, and the physical and numerical parameters of the model;
- Assembling measured water levels and wave data for model calibration and verification;
- Testing and refining the initial model setup;
- Validating the model for historical extreme storm events; and

- Assessing model sensitivity to various factors and adjustable parameters such as bottom friction and wind drag coefficient.

The Delft3D model is calibrated based on historical data obtained from NOAA meteorological and water level recording stations located in the Lake Erie region.

Review of historical data shows that various parts of Lake Erie respond differently to any one particular storm. The storm that produces extreme water levels in one part of Lake Erie might not, and probably does not, produce extreme levels in other parts. Therefore the number of calibration and validation storms selected, to assess model prediction accuracy, covered all parts of the lake shoreline.

Calibration and verification of the coupled Delft3D-FLOW and Delft3D-WAVE models is performed by a time series comparison of measured and predicted/modeled storm surge values at different water level recording stations on Lake Erie. A similar time series comparison is also performed for wave heights.

The Delft3D models are calibrated using extreme historic wind and pressure data from multiple meteorological and water level recording stations. Calibration and verification of the coupled Delft3D-FLOW and Delft3D-WAVE models demonstrates that the hydrodynamic model is capable of computing the storm surge and seiche dynamics for Lake Erie, as well as the significant wave heights and periods at DBNPS from PMWS events.

PMSS

The calibrated Delft3D model is used to determine the PMSS. The historic wind and pressure field data is replaced with candidate PMWS events, and the model is run to determine the critical PMWS.

JLD-ISG-2012-06 and ANSI/ANS 2.8-1992 require the antecedent water level equal to the 100-year maximum recorded water level to be applied as the initial storm surge model still water level. The 100-year water level of 175.05 meters-IGLD85 is used as the initial condition/antecedent water level in all the Delft3D-FLOW models. Since the probable minimum low water level at DBNPS could occur at a time when the monthly mean lake level is at the long-term mean low probable level, the antecedent water level for low water evaluation is set to the long-term low probable level at Lake Erie, which is equal to 173.13 meters-IGLD 85

Various topographic features may affect the storm surge propagation towards DBNPS. The elevated area around DBNPS is protected along the northern, eastern, and along a small portion of the southern sides by an earthfill wave protection dike built up to 591.00 ft-IGLD55. The purpose of the wave protection dike is to protect against the surge and associate wave run-up. Additionally, the DBNPS area along the southern, western, northern, and eastern sides of the plant is protected by a vehicle barrier system (VBS).

Maximum Historical and 25-year Storm Surge

The historical maximum storm surge is the largest of the determined yearly maximum storm surge heights. The historical maximum storm surge height will be used in combined flooding scenarios in a separate calculation.

Storm surges are calculated from monthly data as the difference between monthly maximum and monthly mean based on guidance provided by USACE. The Log Pearson Type III distribution is the commonly accepted frequency procedure for annual maximum water levels. A frequency analysis on the yearly maximum storm surge heights obtained from the Toledo and Marblehead stations is performed using a Log Pearson III statistical analysis. The 25-year storm surge height was used in combined flooding scenarios.

3.5.4. Results

Simulations of all the candidate PMWS events showed that the critical PMWS event is the October 2012 wind storm event, which is the remnants of Hurricane Sandy. This storm is the most intense of all the PMWS events with a maximum wind speed of 103.12 miles/hour and is aligned along the axis of Lake Erie which is in the northeast direction. The maximum PMSS resulting from this PMWS event produced a maximum water surface elevation of 585.90 ft-IGLD55 at the intake forebay location.

The Delft3D modeling results show that the Lake Erie dike, will be overtopped from the PMSS event. Overtopped water will accumulate behind the Lake Erie dike in the marsh and low elevation areas around DBNPS, establishing a higher mean water level (i.e. ponded water) around the plant. As the surge recedes to Lake Erie, the accumulated water is forced to return to Lake Erie (return flow) along the western and southern VBS. Eventually, the water will overtop the southern and western VBS during the recession of surge water to Lake Erie, and flood DBNPS.

The maximum PMSS water surface elevation in the vicinity of the power block due to the critical PMWS is 585.81 ft-IGLD55. The PMSS water surface elevations will remain above the site finish floor elevation (585 ft-IGLD55) for approximately 2.5 hours.

Surge, Seiche, and Resonance

Results from Calculation C-CSS-020.13-016 show that the level of the rise due to seiche is significantly less than the calculated surge height. For this reason, seiches are not the controlling flood event at DBNPS.

Resonance generated by waves can cause problems in enclosed water bodies such as harbors and bays when the period of oscillation of the water body is equal to the period of the incoming waves. The period of oscillation of Lake Erie determined in Calculation C-CSS-020.13-016 is in the range of 12 to 15 hours. This is much greater than that of the peak spectral period of the incident shallow water storm waves. Consequently, resonance is not a detriment at DBNPS during the critical PMWS event.

Probable Minimum Low Water Level resulting from the PMWS

Simulations of all the candidate PMWS storm events show that the critical PMWS event that would result in probable minimum low water level (drawdown) is the transposed January 1978 storm event. This storm had the most intense west and southwest winds of the examined storm events, with maximum southwest wind speed of 89.0 miles/hour. The probable minimum low water elevation (drawdown) associated with the transposed January 1978 storm produces a probable minimum low water level of 547.46 ft-IGLD55 at the western basin of Lake Erie. At this probable minimum low water level, the DBNPS intake canal is completely cut off from Lake Erie for approximately 43 hours.

3.6. Tsunami Assessment (Reference DBNPS 2013j)

NUREG/CR-6966 identifies that earthquakes, landslides, and volcanoes can initiate tsunamis, with earthquakes being the most frequent cause. Dip-slip earthquakes (due to vertical movement) are more efficient at generating tsunamis than strike-slip earthquakes (due to horizontal movement). To generate a major tsunami, a substantial amount of slip and a large rupture area is required. Consequently, only large earthquakes with magnitudes greater than 6.5 on the Richter scale generate observable tsunamis.

3.6.1. Basis of Inputs

- NOAA natural hazards tsunami database
- NOAA natural hazards volcano database
- Historical earthquake hazards database
- Ohio Department of Natural Resources (ODNR) database

3.6.2. Models Used

- None

3.6.3. Methodology

As identified by NUREG/CR-7046, tsunami assessment is referenced to NUREG/CR-6966 and NOAA Technical Memorandum OAR PMEL-136. In addition, the more recently issued NRC guidance, JLD-ISG-2012-06, also addresses tsunami assessment. However, JLD-ISG-2012-06 provides guidance on detailed tsunami modeling and is beyond the scope of this assessment. Technical Memorandum OAR PMEL-136 reflects a similar tsunami screening assessment described by NUREG/CR-6966.

The NUREG/CR-6966 screening assessment is based on a regional screening and a site screening. The regional screening consists of researching historical records for tsunami records and the potential for tsunami-generating sources. The site screening evaluates the site based on the horizontal distance from a coast, the longitudinal distance measured along a river, and the grade elevation in comparison to the effects of a tsunami. This assessment approach is based on a review of historical records and databases.

NUREG/CR-6966 identifies that tsunamis are generated by rapid, large-scale disturbances of a body of water. The most frequent cause of tsunamis is an earthquake; however, landslides and volcanoes can also initiate tsunamis. Because of the tsunami-generation sequence associated with earthquakes, dip-slip earthquakes (due to vertical movement) are more efficient at generating tsunamis than strike-slip earthquakes (due to horizontal movement). Furthermore, to generate a major tsunami, a substantial amount of slip and a large rupture area is required. Consequently, only large earthquakes with magnitudes greater than 6.5 on the Richter scale generate observable tsunamis.

As part of the assessment, the NOAA natural hazards tsunami database was used to review historical tsunami events and associated run-ups for the east coast of the United States and Canada. Of the total events, there were 7 tsunami events that produced 14 run-ups occurring in the Great Lakes region from 1755 to 1954. The USGS hazard fault database findings were reviewed for strong earthquakes or the vertical displacements necessary to induce a tsunami. Additionally, the USGS earthquake hazards program is reviewed for historical earthquakes in the region. Lastly, the NOAA natural hazards volcano database is reviewed to assess volcanoes in the Lake Erie region.

An earthquake-generated tsunami in Lake Erie would require a very large earthquake on the order of magnitude 7.0 or greater along with significant vertical displacement.

Historically, in the Lake Erie region, the largest earthquakes are in the magnitude 5.0 range. Preliminary analysis of post-glacial sediments in the region has not yielded evidence of a large earthquake in the last few thousand years. Furthermore, earthquakes in the region, for which sufficient data are available, show primarily horizontal rather than vertical movement, which is not as conducive to tsunami generation.

Tsunamis can also be generated by the downslope movement of a very large volume of rock or sediment, either from a rockfall above the water or from a submarine landslide. Although large amounts of unconsolidated sediments are washed into Lake Erie each year when shoreline bluffs are undercut by wave action, these masses lack sufficient volume and rapid collapse to displace a volume of water that would create a tsunami. Lake Erie also has a very gentle bottom profile, particularly in the western and central basins. The eastern basin has steeper slopes, but not steep enough for a large amount of sediment to suddenly flow downslope in a submarine landslide.

Lastly, according to the NOAA natural hazards volcano database, there are no volcanoes in the Lake Erie region.

3.6.4. Results

The NOAA natural hazards tsunami database identifies only two occurrences of non-seiche (or non-wind-induced) tsunami events in the Great Lakes region. The two occurrences yielded slight or small wave effects. Various earthquake databases, including the USGS Earthquake Hazards Program earthquake database, the National Center for Earthquake Engineering research catalog, and Natural Resources Canada, identify that the largest events in the vicinity are no greater than magnitude 5.0.

According to the USGS Earthquake Hazards Program, the hazard fault database contains no known Quaternary faults (or current faults) in this region because geologists have not found any faults at the Earth's surface. Consequently, there is not a potential for strong earthquakes or the vertical displacement necessary to induce a tsunami.

Therefore, a tsunami is not expected to be the controlling flood event at DBNPS.

3.7. Combined Effect Flood (including Wind-Generated Waves) (Reference DBNPS 2013n)

Evaluation of the shoreside location is covered in H.4.1 of NUREG/CR-7046 and includes one alternative:

Combination of:

- Probable maximum surge and seiche with wind-wave activity.
- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

There are three alternatives specified in H.4.2 of NUREG/CR-7046 for streamside locations. Each of the alternatives considered has three components contributing to the water surface elevation.

- **Alternative 1 – Combination of:**
  - The lesser of one-half of the PMF or the 500-year flood;
  - Surge and seiche from the worst regional hurricane or windstorm with wind-wave activity; and

- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

- **Alternative 2 – Combination of:**
  - PMF;
  - A 25-year surge and seiche with wind-wave activity; and
  - The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

- **Alternative 3 – Combination of:**
  - A 25-year flood;
  - Probable maximum surge and seiche with wind-wave activity; and
  - The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

3.7.1. Basis of Inputs

Inputs include the following:

- Toussaint River sub-watershed properties for rainfall-runoff modeling from Calculation C-CSS-020.13-011
- Toussaint River HEC-RAS model from Calculation C-CSS-020.13-011
- PMF discharge hydrographs from Calculation C-CSS-020.13-011
- One-half PMF discharge hydrographs
- 25-year event rainfall for input into HEC-HMS model from NOAA Atlas 14
- 25-year event discharge hydrographs
- Lake Erie Probable PMSS elevations from Calculation C-CSS-020.13-017

3.7.2. Computer Software Programs

- ArcGIS Desktop 10.1
- Delft 3D
- HEC-HMS 3.5
- HEC-RAS 4.1
- Microsoft Excel

3.7.3. Methodology

Each combination includes coincident wind-wave activity. Coincident wind-wave activity is determined for the critical flooding combination using the USACE guidance outlined in USACE *Coastal Engineering Manual*. Runup is the maximum elevation of wave uprush above stillwater level.

H 4.1 Combination

Probable maximum surge and seiche is estimated in Calculation C-CSS-020.13-017. Wind-wave activity includes wave height, wind set-up, and wave runup. Wave height and wind set-up are included as part of the PMSS developed using Delft3D model.

H.4.2 Alternative 1

Alternative 1 requires using the lesser of one-half of the PMF or the 500-year event. In this case, the PMF was already determined as part of Calculation C-CSS-020.13-011. Therefore, the one-half PMF is utilized as described in C-CSS-020.13-022. The surge and seiche height from the worst regional hurricane or windstorm with wind-wave activity is estimated using statistical analysis of the historical data. Lake Erie has no outlet control structures. Therefore, the 100-year water surface elevation is used without further consideration of the maximum controlled water elevation. The HEC-RAS model developed as part of the PMF analysis is revised to use the one-half PMF as the inflow boundary condition and 100-year surge-seiche elevation from the worst regional hurricane as the downstream boundary condition. The HEC-RAS model provides the water surface elevation for this alternative.

H.4.2 Alternative 2

Alternative 2 requires using the PMF estimated in Calculation C-CSS-020.13-011. The surge and seiche height from the 25-year event is estimated using statistical analysis of the historical data. Similar to Alternative 1, the HEC-RAS model was updated to obtain the resulting water surface elevation.

H.4.2 Alternative 3

Alternative 3 requires using the 25-year flood in the Toussaint River. Point rainfall data from NOAA was used to estimate the 25-year rainfall event. This rainfall is input into the HEC-HMS model developed as part of the PMF analysis to estimate runoff due to 25-year event. The water surface elevation for the probable maximum surge and seiche in combination with the 100-year water level is determined in Calculation C-CSS-020.13-017. Similar to Alternatives 1 and 2, the HEC-RAS model was updated to obtain the resulting water surface elevation.

3.7.4. Results

The predicted water surface elevation at the site for Alternative 3 is found to be the maximum for the alternatives specified in H.4.2. It is also concluded that the Toussaint River water surface elevations at the site are completely controlled by the backwater conditions at Lake Erie for that alternative (i.e., the predicted water surface elevation in the river is equal to the lake elevation, extending for about one mile upstream of DBNPS).

The water surface elevation of 585.93 ft-IGLD55 for combination H.4.1 is equal to the critical water surface elevation for combination H.4.2 Alternative 3 and represents the critical water surface elevation at the site resulting from the combined events as specified in NUREG CR-7046, Appendix H.4.

DBNPS is protected against flooding due to wave runup during a PMWS by wave protection dikes installed along the northern, eastern, and along a small portion of the southern sides of the site to an elevation of 591 ft-IGLD55. Wave action at DBNPS is governed by the maximum supportable wave at the toe of the north dike during the PMSS. Wave action analysis concludes that a maximum wave runup of 3.98 ft on top of the PMSS level of 585.90 ft-IGLD55 may be generated at the toe of the north dike. The maximum wave runup elevation during the controlling flooding Alternative is therefore equal to 589.88 ft-IGLD55. This elevation is less than the top of the wave protection dikes (591 ft-IGLD55). Therefore, the wave runup analysis shows that wave protection dikes are sufficient to protect DBNPS from wave runup during the critical combined flooding event.

DBNPS is flooded during the PMWS event along the non-diked west and south site boundaries. The maximum PMSS water surface elevation in the vicinity of the power block is 585.81 ft-IGLD55. The maximum wave runup elevation in the vicinity of the power block is 585.90 feet-IGLD55.

3.8. Local Intense Precipitation (Reference DBNPS 2013e and DBNPS 2013f)

The LIP is an extreme precipitation event (high intensity/short duration) at a given location. The duration of the event and the coverage area are needed to quantify an extreme precipitation event fully. Generally, the amount of precipitation decreases with increasing duration and increasing area. NUREG/CR-7046 specifies that the LIP should be equivalent to the 1-hr, 1-mi^2 probable maximum precipitation (PMP) at the location of the site.

3.8.1. Basis of Inputs

- Site topography
- LIP (cumulative and incremental)
- Manning's roughness coefficients
- Supporting GIS data (Reference DBNPS 2013l)

3.8.2. Models Used

- ArcGIS Desktop 10.1
- FLO-2D Pro
- Microsoft Excel

3.8.3. Methodology

The LIP event was evaluated to determine the associated flooding elevation and velocities assuming the active and passive drainage features are non-functional. The entire roof drainage is assumed to be contributing to the surface runoff. The LIP evaluation was performed in accordance with the NUREG/CR-7046.

The runoff caused by the LIP event was estimated using the FLO-2D software. The software uses shallow water equations to route stormwater throughout the site. FLO-2D depicts site topography, using a digital elevation model (DEM), to characterize grading, slopes, drainage divides, and low areas of the site. The DEM is a grid model developed from composite ground surface information. The methodology used within the FLO-2D software included the rainfall function and the levee function (to incorporate site security features which could impact the natural drainage characteristics of the site).

Per NUREG/CR-7046, the 1-hr, 1-mi^2 PMP event was developed using HMR- 52. The total PMP depth per square mile for the 1-hr event was interpolated from the PMP depth contour map provided in HMR-52. The distribution of the 1-hr PMP was developed for the 5-, 15-, and 30-minute time intervals, with the 60-minute interval being the 1-hr PMP depth. The depth for each time interval was calculated using the guidance provided in HMR-52. The 1-hr PMP was modeled in FLO-2D to calculate the subsequent site flooding.

Active and passive drainage system components (e.g., pumps, gravity storm drain systems, small culverts, and inlets) were considered non-functional or clogged during the LIP event, per Case 3 in NUREG/CR-7046. The Manning's roughness coefficient values are selected based on the land cover type using the guidance provided in the FLO-2D manual. Two types of obstructions are modeled: buildings/structures that completely block the water passage, and security wall barriers that could be overtopped if the water depth increases to above the top of the wall.

To determine the flooding elevation associated with the LIP, the 1-hr, 1-mi^2 storm was applied evenly across the site, and the model was allowed to run for 2.5 hours to ensure that only the areas of static ponding would remain. Five temporal distributions similar to the PMF analysis were considered.

3.8.4. Results

The end temporal distribution of the LIP event resulted in the highest water depths and consequently in the highest water surface elevations. The water surface elevations at critical door locations, or doors leading to safety-related SSCs, are listed in Table 1.

Table 1 – LIP Flooding Elevation and Duration at DBNPS

Structure	Door Number	Maximum Water Surface Elevation (ft-IGLD55)	Flood Duration above 585 ft-IGLD55 (minutes)
Auxiliary Building	300	585.25	6
Auxiliary Building	361	585.20	15
Auxiliary Building	362	585.20	15
Auxiliary Building	315	585.17	15
Auxiliary Building	320A	585.17	15
Auxiliary Building	324	585.18	18
Turbine Building	330	585.18	18
Turbine Building	399A	585.19	18
Turbine Building	339	585.20	18
Turbine Building	333	585.22	18
Turbine Building	334	585.41	57*
Intake Structure	224	585.44	30

* The difference in the flooding duration at Door 333 and Door 334 is caused by the ground surface elevations. The ground at Door 334 is higher than the ground at Door 333 (585.92 vs. 585.67 ft NAVD88). Therefore, minimal flooding depths result in the water level increase above the floor elevation. Between minute 27 and minute 54 of flooding event the water depth above 585.0 is 0.01 to 0.05 ft.

The hydrodynamic loads, or impact loads, on the structures due to the LIP are presented in Table 2. FLO-2D reports the impact pressure as a force per unit length (impact pressure times flow depth). The maximum impact force on the structure was estimated by multiplying the impact pressure by the structure length.

Table 2 – Maximum Impact Loads on Buildings

Building	Max Impact (lb/ft)
Containment Structure	0.46
Auxiliary Building	1.53
Turbine Building	7.14
Intake Structure	5.27

4. COMPARISON WITH CURRENT DESIGN BASIS

The reevaluated maximum water surface elevation due to the riverine flooding (Toussaint River) is below the current licensing basis. The reevaluated maximum water surface elevation due to the LIP and PMSS events exceed the current licensing basis.

For lake flooding, the current design basis assumes the surge only in one direction. A site-specific wind and pressure field is developed as part of the re-analysis. More recent storms provided the controlling wind for surge flooding at DBNPS.

For LIP flooding, the current design basis assumes 24.5 inches of rainfall will pond evenly across the site. As part of the re-analysis, recent site topography was used in a two-dimensional hydraulic model (FLO-2D Pro) and additional rainfall estimates were obtained from the more recent HMR-52 guidance.

In the interim, it is understood that an event of such magnitude to approach the postulated accumulation of rainfall is a low probability event. Such an event would likely be associated with a significant tropical storm. Meteorological forecasting would provide sufficient warning well in advance of such an event. The Interim Actions discussed in Section 5 will provide adequate protection until permanent solutions are implemented.

The comparisons of existing and reevaluated flood hazards are provided in Table 3.

Table 3 – Comparison of Existing and Reevaluated Flood Hazards at DBNPS

Flood-Causing Mechanism	Design Basis	Comparison	Flood Hazard Reevaluation Results
Flooding in streams and rivers	PMF Elevation – 579 ft-IGLD55 PMF Flow – 78,500 cfs Cool-season PMP was not evaluated.	Bounded	All-Season PMF Elevation – 575.96 ft-IGLD 55, All-Season PMF Flow – 100,436 cfs Cool-Season PMF Elevation – 575.06 ft-IGLD 55, Cool-Season PMF Flow – 61,943 cfs
Dam breaches and failures	No dams or other regulating hydraulic structures.	Bounded	Dam assessment indicated no critical dams.
Storm surge	Water surface elevation – 583.7 ft IGLD55	Not bounded. Exceeds current design basis.	Water surface elevation –585.81 ft-IGLD55 at power block.
Seiche	This flood-causing mechanism is not described in the USAR.	This flood-causing mechanism is not described in the USAR.	Not a credible scenario. Bounded by storm surge.
Tsunami	This flood-causing mechanism is not described in the USAR.	This flood-causing mechanism is not described in the USAR.	Tsunami assessment indicates there is a slight possibility of tsunamis in Great lakes. However, the seismicity in the region suggests there is no potential for strong earthquakes or the vertical displacement necessary to induce a substantial tsunami.
Ice-induced flooding	Not plausible	Bounded	Ice-induced flooding is bounded by the all-season PMF event.
Channel migration or diversion	As indicated in the USAR, the mean lake level is not subject to variations due to diversions or source cutoff.	Bounded	Channel diversion towards the site is not probable.

Table 3 – Comparison of Existing and Reevaluated Flood Hazards at DBNPS (Continued)			
Flood-Causing Mechanism	**Design Basis**	**Comparison**	**Flood Hazard Reevaluation Results**
Combined effect flood (including wind-generated waves)	Wave runup on wave protection dike- 590.3 ft-IGLD55.	Bounded	Wave runup on wave protection dike-589.88 ft-IGLD55. Maximum wave runup elevation in the vicinity of the power block – 585.90 ft-IGLD55.
LIP	Maximum water surface elevation – 584.5 ft-IGLD55.	Not bounded. Exceeds current design basis.	Maximum water surface elevation – 585.44 ft-IGLD55.
Dam Failure	As indicated in the USAR, there are no structures on Toussaint River that can affect the flow hydrograph at DBNPS.		Dams located in the DBNPs watershed are determined to be noncritical.

5. INTERIM AND PLANNED FUTURE ACTIONS

The Flooding Hazard Reevaluation Report evaluated applicable flooding hazards for DBNPS. Two of the postulated reevaluated flood hazard events, the PMSS and the LIP events, resulted in maximum flood water elevations higher than previously calculated for DBNPS. The assessment of the buildings, resulting from the flood hazard reevaluation, found a number of doors leading to areas containing safety related equipment to be susceptible to the postulated water infiltration. These postulated flooding events are considered beyond design basis events. The reevaluated flood levels are small increases with short durations. These low probability events would likely be identified in advance by meteorological forecasting. Current plant procedures addressing flooding at the site provide actions to be taken in the event flooding is imminent or has occurred at or near the DBNPS site. No additional actions beyond those currently in place are necessary at this time. The total plant response to the reevaluated hazard is to be determined by the Integrated Assessment.

The integrated assessment will be performed to address the need and/or potential designs for temporary or permanent barriers (or alternative countermeasures) to prevent postulated flood water infiltration and/or mitigation of the postulated flood water infiltration. This evaluation will also include a study of emergency procedures. The evaluation of the mitigating strategy and schedule for the implementation of modifications (as necessary) will be documented in the Corrective Action Program. The evaluation will address the following items:

<u>LIP</u>

As indicated earlier, the water surface elevation exceeds the finish floor elevation of 585 ft-IGLD55 by a maximum value of 0.44 ft for a total duration of approximately 1.0 hrs at one (1) door and 0.5 hrs or less at the remaining eleven (11) affected doors, due to the LIP event. The LIP storm mechanisms will be reviewed in the integrated assessment to establish trigger points which support implementation of proceduralized mitigation measures. This includes an evaluation of the forecast information available to personnel that would allow for advanced monitoring and warning of meteorological conditions that could potentially result in an LIP event occurring at the site. Modifications will also be considered to afford protection for the site vulnerabilities or to further reduce the impact of the LIP.

<u>PMSS</u>

As indicated earlier, the water surface elevation exceeds the finish floor elevation of 585 ft-IGLD55 by a maximum value of 0.81 ft for a total duration of approximately 2.5 hrs due to the PMSS event. The PMSS storm mechanisms will be reviewed in the integrated assessment to establish trigger points which support implementation of proceduralized mitigation measures. This includes an evaluation of the forecast information available to personnel that would allow for advanced monitoring and warning of meteorological conditions that could potentially result in a surge event affecting the site. Modifications will also be considered to afford protection for the site vulnerabilities.

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