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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Contents

1. PURPOSE

a. Background

In response to the nuclear fuel damage at the Fukushima-Dai-ichi power plant due to the March 11, 2011, earthquake and subsequent tsunami, the United States Nuclear Regulatory Commission (NRC) established the Near Term Task Force (NTTF) to conduct a systematic review of NRC processes and regulations, and to make recommendations to the 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) (NRC March 2012) 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 (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 Exelon Generation Company, LLC (Exelon) for the Dresden Nuclear Power Station (DNPS), this Flood Hazard Reevaluation Report (Report) provides the information requested in the March 12, 50.54(f) letter; specifically, the information listed under the 'Requested Information' section of Enclosure 2, paragraph 1 ('a' through 'e'). 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.

b. Requested Actions

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

For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard with the hazard evaluation.

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

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

c. Requested Information

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

a. 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:

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

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

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

the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, include how this finding was determined.

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

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

2. SITE INFORMATION

The DNPS site is located at the confluence of the Des Plaines and Kankakee rivers that forms the Illinois River. The DNPS site is also located just upstream of the Dresden Island Lock and Dam, which is owned and operated by the U.S. Army Corps of Engineers (USACE). The nominal ground elevation of the DNPS site is about 516 feet mean sea level (msl) datum at the location of the principal structures of Units 2 and 3, and the design plant grade is 517 feet msl. The finished floor elevation of the plant structure is 517.5 feet msl. Safe operation of the plant during the PMF is accomplished via implementation of flood emergency procedures (Dresden UFSAR).

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

The current design basis is defined in the DNPS UFSAR (Dresden UFSAR) and by reference to an NRC Systematic Evaluation Program (SEP) (FRC 1982). The following is a list of flood causing mechanisms and their associated water surface elevations that were considered for the DNPS current design basis.

1. **LIP –** LIP is addressed under a separate report titled, "Local Intense Precipitation Evaluation Report for the Dresden Nuclear Power Station" (Exelon 2013p).

2. **Flooding in Streams and Rivers –** The UFSAR (Dresden UFSAR) by reference to the SEP (FRC 1982) identifies that a flow rate of 490,000 cfs in the Illinois River at the DNPS site would result in a stillwater flood elevation of 525 feet msl. Adding wave runup to the stillwater flood elevation yields a site PMF elevation of 528 feet msl. Safe operation of the plant during the PMF is accomplished via implementation of flood emergency procedures.

   By reference to the SEP, the PMF is based on a 72-hour PMP storm duration. The PMP is developed using USACE guidance. The approximate 7300 square mile watershed is divided into 13 sub-basins and HEC-1 software is used to transform rainfall to runoff using calibrated hydrographs. The Standard Step Method is used to determine the water surface elevation. HEC-2 software is used to evaluate the reach of the Illinois River between the Dresden Island Lock and Dam and the confluence of the Kankakee and Des Plaines River.

3. **Dam Breaches and Failures –** Dam failure is addressed in the UFSAR (Dresden UFSAR) for the upstream Dresden Cooling Lake and the downstream Dresden Lock and Dam. Assessment of the Cooling Lake is based on instantaneous failure and release of the entire lake into the approximate 4000 acre flood plain of the Kankakee River. Under these conditions the water level could rise 3.2 feet for a short period of time above the normal river elevation of 505 feet msl.

The plant is designed so that it can be safely shut down in the event of failure of the Dresden Lock and Dam and the pool impounded by it. There is enough water impounded by the intake and discharge canals below their high point elevations to allow a safe shutdown of Dresden Units 2 and 3.

4. **Storm Surge –** The UFSAR (Dresden UFSAR) indicates flooding due to surges is not applicable. DNPS has an inland location and does not connect directly with any of the water bodies considered for meteorological events associated with a storm surge. Flooding due to a surge is not plausible at DNPS.

5. **Seiche –** The UFSAR (Dresden UFSAR) indicates flooding due to seiches is not applicable. DNPS has an inland location and does not connect directly with any of the water bodies considered for meteorological events associated with a seiche. Flooding due to a seiche is not plausible at DNPS.

6. **Tsunami –** The UFSAR (Dresden UFSAR) indicates flooding due to tsunamis is not applicable. DNPS has an inland location and does not connect directly with any of the water bodies considered for tsunami events. Flooding due to a tsunami is not plausible at DNPS.

7. **Ice Induced Flooding –** The UFSAR (Dresden UFSAR) indicates an 8-foot diameter deicing line connects the discharge canal headworks and the crib house forebay. Its high point is in the headworks at elevation 495 feet msl and its low point is in the forebay at elevation 489 feet msl. A slide gate valve is used to isolate the deicing line when not in use. This feature is designed to prevent the cribhouse intake bay from freezing and is not designed to prevent ice induced flooding.

   Under certain condition, ice forms on the Kankakee River and will start to flow in immense sheets. To mitigate this problem, the Army Corps of Engineers installed three siphon lines connecting the Dresden Cooling Lake to the Kankakee River. The siphon lines are located on the eastern most part of the Dresden Cooling Lake (along Cottage Road). The requirements and limitations for operating of the siphon lines (Exelon DOP 4450-09) are currently contained in the Dresden National Pollutant Discharge Elimination System (NPDES) permit. The Illinois Environmental Protection Agency (IEPA) allows the siphon to be used twice during the winter seasons for two-week intervals. When severe icing conditions occur on the Kankakee River, IEMA contacts Dresden Station alerting the station to initiate the siphon system.

8. **Channel Migration or Diversion –** By reference to the UFSAR (Dresden UFSAR), the authority to control the Dresden Island Lock and Dam, and therefore, the level of the pool impounded by it, is vested in the USACE. Should the dam become damaged, one method of maintaining the pool level would be through the use of increased water diversion from Lake Michigan. This would be accomplished with the permission of the U.S. Supreme Court.

9. **Combined Effect Flood (including Wind-Generated Waves) –** The UFSAR (Dresden UFSAR) by reference to the SEP (FRC 1982) identifies that adding wave runup to the stillwater flood elevation yields a site PMF elevation of 528 feet msl. By reference to the SEP, coincident wind wave activity is based on a 2-year wind speed of 50 miles per

hour and the procedures outlined in the USACE Shore Protection Manual. The waves would be 2.6 feet high and produce a runup of 3 feet above stillwater level.

b. Flood Related Changes to the License Basis and Any Flood Protection Changes (including Mitigation) Since License Issuance

Although no formal change to the plant's license basis, Dresden is in the process of implementing strategies to better cope with PMF flooding under the current licensing basis. These include installing flood barriers to the Isolation Condenser Makeup Pump Building, flood barriers to protect reactor building areas including Unit 2/3 Emergency Diesel Generator and staging the diesel driven pump on a floating dock.

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

The most significant changes to the watershed include expansion and development of the greater Chicago metropolitan area. From the 1970s to the present, the percent area of developed land use has increased from about 16 percent to 26 percent. The watershed contains 137 dams documented in the USACE National Inventory of Dams (Exelon 2013i). Of these, construction of 57 dams has been completed since Unit 3 license issuance (1971). Construction of 30 dams has been completed since Unit 2 license issuance (1991). There are 27 dams without a documented completion date. However, there are no large storage volume dams. As discussed in the dam failure analysis (Exelon 2013j), all dams are included in the analysis. Security barrier upgrades have been added to the site since license issuance.

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

The current license basis indicates when water level rises above 517 feet msl, the doors to the reactor building will be opened and water will be allowed to flood the plant. Per DNPS flooding procedure DOA 0010-04 (Exelon DOA 0010-04), the flood pump will be placed on a floating platform that will rise and fall with the flood level. The flood pump will operate at an elevation above the reevaluation maximum stillwater elevation. The strategy includes provision for safe shutdown when water levels reach elevation 509 feet msl.

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* (NRC 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 the DNPS site, the combination that produces the highest flood water elevation at the site is the 100-year, cool-season rainfall, the probable maximum snowpack, upstream dam failure, and the effects of coincident wind wave activity.

The UFSAR Section 2.4 provides elevations without datum reference. However, by reference to the SEP the datum is defined by mean sea level (msl). The hazard reevaluation calculations provide elevation results based on the National Geodetic Vertical Datum (NGVD 29). Prior to 1973, NGVD 29 was named the Sea Level Datum of 1929. The SEP defines the normal water surface elevation at Dresden Lock and Dam to be elevation 505 feet msl. Based on USGS quadrangle mapping, the normal water surface elevation at Dresden Lock and Dam is also defined as elevation 505 feet NGVD 29. By association, NGVD 29 is equivalent to the mean sea level datum

reported in the SEP. There is no evidence to suggest that mean sea level reported in the SEP is in reference to a datum other than NGVD 29.

Calculation WGW-DRE-001 (AMEC 2013b) defines the maximum stillwater flood levels at the site safety-related and other critical structures as provided in Table 1. The maximum stillwater flood level at a safety-related structure, Unit 2 Reactor Building, is 524.76 feet NGVD 29. The maximum stillwater elevation is 7.76 feet above the site grade elevation of 517 feet NGVD 29, and 0.24 feet below the existing design basis stillwater flood elevation of 525 feet NGVD 29. Calculation WGW-DRE-001 (AMEC 2013b) defines the maximum water surface elevations including the effects of coincident wind wave at the site safety-related and other critical structures as provided in Table 1. The maximum elevation with added runup at a safety-related structure, Unit 2 Reactor Building, is 528.86 feet NGVD 29. The maximum elevation with added runup is 0.86 feet above the existing design basis flood elevation of 528 feet NGVD 29.

Table 1. Summary of Maximum Stillwater Elevation and Wave Runup Elevation

Structure	Maximum Stillwater Elevation (feet NGVD 29)	Maximum Runup Elevation (feet NGVD 29)
Reactor Building, Unit 1	524.88	526.48
Reactor Building, Unit 2	524.76	528.86
Reactor Building, Unit 3	524.61	527.16
Turbine Building, Unit 1	524.22	524.77
Turbine Building, Unit 2	524.61	529.01
Turbine Building, Unit 3	524.35	525.75
ISFSI-East	524.85	527.35
ISFSI-West	524.27	527.87
HPCI & Diesel Generator	524.60	528.75

Note: ISFSI – Independent Spent Fuel Storage Installation
HPCI – High Pressure Coolant Injection

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

- NRC Standard Review Plan, NUREG-0800, revised March 2007 (NRC NUREG-0800);
- NRC Office of Standards Development, Regulatory Guides, RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976 (NRC RG 1.102);
- RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977 (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 (NRC NUREG/CR-7046);
- American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS-2.8-1992), dated July 28, 1992;
- NEI Report 12-08. Overview of External Flooding Reevaluations (NEI August 2012).

The following provides the flood causing mechanisms and their associated water surface elevations that are considered in the DNPS flood hazard reevaluation.

1. **Flooding in Streams and Rivers –** The PMF is a function of the combined events defined in NUREG/CR-7046 (NRC NUREG/CR-7046) for floods caused by precipitation events.

Alternative 1 – Combination of:

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

Alternative 2 – Combination of:

- Mean monthly base flow
- Probable maximum snowpack
- A 100-year, snow-season rainfall
- Waves induced by 2-year wind speed applied along the critical direction

Alternative 3 – Combination of:

- Mean monthly base flow
- A 100-year snowpack
- Snow-season PMP
- Waves induced by 2-year wind speed applied along the critical direction

a. <u>Basis of Inputs</u>

PMP

The all-season PMP is determined by using the generalized PMP estimates defined by Hydrometeorological Report (HMR) No. 51 and HMR No. 52 guidance (Exelon 2013a, 2013b, and 2013c). The 100-year, cool-season rainfall is determined using precipitation frequency estimates defined by National Oceanic and Atmospheric Administration (NOAA) Atlas 14 guidance and applying regional seasonal guidance (Exelon 2013d). The cool-season PMP is determined by applying seasonal HMR No. 53 guidance to the all-season PMP estimates (Exelon 2013d).

Snowmelt is included in two of the alternatives (Exelon 2013d). For rain-on-snow conditions, air temperature, dew point temperature, and average maximum daily wind speed are obtained from representative weather stations. The basin wind exposure factor is determined based on the density of forest stands in each sub-basin. For rain-free conditions, the snowmelt parameters are based on USACE guidance and maximizing the rain-free snowmelt.

Snowpack is assumed to be ripe at the onset of rainfall events and cover the entire watershed. Soil is assumed to be frozen with no losses during the months of December through February. Minimum saturated infiltration rates are assumed for the months of October through November and March through April. For the probable maximum snowpack, snowpack depth is assumed to be available for the duration of the coincident storm event.

Hydrologic Model

The hydrologic model is developed based on USGS gage locations to define sub-basin areas (Exelon 2013e). Soil and land use maps along with NOAA precipitation data and

USGS flow data are used to develop rainfall runoff transformation, loss rates, and reach routing parameters. During the PMP events it is assumed that the ground is saturated and initial losses are ignored. The navigational locks and dams are assumed to be operated as run-of-river with no effective storage.

Hydraulic Model

The hydraulic model is developed using USACE bathymetry and USGS topography to generate cross sections (Exelon 2013l). Dams are included based on USACE operation manuals. Bridge geometry data is based on aerial images and Illinois Department of Transportation drawings. Manning's roughness coefficients are selected based on calibration and land cover imagery. The navigational lock and dam gate operations are assumed to function as normal operations according to each respective water control manual. The pool elevations at the start of each simulation are assumed to be at the normal operating level.

b. <u>Models Used</u>

PMP

USACE HMR No. 52 software is used to optimize the storm and define the PMP estimates for each sub-basin (Exelon 2013a, 2013b, and 2013c).

Hydrologic Model

USACE HEC-HMS software is used to develop rainfall runoff transformation and reach routing for the watershed (Exelon 2013d, 2013e, 2013f, 2013g, and 2013h).

Hydraulic Model

USACE HEC-RAS software are used to evaluate unsteady-state routing to transform flow hydrographs into a water surface elevation hydrograph (Exelon 2013l and 2013m).

RiverFLO-2D software is used to refine the hydraulics near the site and FLO-2D software is used to determine on-site stillwater elevations. Discussions of these models are included in the Dam Breaches and Failures section. In addition, FLO-2D is used to determine the effects of wind wave activity. Discussion is provided in the Combined Effect Flood (including Wind-Generated Waves) section.

c. <u>Methodology</u>

Rainfall

Each combination 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.

The DNPS watershed is a combination of the individual watersheds for the Des Plaines and Kankakee Rivers. The combined watershed of approximately 7250 square miles is subdivided into 23 sub-basins for analysis (Exelon 2013c). Maximizing the runoff is

based on comparison of separate evaluations for rainfall occurring over only the Des Plaines watershed (Exelon 2013a), over only the Kankakee watershed (Exelon 2013b), or over the combination of the two individual watersheds (Exelon 2013c). For each alternative, the runoff is maximized by application of the rainfall over the combination of the two individual watersheds.

For the all-season and cool-season PMP events, different storm centers throughout the watershed are examined to determine the critical storm center that maximizes runoff. USACE HMR No. 52 software is used to optimize the storm and define the PMP estimates for each sub-basin (Exelon 2013a, 2013b, and 2013c). Because no significant variance throughout the watershed is found, the 100-year, cool-season rainfall is determined for each sub-basin and aggregated to apply as an overall average to the watershed (Exelon 2013d).

USACE HMR No. 52 software is based on a standard temporal distribution. HMR guidance indicates the greatest precipitation may occur at other positions throughout the duration of the storm. Various temporal distributions for each rainfall scenario are then evaluated to further maximize the runoff. Front, one-third, center, two-thirds, and tail end loading temporal distributions are considered in an effort to capture the distribution that maximizes runoff.

Hydrologic Model

Rainfall is applied to each sub-basin and transformed to runoff using unit hydrograph methodology. The sub-basin parameters are calibrated and validated with historic extreme events for which sufficient stream flow and rainfall data is available (Exelon 2013e). USACE HEC-HMS software is used to model and calibrate Clark Unit Hydrograph parameters and Muskingum reach routing parameters. Initial and constant loss rates are based on the soil type and calibrated. Land use is used to define the percent impervious for each sub-basin. Baseflow is calibrated for each historic event.

The calibrated unit hydrographs for each sub-basin are then 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.

Snowmelt is included in two of the alternatives. Snowmelt is determined using the Energy Budget Method (Exelon 2013d). The probable maximum snowpack is assumed to be represented by a snowpack depth available for the duration of the storm. The 100-year snowpack is determined on a monthly basis by statistical analysis of historic snow depth data obtained from weather gages. The Fisher-Tippett Type I, or more commonly named Gumbel distribution is used to estimate the 100-year snowpack. The snow water equivalent is also determined on a corresponding monthly basis using representative snow bulk density obtained from weather gage data.

Each alternative includes coincident wind waves. Coincident wind waves are addressed in the Combined Effect Flood.

Alternative 1 – All-Season PMP (Exelon 2013f, 2013g, and 2013h)

Because baseflow is variable for the historic extreme storms used for calibration and validation, baseflow is determined using the mean monthly flow from stream flow gages in the watershed. A ratio of the monthly baseflow divided by the drainage area is applied to each sub-basin area.

Although losses were calibrated using extreme storms, it is assumed that the ground is saturated at the time of the storm. Initial losses are set to zero and the minimum loss rate for the soil type is used as a constant loss. It is determined by comparison of the calibrated loss rates analysis that the minimum loss rates are more conservative.

It is determined that the 40 percent PMP is less than the 500-year storm event. Therefore, an antecedent storm equivalent to 40 percent of the all-season PMP is applied to the model with a 72-hour dry period between the antecedent storm and the PMP event.

The all-season PMP as described above is applied to the HEC-HMS model to determine flow hydrographs at key points in the model.

Alternative 2 – Probable Maximum Snowpack and 100-Year Cool-Season Rainfall (Exelon 2013d)

Because baseflow is variable for the historic extreme storms used for calibration and validation, baseflow is determined using the mean monthly flow from stream flow gages in the watershed. A ratio of the monthly baseflow divided by the drainage area is applied to each sub-basin area.

During the colder months of December, January, and February, the monthly average temperatures are below freezing. Therefore, it is assumed the ground is frozen and no losses are considered. During the months of October, November, March, and April, the monthly average temperatures remain above freezing. However, it is assumed that the soils are fully saturated. No initial losses are considered, but the minimum saturated loss rate for the soil type is considered as a constant loss rate. It is determined the controlling month for the Alternative 2 scenario is March.

As previously discussed, the Energy Budget Method is used to develop the snowmelt resulting from the probable maximum snowpack and the 100-year cool-season rainfall. The combined rainfall and snow water equivalent are applied to the HEC-HMS model to determine flow hydrographs at key points in the model.

Alternative 3 – 100-Year Snowpack and Cool-Season PMP (Exelon 2013d)

Because baseflow is variable for the historic extreme storms used for calibration and validation, baseflow is determined using the mean monthly flow from stream flow gages in the watershed. A ratio of the monthly baseflow divided by the drainage area is applied to each sub-basin area.

During the colder months of December, January, and February, the monthly average temperatures are below freezing. Therefore, it is assumed the ground is frozen and no losses are considered. During the months of October, November, March, and April, the

monthly average temperatures remain above freezing. However, it is assumed that the soils are fully saturated. No initial losses are considered, but the minimum saturated loss rate for the soil type is considered as a constant loss rate. It is determined the controlling month for the Alternative 3 scenario is October.

As previously discussed, the Energy Budget Method is used to develop the snowmelt resulting from the 100-year snowpack and the cool-season PMP. The combined rainfall and snow water equivalent are applied to the HEC-HMS model to determine flow hydrographs at key points in the model.

Hydraulic Model (Exelon 2013l and 2013m)

The controlling PMF scenario is determined to be Alternative 2, a March probable maximum snowpack and a cool-season 100-year rainfall. A HEC-RAS unsteady flow model is used to perform dynamic channel routing of inflow hydrographs from HEC-HMS and establish boundary conditions for two-dimensional hydrodynamic modeling at the site. For reference and comparison, the all-season PMP scenario, Alternative 1, is also evaluated with the HEC-RAS model.

The hydraulic model is developed for a reach of the Des Plaines and Illinois Rivers from upstream of Lockport Lock and Dam downstream to Marseilles Lock and Dam (Exelon 2013l). Brandon Road Lock and Dam and Dresden Island Lock and Dam are also included in the model. A short reach for the Kankakee River at the confluence with the Des Plaines is also included. The hydraulic model is calibrated for unsteady state flow using extreme historic flow data at multiple gage locations. Manning's roughness coefficients and weir flow coefficients are selected based on calibration and land cover imagery.

d. <u>Results</u>

Hydraulic Model

The PMF flow hydrographs are entered into the HEC-RAS model at the upstream end of the model, at the upstream of the Kankakee River reach, and at three intermediate points representing other tributary rivers (Exelon 2013m). The HEC-RAS model is evaluated using unsteady-state flow. Cross Section 135306 represents the upstream end of the plant and Cross Section 134410 represents the downstream end of the plant. For Alternative 1, the peak flow rate at Cross Section 135306 is approximately 318,000 cfs. For Alternative 2, the peak flow rate at Cross Section 135306 is approximately 380,000 cfs.

2. **Dam Breaches and Failures –** The criteria for evaluation of flooding from dam breaches and failures are provided in NUREG/CR-7046 and JLD-ISG-2013-01 *Guidance for Assessment of Flooding Hazards Due to Dam Failure* (NRC JLD-ISG-2013-01). Development of the calculations is based on NEI guidance (NEI November 2012), because JLD-ISG-2013-01 had not yet been issued. However, the NEI guidance utilized reflects the same guidance presented by JLD-ISG-2013-01. Two scenarios of dam failures are recommended and discussed in NUREG/CR-7046.

- Failure of individual dams (i.e., groups of dams not domino-like failures) upstream of the site.
- Cascading or domino-like failures of dams upstream of the site.

Three failure mechanisms are considered for dam failure analysis.

- Hydrologic
- Seismically-Induced
- Sunny Day

Seismically-induced dam failures are a function of the combined events defined in NUREG/CR-7046 for floods caused by seismic dam failures.

Alternative 1 – Combination of:
- A 25-year flood
- A flood caused by dam failure resulting from a safe shutdown earthquake (SSE), and coincident with the peak of the 25-year flood
- Waves induced by 2-year wind speed applied along the critical direction

Alternative 2 – Combination of:
- The lesser of one-half of the PMF or the 500-year flood (defined by applying the associated precipitation to the site's watershed)
- A flood caused by dam failure resulting from an operating basis earthquake (OBE), and coincident with the peak of the flood in item 1 above
- Waves induced by 2-year wind speed applied along the critical direction

Because seismically-induced failure is being assumed, this analysis only considers the combination with the larger coincidental flood (Alternative 2). Also, the seismically-induced failure mechanism is assumed to bound the sunny day failure mechanism, in terms of flood magnitude and warning time. Therefore, calculations are not performed for the sunny day failure mechanism.

a. <u>Basis of Inputs</u>

The rainfall, hydrologic, and hydraulic inputs are the same as previously described for flooding in stream and rivers. The National Atlas and the USACE National Inventory of Dams are used to identify the watershed dams and obtain general information for each dam (Exelon 2013i). More specific information is obtained from records provided by the Metropolitan Water Reclamation Districts of Greater Chicago, USACE Master Water Control Manuals, the Forest Preserve District of Will County, and the County of Du Page (Exelon 2013j).

For the assessment of watershed dams, the dams without more detailed information are assumed to have a linear elevation-storage relationship. Reservoirs are assumed at full flood stage pool at the top of dam at the beginning of the dam break.

For the dam failure analysis, hydrologic failures are assumed to occur at peak flood stage during the PMF. However, to maximize downstream flooding effects, dams are also set to fail based on a more critical timing than at peak flood stage. The more conservative of the two conditions is used in the analysis.

Lockport Lock and Dam and Brandon Road Lock and Dam navigational structures (concrete structures and lock gates) are considered as not failing (Empyrean 2013). However, failures are assumed for the channel embankment walls at Lockport Lock and Dam and the embankment section of Brandon Road Lock and Dam. The breach areas are assumed maximized based on the range of breach parameters provided by USACE guidance in RD-13, *Flood Emergency Plans, Guidelines for Corps Dams* (USACE 1980). However, because the breach areas are maximized, a mid range failure development time is assumed. Discharge curves for the dams modeled in detail are assumed reduced by 30 percent to account for debris blockage, gate failure, etc. For hydrologic dam failure, domino-type failure is assumed to maximize the effects at DNPS. For seismic dam failure, all dams are assumed to fail at the same time.

Hydrodynamic Model

The hydrodynamic two-dimensional model domain (AMEC 2013a and 2013b) is based on the same set of USACE bathymetry and USGS topography data used to develop the cross sections for the HEC-RAS model. In addition, LiDAR data is used for the area in the immediate vicinity of the site. USGS land cover data is used as a basis for developing appropriate roughness coefficients.

b. <u>Models Used</u>

Hydrologic Model

USACE HEC-HMS software is used to develop rainfall runoff transformation, reach routing for the watershed, and dam failure analysis (Exelon 2013i and 2013j).

Hydraulic Model

USACE HEC-RAS software is used to evaluate unsteady-state routing to perform dynamic channel routing of flood hydrographs to the site and dam failure analysis (Exelon 2013i and 2013j). While water surface profiles are computed in HEC-RAS, stillwater levels at the site are derived from two-dimensional hydrodynamic modeling. Outputs from HEC-RAS are used for the upstream and downstream boundary conditions in the two-dimensional model.

Hydrodynamic Model

RiverFLO-2D computer software developed by Hydronia, LLC is used for two-dimensional hydrodynamic modeling for river reaches just upstream and downstream of the site, with boundary conditions from the HEC-RAS model (AMEC 2013a).

FLO-2D computer software developed by FLO-2D Software, Inc., is used to establish final stillwater levels within the site, along with performing wind-generated wave calculations (discussed later), using results from the RiverFLO-2D model as boundary conditions (AMEC 2013b). RiverFLO-2D also produces stillwater levels within the site but FLO-2D is viewed as superseding the levels from RiverFLO-2D because the FLO-2D stillwater levels form the basis for the wind-generated wave calculations (discussed later).

c. Methodology

Dam failure is addressed in two distinct stages. The first stage is an overall assessment of all watershed dams to screen non-critical and potentially critical dams. Those dams that are not found to have a negligible effect are considered to be potentially critical dams. The second stage is an analysis that incorporates the potentially critical dams in the modeling as individual elements. The remaining non-critical dams are included in the model as lumped hypothetical dams, grouped based on their locations and general physical characteristics, such as dam type, dam height, and storage volume. This same methodology is also applied to seismically-induced dam failure.

Assessment of Watershed Dams (Exelon 2013i)

The National Atlas and the USACE National Inventory of Dams are used to identify the watershed dams. Research is also conducted to determine if there are any potentially significant proposed dams or modifications to existing dams. It is determined there are 137 dams located in the watershed. Of these, there are some database entries that represent the same reservoir, are actually located outside of the watershed, or do not have sufficient pertinent information associated with the entry.

For those entries that did not have sufficient information, by reference to aerial photography, it is determined three reservoirs are very small detention ponds in development areas removed from the near vicinity of the site and could be justifiably removed from additional consideration. In addition, there are two structures in the Chicago area that allow navigation and convey water from Lake Michigan into the Illinois River system. However, during more extreme rainfall events, high flow on the Illinois River system side exceeds the levels in Lake Michigan. Therefore, failure during a PMF type event would release peak flows into Lake Michigan. These structures are assumed not to fail in order to maximize downstream effects at the site.

Because the site is not a dry site under the current license basis, as a practical application, the plant grade elevation is used as an antecedent water surface elevation and a threshold of 0.5 feet is used to screen non-critical and potentially critical dams. The assessment process considers extremely conservative assumptions, and the established threshold does not represent an actual water surface elevation increase. The threshold is a measure of significance of the effect from an individual dam failure. The more detailed dam failure analysis defines the peak water surface elevation and incorporates all dams.

The Peak Outflow with Attenuation Method from the JLD-ISG-2013-01 guidance (NRC JLD-ISG-2013-01) is used to screen non-critical and potentially critical dams. The National Weather Service simplified dam break model regression equation is used to calculate peak outflow from each dam. Attenuation of the peak failure flow is calculated using the U.S. Bureau of Reclamation routing equation. Considering individual effects from dam failure, there are five dams that exceed the established threshold and are considered potentially critical.

To further screen the dams and to group non-critical dams into hypothetical dams to account for multiple dam failures, the Hydrologic Model Method from the JLD-ISG-2013-01 guidance (NRC JLD-ISG-2013-01) is utilized. Using the HEC-HMS model previously discussed for the evaluation of flooding in streams and rivers, the watershed dams are

grouped based on sub-basin location and included in the HEC-HMS model as hypothetical combined reservoirs. The combined reservoirs utilize the largest height from the group of dams, the total storage from the group of dams, and a combined breach width to represent breaches of the individual dams.

Individual dams are removed to assess the significance of the particular dam to the overall contribution of dam failure effects. Through this process it is determined that five dams are potentially critical and are to be included discretely in the dam failure analysis model. The remaining dams are non-critical and included in the dam failure analysis model as hypothetical combined reservoirs.

Dam Failure Analysis (Exelon 2013j)

Hydrologic failure and seismically-induced failure incorporate coincident rainfall with the failure mechanism. Sunny day failure is bounded by the analyses for hydrologic failure and seismically-induced failure, in terms of flood magnitude and warning time. As determined by the assessment of watershed dams, there are five potentially critical dams that are incorporated into the dam failure analysis based on specific characteristics for each dam. The remaining non-critical dams in the watershed are grouped as hypothetical combined reservoirs for failure analysis.

The five potentially critical dams are Busse Woods Dam, Lockport Lock and Dam, Brandon Road Lock and Dam, Sauk Trail Dam, and Fawell Dam. Busse Woods Dam and Lockport Lock and Dam are located on different tributaries of the Des Plaines River upstream of Brandon Road Lock and Dam. Sauk Trail Dam and Fawell Dam are also located on different tributaries of the Des Plaines River and enter the river downstream of Brandon Road Lock and Dam. Each of the dams, with the exception of Brandon Road Lock and Dam, is modeled using HEC-HMS. Brandon Road Lock and Dam is modeled using HEC-RAS.

In addition to the five potentially critical dams, the Dresden Cooling Lake Dam could also be potentially critical. The Dresden Cooling Lake Dam is located immediately adjacent to the Kankakee River just upstream of the site. Failure effects of the dam are dependent on the tailwater in the river. Therefore, the Dresden Cooling Lake Dam is evaluated individually.

The Busse Woods Dam characteristics are based on information obtained from the Metropolitan Water Reclamation Districts of Greater Chicago. The Lockport Lock and Dam and the Brandon Road Lock and Dam characteristics are based on information obtained from their respective Master Water Control Manuals. Sauk Trail Dam characteristics are based on information obtained from the Forest Preserve District of Will County. Fawell Dam characteristics are based on information obtained from the County of Du Page.

Busse Woods Dam (Exelon 2013j)

The controlling PMF snowmelt scenario HEC-HMS model described in the section for flooding in streams and rivers is used as a basis for the dam failure analysis. The model is modified to include a discrete sub-basin for the Busse Woods Dam watershed. As previously described, the sub-basin is analyzed for calibration and validation.

The main dam isn't overtopped during the controlling PMF snowmelt scenario. However, failure scenarios including failure of the principal spillway and failure of the emergency spillway are evaluated. Breach characteristics are based on maximized USACE guidance (USACE 1980). The entire principal spillway section is considered for the breach area with a development time of 0.1 hours. The emergency spillway breach is based on embankment failure with a breach width of three times the dam height, 1:1 side slopes, and a development time of 2 hours. In both cases, failure is assumed to occur at the peak elevation. Tailwater is accounted for by development of a rating curve of the downstream reach.

Failure of the principal spillway provides the more conservative scenario. The failure flow hydrograph is added to the HEC-HMS model directly to the nearest downstream element. This approach ignores routing and attenuation through a portion of the parent sub-basin.

Lockport Lock and Dam (Exelon 2013j)

The HEC-HMS model is modified further to include the appropriate attributes for the Lockport Lock and Dam. The failure scenario evaluated includes failure of the channel walls. Breach characteristics are based on maximized USACE guidance (USACE 1980). The channel wall breach is based on embankment failure with a breach width of three times the wall height, 1:1 side slopes, and a development time of 2 hours. Failure of the channel wall is assumed to occur at the peak elevation. Tailwater is accounted for based on the HEC-RAS model routing. The failure flow hydrograph is incorporated into the HEC-RAS model at the appropriate location. The main navigational (concrete and lock gate) structures are assumed not to fail. See *Non-Failure Justification for Concrete Structures at the Lockport and Brandon Road Navigation Projects During Extreme Flood Events* (Empyrean 2013) for further justification.

Brandon Road Lock and Dam (Exelon 2013j)

Brandon Road Lock and Dam is analyzed for failure using the HEC-RAS model described in the section for flooding in streams and rivers. The HEC-RAS model includes the Brandon Road Lock and Dam as an inline structure. Considering the upstream failures of Busse Woods and Lockport Lock and Dam, Brandon Road Lock and Dam is assumed to fail at the peak elevation. Tailwater is accounted for in the HEC-RAS model with unsteady state routing. The main navigational (concrete and lock gate) structures are assumed not to fail. See *Non-Failure Justification for Concrete Structures at the Lockport and Brandon Road Navigation Projects During Extreme Flood Events* (Empyrean 2013) for further justification.

The maximized outflow hydrographs from the upstream dam failures do not arrive at Brandon Road Lock and Dam at the same time. However, the Busse Woods failure time is adjusted for the peak to align with the peak from the Lockport Lock and Dam. Therefore, the combined effects of upstream dam failure are maximized at Brandon Road Lock and Dam.

The failure scenario evaluated includes failure of the embankment section. Breach characteristics are based on maximized USACE guidance (USACE 1980). The embankment section is based on embankment failure with a breach width of three times

the embankment height, 1:1 side slopes, and a development time of 2 hours. Failure is assumed to occur at the peak elevation.

As a sensitivity analysis, the timing at which failure occurs relative to the downstream flooding is evaluated to maximize the flooding effects at the site. It is determined by adjusting the failure time to occur after the peak inflow, downstream flooding effects at the site are maximized. To maximize flooding conditions, the Lockport Lock and Dam and Busse Woods failure times are adjusted to maximize the flooding effects at Brandon Road Lock and Dam and subsequently at the site. The Brandon Road Lock and Dam failure is triggered at the maximum pool elevation, which corresponds to the time-adjusted and coincident peak flows resulting from the Lockport Lock and Dam and Busse Woods Dam failures.

Fawell Dam (Exelon 2013j)

The HEC-HMS model is modified further to include the appropriate attributes for Fawell Dam. An existing sub-basin of the HEC-HMS model that is slightly larger than the Fawell Dam sub-basin is used to define the watershed. Breach characteristics are based on maximized USACE guidance (USACE 1980). Dam breach is based on embankment failure with a breach width of three times the dam height, 1:1 side slopes, and a development time of 2 hours.

Breach initiation is evaluated assuming failure occurs at the peak elevation and a specific trigger time. Tailwater is determined based on existing flood profile information at the dam. Setting the trigger time to maximize coincident failure effects at the site produces a more conservative result. The failure flow hydrograph is incorporated into the HEC-HMS model at the appropriate location

Sauk Trail Dam (Exelon 2013j)

The HEC-HMS model is modified further to include the appropriate attributes for Sauk Trail Dam and an associated watershed. The watershed is assigned a rainfall hyetograph from the parent sub-basin and all rainfall is converted to runoff without losses or attenuation. Breach characteristics are based on maximized USACE guidance (USACE 1980). The PMF does not overtop the embankment section of the main dam. However, the emergency spillway embankment section does overtop and is assumed to fail. A breach width of three times the dam height, 1:1 side slopes, and a development time of 2 hours are assumed.

Breach initiation is evaluated assuming failure occurs at the peak elevation and a specific trigger time. Tailwater is set to the low chord elevation of a downstream railroad crossing. Setting the trigger time to maximize coincident failure effects at the site produces a more conservative result. The failure flow hydrograph is incorporated into the HEC-HMS model at the appropriate location.

Dresden Cooling Lake Dam (Exelon 2013j)

The Dresden Cooling Lake Dam characteristics are based on information obtained from the SEP and the USACE National Inventory of Dams. A separate HEC-HMS model including only the dam and its watershed is developed for analysis. The rainfall hyetograph for the site is applied to the watershed and all rainfall is converted to runoff

without losses or attenuation. Under these conservative conditions, the dam is subject to overtopping just above the crest elevation. Tailwater conditions are based on the appropriate flow hydrograph from the HEC-RAS model.

The dam is an earthen embankment. Breach initiation is evaluated assuming failure occurs at the peak elevation and a specific trigger time. A breach width of three times the dam height, 1:1 side slopes, and a development time of 2 hours are assumed. Because of the tailwater conditions based on adjacent flooding of the Kankakee River, the dam has no effect on the water surface elevation at the site.

All Other Non-Critical Dams (Exelon 2013j)

As previously identified, the non-critical dams are included in the HEC-HMS model as hypothetical clusters of dams. The individual dams are spread out over the watershed sub-basins. However, a cluster is assumed to discharge directly to the respective sub-basin outlet. The clusters represent the total volume of each dam. The height of a hypothetical dam is a storage weighted average.

Embankment dams and concrete dams are modeled separately in order to accommodate the different breach parameters specific to the dam type. A hypothetical embankment dam cluster breach parameters assume a breach width equivalent to the sum of one half the height of each individual dam included in the cluster, no side slopes, and a development time of 4 hours. A hypothetical concrete dam cluster breach parameters assume a breach section equivalent to the total length of each dam included in the cluster and using a 5-minute development time.

Hydrodynamic Model (AMEC 2013a and 2013b)

Because the resulting elevations of the HEC-RAS model exceed plant grade and to better evaluate the complex topography of the site vicinity, including the downstream dam and the upstream confluence of tributaries, two-dimensional unsteady state modeling is also performed to assess the site. RiverFLO-2D computer software developed by Hydronia, LLC is used for two-dimensional modeling.

The model domain extends from just upstream of the Des Plaines and Kankakee rivers confluence to Morris, Illinois. The site structures are incorporated as a physical boundary in the model domain. The Dresden Island Lock and Dam is incorporated as an internal rating curve based on the rating curve in the HEC-RAS model. The downstream boundary condition is incorporated upstream of the Route 47 bridge at Morris, Illinois based on the rating curve for the corresponding cross section from the HEC-RAS model. The Manning's roughness coefficients are based on the Cowan method and account for only the characteristics of roughness coefficients that are applicable in two-dimensional modeling.

The RiverFLO-2D model is evaluated using the inflow hydrographs from the HEC-RAS model for Alternative 2. Inflow hydrographs for the Des Plaines and Kankakee rivers are applied individually upstream of the confluence.

FLO-2D computer software developed by FLO-2D Software, Inc., is used to establish final stillwater levels within the site, along with performing wind-generated wave calculations (discussed later), using results from the RiverFLO-2D model as boundary

conditions (AMEC 2013b). RiverFLO-2D also produces stillwater levels within the site but FLO-2D is viewed as superseding the levels from RiverFLO-2D because the FLO-2D stillwater levels form the basis for the wind-generated wave calculations (discussed later).

d. Results

Hydraulic Model (Exelon 2013j)

The HEC-HMS model is modified to incorporate all dam failures as discussed in the controlling PMF snowmelt scenario. The combined and routed PMF and dam failure flow hydrographs are entered into the HEC-RAS model previously discussed. The HEC-RAS model is evaluated using unsteady-state flow. Cross Section 135306 represents the upstream end of the plant and Cross Section 134410 represents the downstream end of the plant. For the Alternative 2 PMF scenario with dam failure, the peak flow rate at Cross Section 135306 is approximately 438,000 cfs.

Hydrodynamic Model (AMEC 2013a and 2013b)

Two-dimensional, unsteady state modeling is also performed to assess the site using the RiverFLO-2D and FLO-2D models previously described. The RiverFLO-2D model is evaluated using the inflow hydrograph from the HEC-RAS model for the Alternative 2 PMF with dam failure. FLO-2D is used to establish final stillwater levels within the site, along with performing wind-generated wave calculations (discussed later), using results from the RiverFLO-2D model as boundary conditions. The maximum water surface elevation results are provided in Table 2.

Table 2. Summary of Maximum Stillwater Elevations

Structure	Maximum Stillwater Elevation (feet NGVD 29)
Reactor Building, Unit 1	524.88
Reactor Building, Unit 2	524.76
Reactor Building, Unit 3	524.61
Turbine Building, Unit 1	524.22
Turbine Building, Unit 2	524.61
Turbine Building, Unit 3	524.35
ISFSI-East	524.85
ISFSI-West	524.27
HPCI & Diesel Generator	524.60

Note: ISFSI – Independent Spent Fuel Storage Installation
HPCI – High Pressure Coolant Injection

Seismically-Induced Dam Failure (Exelon 2013o)

Because seismically-induced dam failure is being assumed, the more conservative coincident flooding associated with the OBE is selected to evaluate the seismic dam failure combination. It is determined that the one-half PMP is the lesser rainfall event associated with the OBE and is used for the coincident flooding. The dam failure analysis and HEC-HMS model previously described is evaluated using the one-half PMP and incorporating all dam failures.

The dam failure conditions and parameters previously described for hydrologic dam failure are also applied to the seismically-induced dam failure scenario. However, failure times of the hypothetical dams are reduced to 0.5 hours. In addition, the Lockport Lock and Dam and Brandon Road Lock and Dam concrete structures are assumed to fail. All dams are assumed to initiate failure at the same time. Sensitivity is evaluated to determine the timing that produces the most critical results at the DNPS site.

The combined and routed flood flow and dam failure flow hydrographs are entered into the HEC-RAS unsteady flow model previously discussed. Cross Section 135306 represents the upstream end of the plant and Cross Section 134410 represents the downstream end of the plant. For the Alternative 2 seismic dam failure scenario, the peak flow rate at Cross Section 135306 is approximately 173,000 cfs. Because the seismic dam failure results are less than the hydrologic dam failure result, a hydrodynamic analysis of the seismic dam failure scenario is not evaluated.

Warning Times

The controlling PMF snowmelt scenario with hydrologic dam failure exhibits the fastest flood response time to reach plant grade elevation of 517 feet NGVD 29 (Exelon 2013j). From the onset of the storm event, the plant grade elevation is exceeded within 2 days 10 hours and 40 minutes. The intermediate elevation of 509 feet NGVD 29 is exceeded within 1 day 12 hours and 45 minutes. Therefore, the timing between the two elevations is 21 hours and 55 minutes.

The fastest response time of 24 hours to the intermediate elevation is a result of an all-season PMP scenario without snowmelt. The scenario is preceded by a 72-hour antecedent storm and a 72-hour dry period. By removal of the antecedent storm, the intermediate elevation is reached within 14 hours and 20 minutes from the onset of the storm event. However, by removal of the antecedent storm, plant grade elevation is not exceeded.

The seismic dam failure scenario does not exceed plant grade elevation (Exelon 2013o). The intermediate elevation of 509 feet NGVD 29 is exceeded within 3 days 8 hours and 35 minutes. However, as previously described the seismic dam failure scenario is evaluated coincident with a one-half PMP rainfall event. Sunny day failure is not specifically evaluated. Sunny day failures would result in less flooding compared to the seismic failure scenario. Warning times for sunny day failures would be comparable to the seismic failures.

Downstream Dam Failure

The downstream Dresden Lock and Dam impounds the ultimate heat sink for DNPS. The UFSAR provides assessment of low water conditions by assuming failure of the lock and dam. There is enough water impounded by the intake and discharge canals below their high point elevations to allow a safe shutdown of Dresden Units 2 and 3 (Dresden UFSAR).

3. **Storm Surge –** DNPS has an inland location and does not connect directly with any of the water bodies considered for meteorological events associated with surge. Flooding due to a storm surge is not plausible at DNPS.

4. **Seiche –** DNPS has an inland location and does not connect directly with any of the water bodies considered for meteorological events associated with a seiche. Flooding due to a seiche is not plausible at DNPS.

5. **Tsunami –** DNPS has an inland location and does not connect directly with any of the water bodies considered for tsunami events. Flooding due to a tsunami is not plausible at DNPS.

6. **Ice Induced Flooding –** 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 a 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 a dam downstream of a site may impound water upstream of itself, thus causing a flood via backwater effects.

   a. **Basis of Inputs**

   The USACE National Ice Jam Database is used to determine the most severe historical ice flooding events (Exelon 2013k). The peak elevation of historic ice jam flooding is assumed to represent the full height of an ice jam relative to the normal water surface elevation at the recorded location, maximizing the height of the ice jam. No tailwater effects are assumed.

   b. **Models Used**

   Software models are not used to evaluate ice induced flooding. Comparative analysis with other flood causal mechanisms is used to bound ice induced flooding.

   c. **Methodology**

   Ice induced flooding is assessed by reviewing the USACE National Ice Jam Database to determine the most severe historical events in the DNPS watershed (Exelon 2013k). The historic records are used to develop a conservative estimate for upstream ice jam collapse flooding and downstream ice jam backwater effects. By comparison to the PMF and dam failure results it is demonstrated that the ice-induced flooding effects are bounded by other flood causing mechanisms. It should be noted that NUREG/CR-7046 does not identify any appropriate combination of events coincident with ice jam flooding.

   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 assumed to represent the ice jam. The ice jam is transposed to critical upstream or downstream river crossings without adjustment.

For upstream ice jam failure analysis, the National Weather Service simplified dam failure equation is used to determine a peak outflow (Exelon 2013k). The ice jam is considered a concrete type dam for the purposes of estimating breach parameters (USACE 1980). The breach width is considered to be multiple monoliths equivalent to the distance between river crossing piers. The breach development time is 0.1 hours. The peak outflow is transposed to the site without attenuation. A corresponding water surface elevation is then determined based on the Cross Section 135306 rating curve from the hydraulic model previously discussed.

d. **Results**

The maximum upstream ice jam is determined to be 17.44 feet, based on a historic maximum occurring on a tributary of the Iroquois River, which is a tributary of the Kankakee River. The maximum ice jam is transposed and assumed to occur at the critical upstream river crossing on the Des Plaines River. Assuming collapse of the full ice jam height releases a failure wave downstream without any attenuation results in a water surface elevation at the site of 515.77 feet NGVD 29.

The maximum downstream ice jam is determined to be 31.87 feet, based on a historic maximum occurring on the Fox River, a tributary of the Illinois River. The maximum downstream ice jam is transposed to the nearest downstream river crossing. The nearest downstream river crossing is located downstream of Dresden Island Lock and Dam on the lower elevation pool of the Marseilles Lock and Dam. Assuming the maximum ice jam occurs at the nearest downstream river crossing results in a water surface elevation of 514.87 feet NGVD 29.

7. **Channel Migration or Diversion –** The authority to control the Dresden Island Lock and Dam, and therefore, the level of the pool impounded by it, is vested in the USACE (Dresden UFSAR). Should the dam become damaged, one method of maintaining the pool level would be through the use of increased water diversion from Lake Michigan. This would be accomplished with the permission of the U.S. Supreme Court.

8. **Combined Effect Flood (including Wind-Generated Waves) –** The criteria for combined events are provided in NUREG/CR-7046. The combined events incorporate the flood causal mechanisms previously discussed for precipitation events and hydrologic or seismic dam failures. Each combined event also incorporates waves induced by 2-year wind speed applied along the critical direction.

a. **Basis of Inputs**

Based on the resulting water surface elevations previously discussed, USGS topography is used to develop the fetch length (Exelon 2013n). The 2-year wind speed is determined using the ANSI/ANS-2.8-1992 guidance.

Wind wave characteristics are applied to a hydrodynamic model using the resulting water surface elevations from the RiverFLO-2D model previously discussed. The hydrodynamic model domain (AMEC 2013b) is based on LiDAR data for the area in the immediate vicinity of the site and supplemented with survey data. USGS land cover imagery is used as a basis for developing appropriate roughness coefficients.

b. Models Used

Hand calculations are used to develop wind-generated wave characteristics (Exelon 2013n). FLO-2D computer software developed by FLO-2D Software, Inc. is used for modeling wave propagation through the site (AMEC 2013b). Hand calculations are used to determine wave runup (AMEC 2013b).

c. Methodology

Coincident wind wave characteristics are determined using the methodology outlined in the USACE Coastal Engineering Manual (Exelon 2013n). The 2-year wind speed is determined using the ANSI/ANS-2.8-1992 guidance. Based on the maximum water surface elevation from the flooding analysis, the longest critical straight line fetch is determined. The 2-year wind speed is applied to the straight line fetch to develop the wind wave characteristics. The wind wave characteristics, including wind setup, are applied to a two-dimensional hydrodynamic model to assess the propagation of waves through the site. Wind setup is determined using USACE guidance outlined in Engineering Manual EM 1110-2-1420 (Exelon 2013n). Wind setup is the effect of the horizontal stress of the wind on the water, driving it in the direction of the wind.

FLO-2D computer software developed by FLO-2D Software, Inc. is used for wave propagation two-dimensional modeling (AMEC 2013b). The model domain includes the immediate vicinity of the site area. The site structures are incorporated in the model domain as elevations greater than the maximum water surface elevation. The Manning's roughness coefficients are based on the recommended ranges in the FLO-2D reference manual.

The maximum stillwater elevations determined using the RiverFLO-2D model and wind setup are applied to the FLO-2D model. The propagation of waves through the site is modeled by applying the maximum wave height to the south and south east boundaries of the FLO-2D model domain. FLO-2D software solves the continuity and dynamic wave equation to route flow based on the site topography and Manning's roughness coefficients.

Runup, including wave setup, on a vertical surface is determined using the USACE guidance outlined in Miscellaneous Paper CERC-90-4 (AMEC 2013b). Runup is the maximum elevation of wave uprush above stillwater level. Wave setup is the superelevation of mean water level caused by wave action.

d. Results

Wind setup is determined to be 0.30 feet (Exelon 2013n). The propagation of waves through the model results in the maximum runup elevations provided in Table 3 (AMEC 2013b).

Table 3. Summary of Maximum Wave Runup Elevation

Structure	Maximum Runup Elevation (feet NGVD 29)
Reactor Building, Unit 1	526.48
Reactor Building, Unit 2	528.86
Reactor Building, Unit 3	527.16
Turbine Building, Unit 1	524.77
Turbine Building, Unit 2	529.01
Turbine Building, Unit 3	525.75
ISFSI-East	527.35
ISFSI-West	527.87
HPCI & Diesel Generator	528.75

Note: ISFSI – Independent Spent Fuel Storage Installation
HPCI – High Pressure Coolant Injection

9. Hydrodynamic and Debris Load

a. Basis of Inputs

The RiverFLO-2D hydrodynamic model domain and inputs are identical to those previously discussed for dam breaches and failures (AMEC 2013a).

The FLO-2D hydrodynamic model domain and inputs are identical to those previously discussed for the combined effect flood (AMEC 2013b).

b. Models Used

RiverFLO-2D computer software developed by Hydronia, LLC is used for two-dimensional hydrodynamic modeling (AMEC 2013a).

FLO-2D computer software developed by FLO-2D Software, Inc. is used for two-dimensional hydrodynamic modeling (AMEC 2013b).

c. Methodology

The hydrodynamic analyses previously discussed for dam breaches and failures (AMEC 2013a) and the wind wave analysis (AMEC 2013b) are used to determine hydrodynamic results.

d. Results

The RiverFLO-2D model results in velocity vectors at DNPS safety-related structures coincident with the maximum water surface elevation that are less than 5 feet per second (AMEC 2013a). The flow velocities at DNPS are much less than those in the Illinois River.

The FLO-2D model results in a maximum velocities and maximum hydrodynamic pressure are provided in Table 4 (AMEC 2013b).

Table 4. Summary of Hydrodynamic Results

Structure	Maximum Velocity (fps)	Maximum Hydrodynamic Pressure (psf)
Reactor Building, Unit 1	5.73	81.53
Reactor Building, Unit 2	5.37	71.61
Reactor Building, Unit 3	3.87	37.19
Turbine Building, Unit 1	6.87	117.20
Turbine Building, Unit 2	6.42	102.35
Turbine Building, Unit 3	6.14	93.62
ISFSI-East	2.90	20.88
ISFSI-West	4.52	50.73
HPCI & Diesel Generator	3.87	37.19

Note: ISFSI – Independent Spent Fuel Storage Installation
HPCI – High Pressure Coolant Injection

4. COMPARISON WITH CURRENT DESIGN BASIS

The reevaluation maximum stillwater elevation does not exceed the current design basis. The current flood mitigation strategy remains valid. The current license basis indicates when water level rises above 517 feet NGVD 29, the doors to the reactor building will be opened and water will be allowed to flood the plant. Per DNPS flooding procedure DOA 0010-04 (Exelon DOA 0010-04), the flood pump will be placed on a floating platform that will rise and fall with the flood level. The flood pump will operate at an elevation above the reevaluation maximum stillwater elevation. The strategy includes provision for safe shutdown when water levels reach elevation 509 feet NGVD 29.

The reevaluation results differ from the current design basis. Table 5 provides a summary of input parameters from the design basis and the hazard reevaluation (Exelon 2012). Limited details are available for the current design basis analysis. Common data points for the PMP indicate the estimates are close. However, the design basis details for storm placement and distribution throughout the watershed are not available.

The current design basis developed a hydrologic model using 13 sub-basins, while the reevaluation developed a hydrologic model with 23 sub-basins. The total area of the watershed listed for the current design basis varies from the reevaluation total area. It is not clear if the current design basis is rounding a result that more closely matches the reevaluation total area or incorporates additional area below the confluence in reporting the approximated total area. The reevaluation total area is based on current mapping and incorporates the contributing area between the site and the confluence. The reevaluation resulting flow hydrographs produce a higher combined peak flow for the Des Plaines River and associated tributaries, but a lower combined peak flow for the Kankakee River and associated tributaries. In addition, the reevaluation demonstrates that the combined flow response is spread out more over time when compared to the resulting flow hydrographs for the current design basis.

Table 5 - Summary of Existing and Reevaluated Parameters

Parameter	Design Basis (FRC 1982) Value/Incorporated	Hazard Reevaluation Value/Incorporated
Probable Maximum Precipitation		
Methodology	EM 1110-2-1411	HMR 51 and HMR 52
Storm Duration	72 hours	72 hours
200 sq. mi., 24-hour index PMP	23.25 in.	24.1 in.
Probable Maximum Flood		
Nonlinear	No	Yes
Hydrologic Model		
Model	HEC-1	HEC-HMS
Total area	~7300 sq. mi.	~7250 sq. mi.
Subbasins	13	23
Kankakee area	~5895 sq. mi.	5148 sq. mi.
Percent Impervious	<1%	1.84% - 6.16%
Des Plaines area	~1370 sq. mi.	2105 sq. mi.
Percent Impervious	18%	22.18% - 63.33%
Calibration Events	July 1957	Multiple events 1996 to 2009
Hydraulic Model		
Methodology	Standard Step (Steady State)	Unsteady State
Model	HEC-2	HEC-RAS
Model Coverage	~1.4 mi.	~47.7 mi.
Cross Sections	7	190
Cross Sections Source	USACE river soundings and USGS topo	USACE bathymetry and USGS topo
Coefficient of Contraction	0.3	Unsteady State Momentum Equation
Coefficient of Expansion	0.5	Unsteady State Momentum Equation
Manning's n – channel	0.03	0.035
Manning's n - overbanks	0.05	0.035 to 0.1
Dresden Dam Weir Coefficients	2.70 to 3.09	3.2 to 3.8
Calibration Events	1947 and 1957	September 2008
Wind Wave Activity		
Methodology	USACE Shore Protection Manual	USACE Coastal Engineering Manual & Hydrodynamic
Wave Height	2.6 ft.	Hydrodynamic Modeling
Runup	3 ft.	Hydrodynamic Modeling
Combined Events		
Baseflow	No	Yes
Antecedent Sequential Storms	No	Yes
Snowmelt	No	Yes
Dam Failure	No	Yes

The hydraulic model developed for the current design basis is a steady-state model incorporating a limited area in the vicinity of the site. The reevaluation incorporates a much greater area upstream and downstream and utilizes an unsteady-state hydraulic model. The channel cross section determined for the reevaluation is determined to be a smaller area. In addition, the channel roughness coefficient is determined to be higher. The overbank roughness coefficients are applied as a range of values that bracket the overbank roughness coefficient used in the current design basis. The reevaluation analysis also accounts for nonlinear basin response, incorporates dam failures in the PMF analysis, and evaluates the combined effects at the site using hydrodynamic modeling. The comparison of existing and reevaluated flood hazard is provided in Table 6.

Table 6 - Comparison of Existing and Reevaluated Flood Hazard at DNPS

Flood Causing Mechanism	**Design Basis**	**Comparison**	**Flood Hazard Reevaluation Elevation**	
Flooding in Streams and Rivers	PMF Elevation – 525 feet msl Corresponding Flow – 490,000 cfs No cool-season PMP was evaluated (UFSAR Section 2.4.3)	Bounded	All-Season PMF Flow – 318,000 cfs Cool-Season PMF Flow – 380,000 cfs (Refer to Dam Breaches and Failures for reevaluated stillwater elevations)	
Dam Breaches and Failures	The UFSAR evaluates only failure of Dresden Cooling Lake Dam under normal river conditions. Resulting elevation is 508.2 feet msl. (UFSAR Section 2.4.8.2)	Not Bounded (Hydrodynamic results for dam failure is bounded by Flooding in Streams and Rivers)	Cool-Season PMF with Dam Failure Flow – 438,000 cfs Cool-Season PMF with Dam Failure Elevations (feet NGVD 29)	
			Reactor Building, Unit 1	524.88
			Reactor Building, Unit 2	524.76
			Reactor Building, Unit 3	524.61
			Turbine Building, Unit 1	524.22
			Turbine Building, Unit 2	524.61
			Turbine Building, Unit 3	524.35
			ISFSI-East	524.85
			ISFSI-West	524.27
			HPCI & Diesel Generator	524.60
			Seismic Dam Failure Flow – 173,000 cfs	
Storm Surge	Not Plausible	Not Plausible	Not Plausible	
Seiche	Not Plausible	Not Plausible	Not Plausible	
Tsunami	Not Plausible	Not Plausible	Not Plausible	

<table>
<tr><th>Flood Causing Mechanism</th><th>Design Basis</th><th>Comparison</th><th colspan="2">Flood Hazard Reevaluation Elevation</th></tr>
<tr><td>Ice Induced Flooding</td><td>This flood causing mechanism is not described in the UFSAR</td><td>Criteria Not Described in the UFSAR</td><td colspan="2">Upstream Ice Jam Corresponding Elevation at DNPS – 515.77 feet NGVD 29

Downstream Ice Jam Corresponding Elevation at DNPS – 514.87 feet NGVD 29

Ice Induced Flood is bounded by the PMF</td></tr>
<tr><td>Channel Migration or Diversion</td><td>As indicated in the UFSAR, the authority to control the river is vested in the USACE. Should the dam become damaged, one method of maintaining the pool level would be through the use of increased water diversion from Lake Michigan. This would be accomplished with the permission of the U.S. Supreme Court.

(UFSAR Section 2.4.9)</td><td>Bounded</td><td colspan="2">Same procedures as discussed in the UFSAR.</td></tr>
<tr><td rowspan="2">Combined Effect Flood (including wind-generated waves and upstream dam failure)</td><td rowspan="2">PMF + Wind Wave Runup = 525 feet + 3 feet = 528 feet msl

(UFSAR Section 2.4.3)</td><td rowspan="2">Not Bounded</td><td colspan="2">Cool-Season PMF with Dam Failure Elevation + Wave Runup and Setup + Wind Setup (feet NGVD 29)</td></tr>
<tr><td>Reactor Building, Unit 1
Reactor Building, Unit 2
Reactor Building, Unit 3
Turbine Building, Unit 1
Turbine Building, Unit 2
Turbine Building, Unit 3
ISFSI-East
ISFSI-West
HPCI & Diesel Generator</td><td>526.48
528.86
527.16
524.77
529.01
525.75
527.35
527.87
528.75</td></tr>
</table>

Flood Causing Mechanism	Design Basis	Comparison	Flood Hazard Reevaluation Elevation	
Hydrodynamic and Debris Loads	Criteria Not Described in the UFSAR	Criteria Not Described in the UFSAR	Maximum flow velocities (feet per second)	
			Reactor Building, Unit 1	5.73
			Reactor Building, Unit 2	5.37
			Reactor Building, Unit 3	3.87
			Turbine Building, Unit 1	6.87
			Turbine Building, Unit 2	6.42
			Turbine Building, Unit 3	6.14
			ISFSI-East	2.90
			ISFSI-West	4.52
			HPCI & Diesel Generator	3.87
			Maximum hydrodynamic pressure (pounds per square foot)	
			Reactor Building, Unit 1	81.53
			Reactor Building, Unit 2	71.61
			Reactor Building, Unit 3	37.19
			Turbine Building, Unit 1	117.20
			Turbine Building, Unit 2	102.35
			Turbine Building, Unit 3	93.62
			ISFSI-East	20.88
			ISFSI-West	50.73
			HPCI & Diesel Generator	37.19

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Enclosure 3

CD-R labeled:

Dresden Nuclear Power Station
Pertinent Site Data

Enclosure 4

SUMMARY OF REGULATORY COMMITMENTS

The following table identifies commitments made in this document. (Any other actions discussed in the submittal represent intended or planned actions. They are described to the NRC for the NRC's information and are not regulatory commitments.)

COMMITMENT	COMMITTED DATE OR "OUTAGE"	COMMITMENT TYPE	
		ONE-TIME ACTION (Yes/No)	PROGRAMMATIC (Yes/No)
The Dresden Nuclear Power Station current design basis flood does not bound the reevaluated hazard for all flood causing mechanisms. Specifically, combined-effects (combination of river flooding from precipitation, upstream dam failure, and wind-generated waves), hydrodynamic/debris loads, and local intense precipitation flooding were not bounded by the current design basis flood hazard. Therefore, Dresden Nuclear Power Station plans to prepare a full Integrated Assessment Report (Scenario 4).	May 10, 2015	Yes	No