Enclosure 1

Byron Nuclear Generating Station
Flood Hazard Reevaluation Report
Revision 0

(54 pages)

FLOOD HAZARD REEVALUATION REPORT

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

for the

Byron Nuclear Generating Station

4450 North German Church Road
Byron, Illinois

Exelon

Exelon Generation Co., LLC
300 Exelon Way
Kennett Square, PA 19348

Prepared by:

ENERCON
Excellence—Every project. Every day

Enercon Services, Inc.
6525 North Meridian, Suite 400
Oklahoma City, OK 73116

Revision 0
Submitted Date: February 25, 2014

	Printed Name	Affiliation	Signature	Date
Preparer:	Anubhav Gaur	ENERCON		02/25/2014
Verifier:	Freddy Dahmash	ENERCON		02/25/2014
Approver:	Pat Brunette	ENERCON		2/25/2014
Lead Responsible Engineer:	Jeff Sanchez	Exelon	Jeff Sanchez	2/25/14
Branch Manager	Jon Cunzeman	Exelon		2/26/14
Senior Manager Design Engineering	Edward Blondin	Exelon	Edward Blondin	2/26/14
Exelon Corporate	Joe Bellini	Exelon		2/26/14

Contents

List of Tables

List of Figures

Acronyms and Abbreviations

ANS	American Nuclear Society
ANSI	American National Standards Institute
APM	available physical margin
CEM	Coastal Engineering Manual
cfs	cubic feet per second
CLB	Current Licensing Basis
DEM	Digital Elevation Model
FFT	Fast Fourier Transform
EM	Engineer Manual
ESP	Early Site Permit
ESRI	Environmental Systems Research Institute
ft	foot, feet
GIS	Geographic Information System
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HHA	Hierarchical hazard assessment
HMR	Hydrometeorological Report
hr	hour(s)
in	inch (inches)
LiDAR	Light Detection and Ranging
LIP	Local Intense Precipitation
mi	mile(s)
min	minute(s)
MSL	mean sea level
NAVD	North American Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NRC	U.S. Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
NTTF	Near-Term Task Force
NWS	National Weather Service
PMF	probable maximum flood
PMP	probable maximum precipitation
PMS	probable maximum seiche
PMSS	probable maximum storm surge
PMWS	probable maximum wind storm
sq.mi.	square mile(s)
SPF	standard project flood
SSCs	structures, systems, and components
UFSAR	Updated Final Safety Analysis Report
UHS	Ultimate Heat Sink
USACE	Unites States Army Corps of Engineers
USGS	Unites States Geological Survey

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) (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 Co. (Exelon) for the Byron Station (Byron), 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.

1.2 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 for Byron, including the effects from local intense precipitation (LIP) on the site, probable maximum flood (PMF) on streams and rivers, storm surges, seiches, tsunami, and dam failures. It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for 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 theNRC 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.*

1.3 Requested Information

Per Enclosure 2 of NRC issued information request 50.54(f) letter, the Report should provide documented results, as well as pertinent 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

2.1 Detailed Site Information

The Byron Station is located 3 miles southwest of Byron in Ogle County, in north central Illinois.

Byron is 2 miles east of the Rock River at about river mile 115 from the confluence with the Mississippi River. The Rock River rises in Fond du Lac County in southeastern Wisconsin and flows in a southerly direction into Illinois. The drainage area upstream of the site is about 8170 mi^2.

The plant site occupies about 1,300 acres and includes a portion of Woodland Creek which is an intermittent stream and a 3-mile long tributary to the Rock River.

The elevations of the Rock River at the site corresponding to the mean annual flow and the PMF are 672.0 feet USGS 1929 and 708.3 feet USGS 1929, respectively. The plant grade elevation is 869.0 feet USGS 1929. The grade floors of the safety-related buildings are at elevation 870.0 feet USGS 1929.

The river screen house, which withdraws makeup water from the Rock River, is protected from the combined event flood with coincident wind wave activity (see UFSAR Subsection 2.4.3.9). The safety-related equipment at the river screen house is at elevation 702.0 feet USGS 1929.

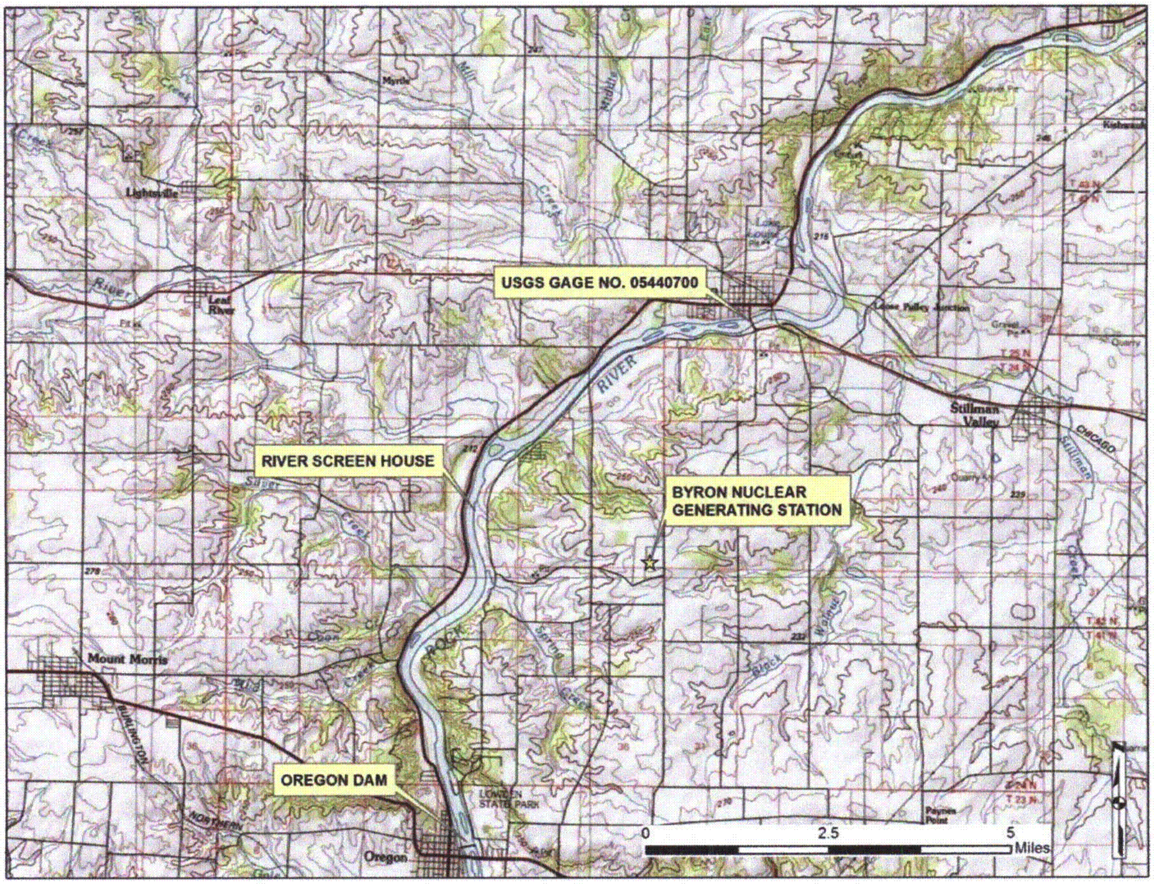

Figure 2.1.1 – Present-Day General Site Map

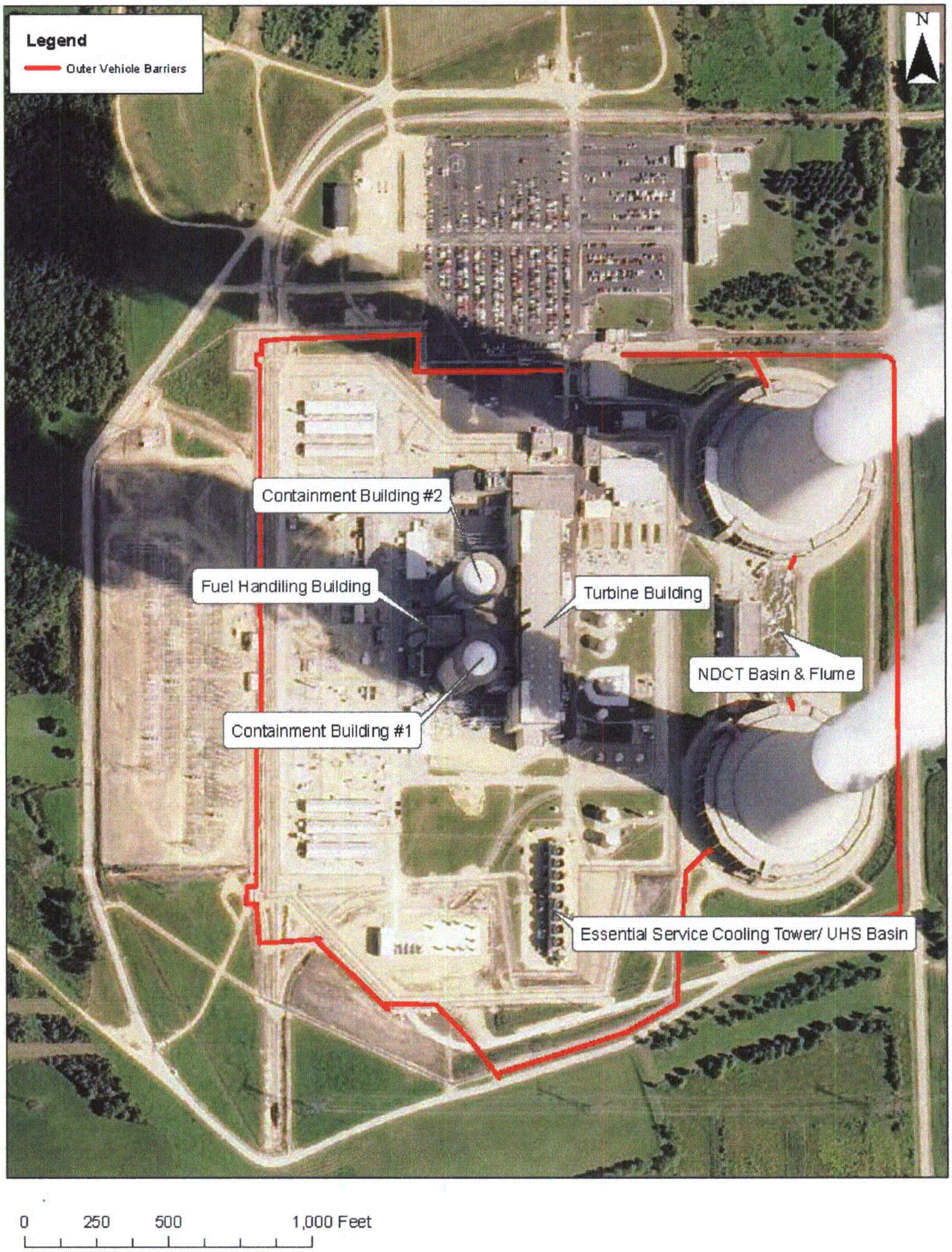

Figure 2.1.2 – Present-Day Detailed Site Layout

2.2 Current Licensing Basis

The following is a list of flood causing mechanisms and their associated water surface elevations that are considered in the Byron current licensing basis (CLB).

2.2.1 Effects of Local Intense Precipitation

For the analysis of local intense precipitation, the 1-hour PMP on 1-mi^2 area for the site is taken from the Hydrometeorological Report No. 52 (HMR No. 52). The 1-hour PMP is nested in 6-hour, 10-mi^2 PMP values given in Hydrometeorological Report No. 51 (HMR No. 51) and is considered the point rainfall value. This 1-hour, 1-mi^2 PMP is distributed into values for small durations. The 1-hour LIP at the site is equal to 17.6 inches. The 5 minute LIP at the site is equal to 5.91 inches.

The Rational formula was used in estimating the peak runoff from the area. The coefficient of runoff was assumed conservatively to be 1.0. The time of concentration was computed from Kirpich's formula.

The water surface elevation was estimated for peak flow over the peripheral roads and railroad track through the openings between the peripheral concrete barriers and through the openings between the security concrete barriers using a broad-crested weir formula with a coefficient of discharge of 2.64. Whenever the tail water level is higher than the weir crest, submergence factors are applied to the coefficient of discharge. Backwater calculations are performed for the areas from the peripheral roads and railroad tracks upstream to plant buildings to estimate the maximum water level adjacent to the buildings housing the safety-related equipment.

The CLB maximum water surface elevation at the site due to the LIP event is equal to 870.90 feet USGS 1929 on the east side of the plant.

The roof loads due to the LIP are determined assuming that the roof drains are clogged at the time of the LIP. The maximum water accumulation on structures is limited by the height of parapet walls, which is equal to 1 foot 4 inches (UFSAR).

2.2.2 Flooding in Streams and Rivers

A synthetic standard project storm was constructed on the basis of the method described in the U.S. Army Corps of Engineers (USACE) publication EM-1110-2-1411, titled, "Standard Project Flood Determination," using a depth equal to one-half of the probable maximum precipitation given in the Hydrometeorological Report No. 33 (HMR No. 33).

Initial precipitation retention of 1 inch followed by uniform hourly retention rates based on soil and land use was used in deriving the SPF. Hourly retention rates ranged from 0.04 in/hr to 0.40 in/hr (UFSAR). Unit hydrographs for sub-basins of the Rock River drainage area have been derived by the USACE. Snyder's method was used to derive the unit hydrographs. The basin lag coefficients (C_t) and peaking coefficient (C_p) were based on those determined by the USACE from recorded floods for the same general area. However, the peak of the unit hydrograph for Sub-basin I, at the upper end of the watershed, was increased by 25 percent. This adjustment was made to account for the relatively large routing effect of the lakes and marshes in the sub-basin. The Muskingum method was used for channel routing. The routing parameter was approximated by travel time determined by the USACE, and the parameter X was selected as 0.2. The peak discharge of the standard project flood at the site is 154,000 cfs.

The peak discharge of the probable maximum flood for the Rock River at the intake was calculated to be 308,000 cfs. This discharge was derived by doubling the peak discharge of the standard project flood.

The water surface elevations of the Rock River near the project site corresponding to the PMF, combined event flood, SPF, flood of record, mean annual flow, and lowest 1-day flow were determined by computer backwater analysis using HEC-2 software. The overbank elevations of the cross sections were measured from United States Geological Survey (USGS) maps with a scale of 1:62500 and contour interval of 20 feet; channel data were measured from a set of USACE maps surveyed during 1912-1913 showing river depths, with a scale of 1:4800. Distances between sections were measured from the same maps for channel and left and right overbank reaches. Flow roughness coefficients for channel and overbank flow were estimated on the basis of the coefficients determined by the USACE for the Rock River. PMF water surface elevation at the intake is 708.3 feet USGS 1929 (UFSAR Table 2.4-12).

The significant and maximum wave effects of a coincident 40-mph overland wind were superimposed on the combined flood water level at the river screen house. The wave runups including setups were calculated for the CEF flood are 2.77 feet and 4.71 feet for the significant and maximum waves, respectively. Superimposing the runup values on the combined event flood level at the screen house resulting in a wave runup elevation of 701.45 feet USGS 1929 for significant waves and elevation 703.39 feet USGS 1929 for maximum waves.

Wave effects were not considered for the PMF water surface elevation.

2.2.3 Dam Breaches and Failures

The river screen house, which provides the makeup for the essential service cooling towers, is the only structure which could be affected by the failure of an upstream or downstream dam. Failure of the Rock River dams at Rockford (13 feet high), 22 miles upstream, and at Rockton (11 feet), 44 miles upstream, would create minor flood waves which would dissipate before reaching the site area. Failure of upstream or downstream dams during flood conditions would not affect the supply of makeup water to the plant.

2.2.4 Storm Surge & Seiche

The Byron CLB considers flooding due to surges or seiches is not a design consideration as there are no large bodies of water near the site.

2.2.5 Tsunami

The Byron CLB considers flooding due to tsunami is not a design consideration as the site is not near a coastal area (UFSAR).

2.2.6 Ice Induced Flooding

The flood of February 1937 reached a stage of 14.6 feet due to backwater from ice and is considered to be the worst ice jam flood on the Rock River at Rockton. More recently, in the early spring of 1971, there was major ice flooding approaching the magnitude of the 1937 ice floods; ice jams occurred all the way from Rockford downstream to Grand Detour, 15 miles below the site area. However, the 1937 flood stage was 0.94 feet lower than the second largest recorded flood of March 1975 at Rockton. The computed March 1975 flood reached an elevation of 679 feet USGS 1929 at the intake. Since the safety-related equipment at the river screen house is at elevation 702.0 feet USGS 1929, ice-induced high flood levels were not

expected to have an adverse effect on the performance of the river screen house. Similarly, ice jams are not expected to create low water levels at the river screen house since the water level at the screen house is controlled by the downstream dam at Oregon.

2.2.7 Channel Migration or Diversion

Due to the great width of the Rock River and the relatively flat surrounding terrain, there is little possibility that rock falls, ice jams, or subsidence could completely divert the flow away from the makeup water intake.

2.2.8 Low water Considerations

Low flow frequency analyses for the Rock River at Rockton and at Como were made using the Log-Pearson Type III distribution. A minimum daily flow of 440 cfs was recorded at Como on August 20, 1934. The historical 1-day low flow at the intake is estimated to be 400 cfs and has a recurrence interval of more than 100 years. The corresponding river elevation at the intake is 670.4 feet USGS 1929.

In the unlikely event that emergency cooling water requirements cannot be satisfied by makeup from the Rock River, deep wells will provide makeup to the essential service water cooling tower.

2.2.9 Combined Effect Flood

The combined event flood is defined as a flood on the Rock River having 1×10^{-6} annual probability of exceedance at a 90 percent confidence level. The peak discharge of the combined event flood at the site as computed from flood data from 1915 to 1974 is 178,000 cfs and the flood level corresponding to this discharge is 698.68 feet USGS 1929. When the flood data for 1975 and 1976 are included in the analysis, the resulting combined event flood at the site is calculated as 176,900 cfs. The river screen house is the only structure that could be affected by flooding on the Rock River and is designed for the combined event flood.

2.3 Flood Related Changes to the License Basis

Byron has been evaluated to adequately withstand the effects of flooding in the current licensing basis. Several changes have been identified related to the Byron flood license basis since license issuance. During upgrades to the site that may create an impediment of outflow of precipitation onsite, the effect of local intense precipitation is re-evaluated in accordance with approved calculation methodology and appropriately incorporated into the licensing basis. Due to changes in the evaluated effect of local intense precipitation, several locations in the immediate plant area require protection against flooding. To prevent water from entering the areas where essential equipment/systems are located, reinforced concrete curbs or steel barriers are provided at the following locations (UFSAR Section 2.4.2.3):

a. External hatches to the RWST tunnel
b. Radwaste building access to the transfer tunnel
c. MSIV rooms adjacent to 401' elevation exterior access doors
d. Personnel access locations to the Auxiliary Building from the FHB 401 feet elevation

The only safety related equipment at Byron Station that could be impacted by flooding along the Rock River is the Essential Service Water (SX) makeup pumps. These pumps are installed in the river screen house at elevation 702 feet USGS 1929 and are housed within a 4 foot high fire

wall. The walls of these rooms are designed to be flood protected to elevation 706 feet USGS 1929. Further evaluation has been completed, and it was it was determined the lower battery post elevation is the limiting height under flood conditions. Additional information is included in Section 2.5.

2.4 Changes to the Watershed and Local Area since License Issuance

The Byron watershed has a total area of approximately 8,174 square mile. 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 Byron include buildings, parking lots, and security barrier upgrades that have been added to the site since license issuance.

2.5 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

The only safety related equipment at Byron Station that could be impacted by flooding is the Essential Service Water (SX) makeup pumps. These pumps are installed in the river screen house at elevation 702 feet USGS 1929 and are housed within a 4 foot high fire wall. The walls of these rooms are designed to be flood protected to elevation 706 feet USGS 1929. The combined event flood (CEF) still-water level is 698.68 feet USGS 1929. Superimposing the wind wave effect will result in a maximum wave runup elevation of 703.39 feet USGS 1929 (UFSAR Section 2.4.3.9). The engine for the SX makeup pumps is mounted on its sub-base at elevation 703.71 feet USGS 1929. The engine shaft centerline is at elevation 705.33 feet USGS 1929 and the lower battery post elevation is approximately 703.67 feet USGS 1929. It is anticipated that this elevation would be limiting under flood conditions. This is above the CEF plus wave runup elevation and is anticipated to be the elevation at which the engine would stop. In the unlikely event that the engines are rendered inoperable by a flood level in excess of 703.67 feet, the plant site deep wells will be powered from their respective Unit 1 Engineered Safety Features (ESF) buses. These deep wells will then provide makeup for the SX cooling towers (UFSAR Section 2.4.10).

Safety-related equipment is located below grade in the Auxiliary and Containment buildings. No safety-related equipment is located in the Turbine or Radwaste buildings. All of the buildings with exterior walls below grade, including the Turbine and Radwaste buildings, are designed to be water tight up to plant floor elevation 870 feet USGS 1929, which is above the grade elevation 869 feet. Potential sources of external flooding of the main power block buildings are LIP and groundwater ingress. The design basis groundwater elevation is 824 feet USGS 1929. All substructures below elevation 869 feet USGS 1929 at the Byron site are designed to withstand full hydrostatic head of groundwater (DC-ST-03-BY/BR Section 12.1.4). LIP has been determined to result in a maximum water surface elevation of 870.82 feet USGS 1929 in the immediate station area where safety-related facilities are located (Auxiliary, Containment and Fuel Handling buildings). Local surface drainage systems are assumed not to function during the LIP event. The areas surrounding the plant are graded to direct surface runoff away from the plant (UFSAR Section 2.4.2.3). To prevent water from entering areas containing essential equipment/systems, incorporated reinforced concrete curbs or steel barriers are provided (UFSAR Section 2.4.2.3).

In general, all flood protection features designed to protect safety-related equipment are passive incorporated features, and as such does not involve invoking any procedures. The only active features are the sump pumps in the SX rooms. These are credited in the current licensing basis as protecting against internal flooding (not part of the NTTF Recommendation 2.3 flooding

walkdowns) but are providing an additional function of removing water leaks from external sources in the SX rooms.

The licensing basis does not explicitly address flood duration or adverse weather conditions concurrent with flooding, presumably because the protection features are all incorporated passive. In addition, Byron flood protection features are designed to function during any plant mode of operation.

3. SUMMARY OF FLOOD HAZARD REEVALUATION

Flooding hazards from various flood-causing mechanisms are evaluated for Byron in accordance with Enclosure 2 of the NRC's March 12, 2012, 50.54(f) Request for Information Letter (NRC March 2012).

Following the guidance outlined in NUREG/CR-7046 *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America* (NUREG/CR-7046), the Hierarchical Hazard Assessment (HHA) approach is utilized in the reevaluation study. The HHA approach is a progressively refined, stepwise estimation of site-specific hazards that evaluates the safety of SSCs with the most conservative plausible assumptions consistent with available data. Consistent with the HHA approach, flooding mechanisms that are determined to be not applicable for the site are screened out using qualitative and quantitative assessments with conservative, simplified assumptions and/or physical reasoning based on physical, hydrological and geological characteristics of the site. For the flooding mechanisms that can potentially affect the design basis, detailed analyses are performed based on present-day methodologies, standards and available engineering practices.

This section describes in detail the reevaluation analysis performed for each plausible flooding mechanism: flooding due to local intense precipitation, flooding on Rock River, storm surge and seiche, ice induced flooding and channel migration, and combined effects flood.

The methodology used in the flooding reevaluation study performed for Byron 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);
- NUREG/CR-6966 "Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America" dated March 2009 (NRC NUREG/CR-6966);
- American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS-2.8-1992), dated July 28, 1992;
- NRC JLD-ISG-2012-06, "Guidance for Performing a Tsunami, Surge and Seiche Flooding Safety Analysis", Japan Lessons-Learned Project Directorate Interim Staff Guidance, Revision 0 (NRC ISG-2012-06)

- NRC JLD-ISG-2013-01, "Guidance For Assessment of Flooding Hazards Due to Dam Failure", Japan Lessons-Learned Project Directorate Interim Staff Guidance, Revision 0 (NRC ISG-2013-01)

The following provides the flood causing mechanisms and their associated water surface elevations that are analyzed in the Byron flood hazard reevaluation study.

3.1 Effects of Local Intense Precipitation

LIP is an extreme precipitation event at the site location. The LIP is equivalent to the 1-hour, 1-sq.mi. PMP as described in NUREG/CR-7046.

The effects of the LIP are the resulting impacts from the flood water surface elevations, flow velocities and impact loads. The effects of the LIP are computed for the safety-related structures at Byron. The effects of LIP are determined in Calculation BYR13-FUK-06, Local Intense Precipitation Analysis (Fukushima) (Exelon 2014f). The assumptions associated with Effects of LIP are listed in Calculation BYR13-FUK-06.

3.1.1 Inputs

The inputs for the analysis are described below.

3.1.1.1 Local Intense Precipitation

The site-specific study for the LIP is performed in Calculation L-003859, Beyond Design Basis Site-Specific Local Intense Precipitation Analysis (Fukushima) (Exelon 2014l). The approach, methodology, and assumptions utilized in the site-specific hydrometeorological study is outlined in detail in Calculation L-003859.

3.1.1.2 Ground Surface Topography

Site topography was based on Light Detection and Ranging (LiDAR) data (ENERCON 2013) for the power block and adjacent areas provided in Geographic Information System (GIS) format supplemented by digital elevation model (DEM) information for Ogle County, Illinois from the National Elevation Dataset (managed by the USGS National Geospatial Program; (USGS 2013a). A single DEM for use in the FLO-2D model was developed in ArcMap 10.1 from the LiDAR data (ENERCON 2013) and from the DEM data (USGS 2013a). A limited site survey was conducted to verify accuracy of the LiDAR data and to capture pertinent site information such as vehicle barriers, culverts etc. The DEM included the LIDAR data, vehicle barriers, and other pertinent site information.

3.1.1.3 Manning's Roughness Coefficients

A Manning's Roughness Coefficient GIS Shapefile was created by correlating Manning's Roughness Coefficients with land cover selected from visual assessment of the aerial photography (ESRI 2013).

3.1.2 Methodology

The Effects of LIP analysis uses a two-dimensional (2D) hydrodynamic model, the FLO-2D model (FLO-2D). FLO-2D is a physical process model that routes flood hydrographs and rainfall-runoff over unconfined flow surfaces or in channels using the dynamic wave approximation to the momentum equation. FLO-2D moves flood volume on a series of tiles (grids) for overland flow or through stream segments for channel routing. Overland flood

routing in two-dimensions is accomplished through a numerical integration of the equations of motion and the conservation of fluid volume for a water flood (FLO-2D).

The method of analysis for FLO-2D simulation is summarized in general terms below:

- Delineate FLO-2D Model computational boundary.
- Generate FLO-2D Model grid elements.
- Calculate Manning's roughness coefficients.
- Assign boundary conditions.
- Assign rainfall hyetographs.
- Perform LIP induced flood simulation to establish maximum water surface elevations at Byron resulting from the LIP.

Following the guidance outlined in NUREG/CR-7046, runoff losses are ignored. The roof rainfall is assumed to be contributing to the overland runoff. The drainage system at the site is assumed to be non-functional at the time of the LIP event.

The 1-mi^2, 1-hr PMP for input into the LIP for Byron was determined to be 13.6 inches (Exelon 2014l).

The upstream areas around the site are included in the computational boundary to account for all runoff possibly contributing to the runoff at the site as shown in Figure 2.1.2. The north boundary is approximately 1,300 feet north of the pre-screening facility. The east boundary varies from approximately 350 to 700 feet east of North German Church Road. The southern boundary is located approximately 2,200 feet south of the southern natural draft cooling tower. The west boundary is immediately west of the transformer yard.

The DEM data provided from the site survey, supplemented by DEM data for Ogle County, was used to develop the FLO-2D model. The elevations at Byron are referenced to USGS 1929 vertical datum as stated in the UFSAR. No such datum exists; however it is assumed that the datum refers to National Geodetic Vertical Datum of 1929 (NGVD 1929). The ground surface topography and the site-related features are referenced to North American Vertical Datum of 1988 (NAVD88).

The vertical datum difference between NAVD88 and NGVD 1929 (or MSL) is obtained using VERTCON (NOAA 2013e). In order to use VERTCON, it is assumed that where the UFSAR refers to MSL, the vertical datum is actually NGVD29. Based on VERTCON, the vertical datum difference is NAVD88 – NGVD 1929 (or MSL) = 0.14 feet. To be consistent with the CLB, the elevations within this report are referenced to USGS 1929 datum.

3.1.3 Results

The maximum water surface elevation in the vicinity of the power block is 870.8 feet USGS 1929 and occurs to the east and south of the Turbine Building. Reinforced concrete curbs or steel barriers were provided above elevation 870.82 feet USGS 1929 (Exelon 2012). The maximum water surface elevation in the Immediate Station Area is 870.7 feet USGS 1929 and is below the elevation of the concrete curb/ steel barrier as described in Flooding Walkdown Report (Exelon 2012).

3.1.4 Conclusions

The reevaluated LIP elevations are below CLB LIP elevation of 870.90 feet USGS 1929.

3.2 Probable Maximum Flood on Rock River

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

As outlined in the guidance provided in ANSI/ANS-2.8-1992 and in NUREG/CR-7046, Appendix H, the flood hazards design basis should include several flood-causing mechanisms and combinations of these mechanisms. For the floods caused by precipitation events, the following should be examined:

<u>Flooding in Rivers and Streams</u>

Alternative 1 – Combination of:

- Mean monthly base flow
- Median soil moisture
- Antecedent or subsequent rain: the lesser of 1) rainfall equal to 40% 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.

The precipitation input for Alternative 1, the all-season PMF, and Alternatives 2 and 3, the cool-season PMF, are determined in Calculation BYR13-FUK-01, Probable Maximum Precipitation Analysis (Fukushima) (Exelon 2014a).

The PMF flow hydrograph for Rock River is determined in Calculation BYR13-FUK-02, Byron Nuclear Generating Station: Probable Maximum Flood Analysis- Hydrology (Fukushima) (Exelon 2014b). The PMF water surface elevations for Rock River are determined in Calculation BYR13-FUK-03, Probable Maximum Flood Analysis - Hydraulics (Fukushima) (Exelon 2014c).

The results of the wind wave activity coincident with the PMF and dam failure are determined in Calculation BYR13-FUK-09, Combined Events Analysis (Fukushima) (Exelon 2014i). The

assumptions associated with PMF on Rock River are listed in Calculations BYR13-FUK-01, BYR13-FUK-02, and BYR13-FUK-03.

3.2.1 Inputs

The inputs for the analysis are described below.

3.2.1.1 All-Season PMP

The all-season PMP estimates for Byron are derived from the charts presented in the generalized hydrometeorological reports (HMR No. 51 and HMR No. 52). The PMP estimates are derived based on the site location and site watershed area.

3.2.1.2 100-Year Rainfall

The 100-year rainfall estimates are obtained from NOAA Precipitation Frequency Data Server (NOAA 2013b) for the Byron location.

3.2.1.3 Maximum Dew Point Temperatures

The dew point temperatures are used as an input to the energy budget equation to calculate snowmelt rate. Hourly dew point temperature data from selected Global Hourly station is provided by NCDC (NOAA 2013d).

3.2.1.4 Typical Regional Wind Speed

The wind speeds are used as an input to the energy budget equation to calculate snowmelt rate. Hourly wind speed data at selected Global Hourly station are provided by the NCDC (NOAA 2013d).

3.2.1.5 Snow Data

Using "Median and Extreme Daily Snow Cover by Month", three selected weather stations provided by NCDC, NOAA Satellite and Information Service (NOAA 2013c), identify the months with historically documented snow cover. Statistical snow records were obtained from NCDC (NOAA 2013c) for all three stations. In lieu of site-specific data, the assigned snow water equivalent (SWE) (ratio) was 0.25 to January, 0.30 for February and 0.35 for March and April based on observed SWE data for other parts of the country.

3.2.1.6 Mean Monthly Baseflow

Baseflow was calculated for each sub-watershed based on USGS Monthly data for three USGS Stream Gages: 05427570 on Rock River at Indianford, Wisconsin, 05437500 on Rock River at Rockton, Illinois, and 05440700 on Rock River at Byron, Illinois

3.2.1.7 Soil Data

The Natural Resources Conservation Service (NRCS) data is used (NRCS 2013) to provide the soil data for the site.

3.2.1.8 Surface Roughness Coefficients

Manning's n values for Rock River were developed using orthoimagery photos and using published guidance for selection of Manning's Roughness Coefficients based on "Open-Channel Hydraulics" (Chow 1959). The Federal Emergency Management Agency (FEMA)

Flood Insurance Study (FIS) and the HEC-2 model for Ogle and Lee Counties were also used.

3.2.1.9 Ground Surface Topography

The topographic information used in the calculation is from the FIS for Lee (FEMA 2010a) and Ogle (FEMA 2010b) counties. The HEC-2 water surface profiles data files were obtained from FEMA. The elevation data is referenced to the NGVD 1929 datum.

3.2.2 Methodology

The PMF analysis included the following steps:

- Delineate watershed and sub-watersheds and calculate sub-watershed areas for input into USACE HEC-HMS rainfall-runoff hydrologic computer model.
- Determine PMP.
- Identify candidate floods to use for calibration and verification of the rainfall runoff model. Estimate initial input parameters for the HEC-HMS rainfall runoff model: Snyder's unit hydrograph method – Basin Lag and Peaking Coefficient.
- Calculate initial loss input parameters for the HEC-HMS model: Initial Loss and Constant Loss rate.
- Calculate precipitation gage weights for calibration and verification floods.
- Calibrate and verify HEC-HMS rainfall runoff model utilizing observed USGS stream flow data by optimization of model input parameters: Basin Lag, Peaking Coefficient, Initial Loss, and Constant Loss.
- Perform PMF simulation with PMP input using calibrated and verified HEC-HMS model.
- Maximum water surface elevation determined using the USACE HEC-RAS model.

3.2.2.1 Alternative 1 Precipitation Input

As described above, the precipitation input for the Alternative 1 PMF consists of the all-season PMP and an antecedent storm.

The all-season PMP estimates for Byron are derived using the BOSS HMR-52 software. The depth-area-duration values for 6, 12, 24, 28 and 72-hours and 10, 200, 1,000, 5,000, 10,000 and 20,000 mi^2 (HMR No. 51, Figures 18 through 47) are determined. These values are input into the BOSS HMR-52 software. This program utilizes algorithms to calculate maximum rainfall depths and hyetographs for a given watershed, which are developed based on methods presented in HMR-51 (HMR No. 51) and HMR-52 (HMR No. 52). BOSS HMR52 is an enhanced version of an original program (HMR52) developed by the USACE's Hydrologic Engineering Center (HEC) in 1987.

As a summary, the Alternative 1 precipitation event consists of a 72-hour 40% all-season PMP rainfall, followed by 3 dry days and then 72-hour all-season PMP.

3.2.2.2 Alternative 2 Precipitation Input

The precipitation input for Alternative 2 PMF consists of 100-year, snow-season rainfall and coincident snowmelt from the probable maximum snowpack.

The 100-year rainfall estimates are obtained from NOAA Precipitation Frequency Data Server (NOAA 2013b) for the Byron location. A ratio of cool-season to all-season rainfall is

determined and applied to the 100-year, all-season rainfall depths based on regional guidelines contained in Bulletin 71, Rainfall Frequency Atlas of the Midwest (Huff 1992).

The snowmelt rate is calculated using the energy budget equation for the rain-on-snow condition following the guidance outlined in USACE EM 1110-2-1406, Runoff from Snowmelt (USACE 1998). The dew point temperatures and wind speed used as the input to the energy budget equation are determined from historical data obtained from NCDC for three nearby meteorological stations.

The probable maximum snowpack is assumed to be equal to an unlimited snowpack depth during the entire coincident 72-hour rainfall. While the snowpack can be determined directly from the snow depth, there is not adequate data to reliably extrapolate snowpack for the probable maximum event. Any estimated probable maximum snowpack would have an associated physical limit, i.e., maximum snow depth; therefore, an unlimited snow depth is a conservative assumption.

As a summary, the Alternative 2 precipitation event consists of a 72-hour 100-year, cool-season rainfall coincident with the snowmelt from the probable maximum snowpack.

3.2.2.3 Alternative 3 Precipitation Input

The precipitation input for Alternative 3 PMF consists of the cool-season PMF coincident with the snowmelt from a 100-year snowpack.

The cool-season PMP estimates for the Byron location are determined using the charts provided in the hydrometeorological Report No. 53 (HMR No. 53) for each cool-season month (October to April).

The snowmelt rate is determined for each cool-season month using the energy budget equation for the rain-on-snow condition (USACE 1998). The meteorological parameters used to determine snowmelt for Alternative 2 precipitations are used here as well. The snowmelt for the Alternative 3 PMF is limited by a 100-year snowpack. The 100-year snow depth is determined using statistical analysis based on the historical data for snow depth obtained from the meteorological stations in close proximity to Byron.

As a summary, the Alternative 3 precipitation event consists of a 72-hour cool-season PMP coincident with the snowmelt from a 100-year snowpack.

3.2.2.4 Watershed Delineation

The Rock River watershed and its sub-watersheds were delineated using United States Geologic Services (USGS) StreamStats and verified manually and using Hydrologic Unit Code (HUC) Geographic Information Systems (GIS) data included in the National Hydrography Dataset (NHD) provided by the USGS. The overall watershed was delineated on the main stem of the Rock River near Byron. In total, four sub-watersheds were developed for this calculation based on the location of existing stream gages and HUC boundaries. The watershed area for Woodland Creek is included in the overall Rock River watershed that was analyzed. No separate analysis (i.e. local PMF) of Woodland Creek was performed because the Woodland Creek channel is located below and down gradient of Byron.

3.2.2.5 Infiltration Loss Rate

The initial and constant loss method was used in this calculation. Initial and constant losses are calibrated based on historical data.

3.2.2.6 Unit Hydrograph

The transform method used in this calculation was the Snyder Unit Hydrograph (UH) method. Snyder basin lag time (t_p) and peak coefficient (C_p) are calibrated based on historical data. A portion of the Rock River watershed is ungaged. The Snyder peaking coefficient and constant loss rate for the ungaged subwatersheds were judged to be similar to the parameters for the Rock River at Byron and the Rock River at Rockton gaged subwatersheds (i.e., surrogates) located in the same general geographic area with similar hydrologic characteristics.

The potential for non-linearity to affect the PMF was included as per the recommendations of NUREG/CR-7046. While the storms utilized in the calibration and verification process are the largest historic floods, the PMF is anticipated to be significantly larger than the recorded historical floods. NUREG/CR-7046 recommends increasing the peak discharge of the unit hydrograph by one-fifth and decreasing the time to peak by one-third.

3.2.2.7 Temporal Distribution

The temporal order of the twelve 6-hour increments is: 12, 10, 8, 6, 4, 2, 1, 3, 5, 7, 9, 11, where the numbers indicate the largest precipitation intensity (1) to the smallest (12). This is the default temporal order used by the software.

3.2.2.8 Hydrologic Modeling

Hydrologic modeling is performed using USACE HEC-HMS computer software. The various precipitation estimates are applied to the basin models using time series precipitation gages. Base flow is applied as a constant flow rate. The initial and constant loss method is used. A user-specified unit hydrograph rainfall runoff transform method is used by applying the modified unit hydrographs that account for the effects of nonlinear basin response. The HEC-HMS model produces results for the water surface elevation in the flow hydrograph for Rock River for future use in the hydraulic model.

3.2.2.9 Hydraulic Modeling

Hydraulic modeling for Rock River is performed using USACE HEC-RAS computer software. Cross sections are designated along the Rock River based on bathymetric data from the USACE HEC-2 files and topographic information from the Illinois State Geologic Survey DEM using ArcGIS 10.1 computer software. The HEC-RAS unsteady-state model is used to analyze the flow hydrographs obtained from the HEC-HMS hydrologic model to determine the stillwater surface elevation at each cross section.

The controlling PMF scenario is determined to be from Alternative 2, a probable maximum snowpack with a 100-year cool-season rainfall. The PMF flow hydrographs are entered into the HEC-RAS model at the upstream end of the model and at the intermediate points representing other tributary rivers/streams.

The HEC-RAS model includes two (2) dam structures (Oregon and Dixon Dams). The dams are included based on the information contained in the National Inventory of Dams,

the FEMA Flood Insurance Study's for Ogle and Lee counties, and FEMA HEC-2 model. Major bridge structures within the modeled reach of the Rock River are also incorporated into HEC-RAS. Bridge geometry data is based on aerial images and state agency data.

The hydraulic model is calibrated for unsteady state flow using extreme historic flow data at multiple gage locations. Model calibration is the process of selecting and refining HEC-RAS input parameters to produce a simulated profile for a given flood that shows consistent agreement with an observed profile for the same flood. The consistency of agreement is ultimately based on engineering judgment; however, for this calculation a target elevation difference of 1 feet or less is utilized. The main parameter that is adjusted in the calibration is the Manning's n.

3.2.2.10 Vertical Datum

The elevations at Byron are referenced to USGS 1929 vertical datum as stated in the UFSAR. The ground surface topography and the site-related features are referenced to NAVD88. The vertical datum difference between NAVD88 and MSL is discussed in Section 3.1.2.

3.2.3 Results

The Alternative 2 (Probable Maximum Snowpack and 100-year cool-season rainfall) resulted in the controlling PMF at Byron. The PMF peak discharge in the Rock River at Byron resulting from Alternative 2 is 193,800 cfs, including non-linearity adjustments as per NUREG/CR-7046.

The calculated peak PMF stillwater surface elevation in the Rock River at the Byron river screen house, based on a maximum discharge after hydraulic routing within HEC-RAS of 193,200 cfs, is 699.2 feet USGS 1929. It is noted that the modeled flow at the Byron river screen house is approximately 0.4 percent less than the peak flow in the input hydrograph (193,800 cfs). This is due to the slight attenuation of the hydrograph in the ten mile reach of the Rock River between the Byron river screen house and the upstream limit of the model, which is realistic given the capacity of the floodplain.

The PMF stillwater surface elevation, 699.2 feet USGS 1929, is 6.8 feet below the four foot high fire wall (at elevation 706.0 feet USGS 1929) surrounding the safety-related equipment in the river screen house and 169.8 feet below the plant grade (at elevation 869 feet USGS 1929) (UFSAR).

3.2.4 Conclusions

The PMF stillwater surface elevation, 699.2 feet USGS 1929, is well below the CLB PMF elevation of 708.3 feet USGS 1929.

3.3 Storm Surge, Seiche, and Tsunami Screening

The Probable Maximum Storm Surge (PMSS) and Probable Maximum Seiche (PMS) are analyzed in Calculation BYR13-FUK-07, Probable Maximum Surge, Seiche, and Tsunami (Fukushima) (Exelon 2014g).

A storm surge evaluation is performed following the guidance outlined in NUREG/CR-7046, ANS-2.8-1992, JLD-ISG-2012-06 (NRC ISG-2012-06), and NUREG/CR-6966. NUREG/CR-7046, Appendix H.4, describes the combined events criteria for an enclosed body of water, which is appropriate for analyzing surge and seiche flooding along Rock River, the natural draft cooling

tower and essential service cooling tower basin. NRC JLD-ISG-2012-06 requires: "all coastal nuclear power plant sites and nuclear power plant sites located adjacent to cooling ponds or reservoirs subject to potential hurricanes, windstorms, and squall lines must consider the potential for inundation from storm surge and wind waves." Byron is not a coastal location; however, the Rock River, the natural draft cooling tower and essential service cooling tower basin are analyzed to determine if they could be subjected to storm surge (wind setup and wave setup) due to severe wind storms. The assumptions associated with Storm Surge, Seiche, and Tsunami screening are listed in Calculation BYR13-FUK-07 (Exelon 2014g).

3.3.1 Inputs

The inputs for the analysis are described below.

3.3.1.1 Rock River Bathymetry

Bathymetry for the Rock River was determined from a USACE HEC-2 model used by FEMA to determine the flood profile for Rock River for the FIS for Lee (FEMA 2010a) and Ogle (FEMA 2010b) counties.

3.3.1.2 Ground Surface Topography

Site topography is discussed in Section 3.1.1 above.

3.3.2 Methodology

The flooding potential at Byron, resulting from storm surge and seiche, is evaluated. Seiche is defined as an oscillation of the water surface in an enclosed or semi-enclosed body of water initiated by an external cause (NUREG/CR-7046, Section 3.6). A storm surge is the rise in water surface elevation along shorelines produced largely by the shear force of the storm winds (NUREG/CR-7046, Section 6.5).

A tsunami is a series of water waves generated by a rapid, large-scale disturbance of a water body due to seismic, landslide, or volcanic tsunamigenic sources. Therefore, only geophysical events that release a large amount of energy in a very short time into a water body generate tsunamis. The most frequent cause of tsunamis is an earthquake. Less frequently, tsunamis are generated by subaqueous and sub-aerial landslides.

3.3.2.1 Storm Surge

Storm surge is the rise in offshore water elevation caused principally by the shear force of wind due to hurricanes, extra-tropical storms, or squall lines acting on the water surface. A secondary rise in water surface is also caused by the lowering of the air pressure within a low pressure storm system (NUREG/CR-7046). The Probable Maximum Storm Surge (PMSS) is the surge that results from a combination of meteorological parameters of a Probable Maximum Hurricane or Probable Maximum Wind Storm.

JLD-ISG-2012-06 requires: "all coastal nuclear power plant sites and nuclear power plant sites located adjacent to cooling ponds or reservoirs subject to potential hurricanes, windstorms, and squall lines must consider the potential for inundation from storm surge and wind waves." However, Byron is not a coastal location. Byron is located approximately 820 miles from the Gulf of Mexico coast, 800 miles from the Atlantic Ocean coast and 82 miles from Lake Michigan. Additionally, plant grade is approximately 869 feet USGS 1929

and above coastal storm surge levels. The Rock River near Byron is not a tidal river, as it is approximately 672 feet USGS 1929.

The basins for the circulating water natural draft cooling tower and the essential service cooling tower have obstructed fetches. Therefore wave generation is limited and the basins are not subject to surge. Evaluation of the PMSS is not warranted based upon the distance between Byron and the coast.

3.3.2.2 Storm Seiche

A description of the site's physical setting (elevation and floodplain geometry) was used to demonstrate that the effects of a potential seiche in the Rock River will be physically confined by the geometry of the floodplain. A literature review of listed sources for historical seiche instances in the region was also performed.

Rock River
The river is approximately 650 feet wide in the vicinity of the screen house. There is not a smooth transition from the Rock River to its uplands. Additionally, the uplands surrounding the river (east/west and north/south) are not symmetric. The varied topography adjacent to the river, in particular the asymmetry of the floodplains beyond the banks, would prevent resonance of a hypothetical seiche. The mean annual elevation of the Rock River is 673.2 feet USGS 1929.

Seismic seiches resulting from the March 1964 Alaska earthquake were recorded at more than 850 surface water gaging stations in North America and at four in Australia (USGS 1966). The authors of a 1966 USGS report on the 1964 Alaska earthquake concluded that neither the direction nor distance from the epicenter had any apparent effect on the distribution of seiches that resulted. The most important factor is the variation of thickness of low-rigidity sediments. The presence of major tectonic features such as thrust faults, basins, arches and domes seem to control seismic waves and thus affect the distribution of seiches. Additionally, they noted that lateral refraction of seismic surface waves due to variations in local phase-velocity values was responsible for increase in seiche density in certain areas. Finally, the authors concluded that because both seiches and seismic intensity depend on the horizontal acceleration from surface waves, the distribution of seiches may be used to map the seismic intensity that can be expected from future local earthquakes (USGS, 1966).

Within the United States, 763 of 6,435 USGS surface water gages registered seiches as a result of the 1964 Alaska earthquake. According to the report, seiches were not recorded at the gaging stations on the Rock River. However, a seiche with double amplitude of 0.01 feet was recorded on the East Branch of the Rock River near Mayville, Wisconsin (USGS 1966). The absence of significant observed seismic seiche during the Alaska earthquake demonstrates that the Rock River is not prone to seiche development. Furthermore, any forced oscillation of the Rock River could send water into the floodplain but it would not be able to reach Byron 196 feet above and 2.0 miles away from the Rock River. Thus, the physical setting of Byron is not conducive to flooding from a potential seiche at Byron.

Natural Draft Cooling tower and Essential Service Cooling Tower Basin
Seiche potential was also analyzed for the on-site open water reservoirs associated with the cooling water systems. Merian's Formula provides a method for estimating the natural

periods for the seiche modes in enclosed and semienclosed basins in accordance with NUREG 7046 (NUREG/CR-7046, Appendix F).

Byron has two circulating, natural draft cooling towers on site. These towers use water from circular basins at their base, which are approximately 605 feet in diameter and have a 7 foot maximum depth. The towers are connected by a trapezoidal flume that is approximately 128 feet long and 22 feet at the deepest point. Byron also has an essential service mechanical cooling basin that is comprised of eight cells that are approximately 41 feet by 41 feet with a 7.5 foot normal depth. Each cell was treated as its own basin due to structural elements which effectively serve as baffles between each cell (Exelon, 1997a & Exelon 1997b). For the purpose of these analyses, all on-site reservoirs were modeled as enclosed, rectangular bodies with respect to the application of Merian's Formula.

Using the above dimensions, the periods for the respective basins, based on Merian's Formula, ranged from 5.3 seconds to 80.6 seconds in both the longitudinal and transverse directions. These periods were used as inputs to the earthquake nomographs for the site to generate expected accelerations which might drive the formation of a seiche in each basin. The horizontal response spectra for both the operational and shutdown earthquakes show only small accelerations for the periods in this range. Therefore, seiches in the longitudinal and transverse directions are not expected to present a risk in any of the cooling basins examined at Byron.

3.3.2.3 Tsunami

Tsunamis are generated by rapid, large-scale disturbance of a body of water. Therefore, only geophysical events that release a large amount of energy in a very short time into a water body generate tsunamis. The most frequent cause of tsunamis is an earthquake. Less frequently, tsunamis are generated by subaqueous and sub-aerial landslides. As noted in NUREG/CR-6966, meteorite impacts, volcanoes, and ice falls have the potential to result in tsunami but are not considered herein in accordance with accepted international practice for tsunami hazard assessment.

NGDC Database Review

The Global Historical Tsunami Database (NOAA 2013a), maintained by the National Oceanic Atmospheric Administration's National Geophysical Data Center (NGDC), is reviewed to determine the history of tsunamis in the region. The regional survey considered tsunami-like waves in the area around Byron, extending from 37° to 47° N Latitude and 84° to 94° W, an approximately 700 mile by 700 mile area.

Earthquakes

A substantial amount of slip and a large rupture area is required to generate a major tsunami. Consequently, only large earthquakes with magnitudes greater than 6.5 generate observable tsunamis (NUREG/CR-6966, Section 1.3.1). The USGS Illinois Earthquake History website (USGS 2013b) is reviewed to determine the largest earthquake in Illinois history.

Landslides

There are two broad categories of landslides which are relevant to the generation of tsunamis: (1) subaqueous that are initiated and progress beneath the surface of the water body, and (2) sub-aerial that are initiated above the water and impact the water body during their progression or fall into the water body.

The geographical areas where sub-aerial landslides occur are generally limited to areas of steep shoreline topography (NUREG/CR-6966, Section 1.3.2). The USGS compiled a map of landslide incidence and susceptibility for the contiguous United States. Susceptibility to a landslide is classified as high, medium or low based on the probable degree of response of soil and rock to cutting or loading of slopes or abnormally high precipitation.

The outgoing wave from a subaqueous landslide source propagates in the direction of the slide with its amplitude affected by the terminal velocity of the movement, which in turn is a function of the repose angle, i.e., the slope angle (NUREG/CR-6966, Section 1.3.2). A cross section of the Rock River channel near Byron is included in the 2012 update to the Byron UFSAR. The maximum slope calculated from the cross section is less than ten degrees.

The regional survey demonstrated that tsunamis have been generated on the Great Lakes but have not otherwise been generated near the site. Earthquakes with a magnitude sufficient to generate a tsunami have not been recorded in Illinois but have been generated in the New Madrid Seismic Zone (approximately 380 miles away). Sub-aerial landslide incidence is low. The potential for subaqueous landslide with a significant velocity is low. In summary, there is a potential for tsunami in the vicinity of Byron but the area of potential tsunami is limited to the area in or near the Great Lakes.

Site Screening Assessment

Byron is located approximately 820 miles from the Gulf of Mexico coast and 800 miles from the Atlantic Ocean coast. Byron is approximately 82 miles west of Lake Michigan and 2.0 miles east of the Rock River. The Byron site grade is typically at elevation 869 feet USGS 1929. The mean annual Rock River water surface elevation is 673.2 feet USGS 1929. Thus the vertical difference between the typical Rock River level and the plant site grade is approximately 195.8 feet. The vertical difference between the plant site grade and the maximum average annual water surface elevation Lake Michigan is approximately 288.8 feet USGS 1929.

A tsunami on the Rock River would need to have a wave height in excess of 195 feet to affect Byron. It is not anticipated that a tsunami of sufficient magnitude to cause flooding at Byron could be generated on the Rock River due to the elevation difference, the typical depth of water within the Rock River, and the distance of Byron from the river. The river screen house sits on the river bank at the edge of the floodplain. The Essential Service Water Makeup pumps are protected up to elevation 703.7 feet USGS 1929, which is approximately 22 feet above the surrounding floodplain and 30 feet above the typical surface water elevation of the Rock River.

A tsunami on Lake Michigan would need to have a wave height in excess of 288 feet to affect Byron. It is not anticipated that a tsunami of sufficient magnitude to cause flooding at Byron could be generated on Lake Michigan due to the elevation difference and the distance of Byron from Lake Michigan.

3.3.3 Results

Surge
Flooding of Byron due to storm surge is not anticipated because Byron is not a coastal site, because the open water lengths of the Rock River are too small for a surge to develop, and because obstructed fetches limit the shear forces that wind can apply to the Rock River. The cooling basins are not subject to surge due to obstructed fetches.

Seiche
Byron site settings and the elevation margin from the Rock River precludes flooding of the Byron due to seiches. Seiches at the cooling basin are not expected due to small magnitude of accelerations at periods in range of the periods calculated for the basins that can cause resonance.

Tsunami
Two events recorded by NGDC occurred within the region queried. The first event was in 1895 and was the result of a 6.7 magnitude earthquake in Charleston, MO. The second event occurred in 1954 and was likely the result of a meteorologic source (and thus not a true tsunami). The maximum recorded run-up associated with a historical tsunami was 3.0 feet due to the 1954 hurricane.

The USGS Illinois Earthquake History website (USGS 2013c) shows that the largest earthquake in Illinois occurred in 1968 and had a magnitude of 5.4, which is below the threshold of a magnitude 6.5 earthquake.

The river channel in the vicinity of Byron is judged unlikely to generate a subaqueous landslide which would result in a tsunami-like wave that could affect the Byron site. Landslide with tsunami potential may occur at steeper sections of the Rock River (that are not nearby to Byron) however they would not result in tsunami waves with the potential to affect the site.

3.3.4 Conclusions

Based on a screening analysis, flooding due to storm surge, seiches, and tsunami is not expected to affect Byron. This is consistent with the CLB.

3.4 Dam Breaches and Failures

The impact of dam failure is analyzed in Calculation BYR13-FUK-04, Dam Failure Analysis (Fukushima) (Exelon, 2014d).

A dam assessment and dam failure evaluation is performed following the guidance outlined in JLD-ISG-2013-01 (NRC ISG-2013-01) and NUREG/CR-7046. The criteria for flooding from dam breaches and failures evaluation is provided in NUREG/CR-7046, Appendix D (NUREG/CR-7046). Two scenarios of dam failures are recommended and discussed in NUREG/CR-7046, Appendix D including:

1. Failure of individual dams
2. Cascading or domino-like failures of dams upstream of the site.

The methodology adopted in this calculation for the upstream dams is largely based on JLD-ISG-2013-01 (NRC ISG-2013-01), "Peak Outflow with Attenuation" method. The assumptions associated with Dam Failure analysis are listed in Calculation BYR13-FUK-04 (Exelon 2014d).

3.4.1 Inputs

The inputs for the analysis are described below.

3.4.1.1 Upstream Dam Information

Dam height and reservoir storage capacity from the National Inventory of Dam (USACE 2013b) Database maintained by the USACE is gathered for all dams upstream of Byron.

3.4.1.2 PMF Flow Hydrographs

PMF hydrographs were developed in Calculation BYR13-FUK-02, Byron Nuclear Generating Station: Probable Maximum Flood Analysis- Hydrology (Fukushima) (Exelon 2014b).

3.4.1.3 Hydraulic Models for Rock River and the LIP

The HEC-RAS model developed to determine the water surface elevations at the Byron screen house was developed in Calculation BYR13-FUK-03, Probable Maximum Flood Analysis - Hydraulics (Fukushima) (Exelon 2014c). The FLO-2D model developed in support of the Local Intense Precipitation Analysis was developed in Calculation BYR-13-FUK-06, Local Intense Precipitation Analysis (Fukushima) (Exelon 2014f).

3.4.1.4 Dam Information for Natural Draft Cooling Tower and Ultimate Heat Sink Basin Dams

Natural Draft Cooling Tower (NDCT) and UHS basin characteristics were determined from historical information from the site.

3.4.2 Methodology

JLD-ISG-2013-01 (NRC ISG-2013-01) states "In general, failure of any dam upstream from the plant site is a potential flooding mechanism (consideration of upstream dams should include all water-impounding structures, whether or not they are defined as dams in the traditional sense). Dams that are not upstream from the plant, but whose failure would impact the plant because of backwater effects, may also present potential flooding hazards. Failures of dikes or levees in the watershed surrounding the site may contribute to or ameliorate flooding hazards, depending on the location of the levee and the circumstances under which it fails." The impact of dam failure on Byron is considered.

The methodology used for the dam failure evaluation of watershed dams is as follows:

1. Identify dams upstream of Byron using the NID.
2. Determine the total storage and storage-weighted height for the upstream dams; assuming maximum storage for each dam (i.e. top of dam).
3. Calculate the peak dam breach outflow using a regression equation.
4. Calculate the attenuation of peak outflow from dam break to the river screen house.
5. Use the attenuated constant peak outflow in the unsteady HEC-RAS model to determine the maximum water surface elevation due to dam failure during the PMF.

3.4.2.1 Identify Major Upstream Dams

The National Inventory of Dams (USACE 2013b) was used to obtain dam coordinates that were then mapped into the Rock River watershed to identify dams upstream of Byron.

3.4.2.2 Estimate Storage Volume and Representative Dam Height of Hypothetical Dam

The dams identified in the NID were "combined" and modeled as one hypothetical dam at the nearest upstream dam, 12 miles upstream of Byron. The storage of the hypothetical reservoir is equal to the sum of the storage of the dams upstream of Byron. The hypothetical dam height was calculated as a storage-weighted height of the dams upstream of Byron.

3.4.2.3 Estimate Peak Dam Breach Outflow

The peak breach outflow for the hypothetical dam was calculated using three different equations (described below). The highest resulting peak breach outflow was then selected for use in modeling the resultant water surface elevation. The selected equations use only reservoir storage and/or dam height as input parameters. This avoids the need to estimate time to breach and breach width parameters. The equations used are:

- The United Stated Bureau of Reclamation (USBR) peak breach outflow equation (USBR, 1982).
- The NRCS (formerly Soil Conservation Service) peak breach outflow equation (NRCS 1985).
- The Froehlich peak breach outflow equation (Froehlich, 1995).

The distance between the nearest upstream dam and Byron along the Rock River was measured using the measurement tool in ArcMap. Attenuation of the peak dam breach outflow was calculated using the USBR empirical attenuation formula (USBR 1982).

3.4.2.4 Perform Hydraulic Simulation

The Rock River peak water surface elevation (or the flood stage) resulting from the combination of upstream dam breach and the PMF at Byron was calculated using the unsteady-flow HEC-RAS model. The attenuated peak dam breach outflow was conservatively input into HEC-RAS as a steady-state lateral inflow approximately 0.1 miles upstream of the river screen house.

The methodology used for the dam failure evaluation of on-site dams is as follows:

- Identify the basins with a potential for failure.
- Identify dam failure parameters.
- Calculate peak breach outflow in HMS.
- Perform simulation of dam failure in FLO-2D and estimate maximum water surface elevations throughout the Byron site.

3.4.2.5 Evaluate On-Site Basin Failure

The Natural Draft Cooling Towers (NDCT) Flume and the Essential Service Cooling Tower/ Ultimate Heat Sink (UHS) Basin were assessed for potential dam failures. The NDCT Basin is a rectangular basin constructed with concrete walls and extends two feet above grade. The Essential Service Cooling Tower / UHS Basin is a rectangular basin constructed with concrete walls and extends 4.0 feet above grade. An initial screening was done to determine whether dam failure was possible due to a hydrologic event (i.e.

PMF), a seismic event coincident with the ½ PMF or embankment failure due to piping through the embankment (sunny-day failure) for each basin. Scenarios that were determined to lead to dam failure were then modeled using HEC-HMS and FLO-2D.

<u>NDCT Basin</u>
The peak water surface in the basin for the ½ PMF and PMF was calculated using a mass balance approach. Outflows (discharges via water use, withdrawals, or blowoff pipes) were conservatively not considered. A water surface elevation equal to the normal water elevation of 873.0 feet USGS 1929 for the NDCT was assumed at the onset of the ½ PMF. Precipitation falling upon the NDCT openings and on the flume water surface was contained within the basin to produce the resultant water surface elevation in the NDCT basin.

The USACE HEC-HMS computer model was used to calculate the basin failure outflow due to dam failure of the NDCT Basin with the PMF water surface elevation. The model was used to calculate the effects of both the hydrologic failure scenario and the seismic failure scenario (the sunny-day scenario was screened out). The maximum water surface in the basin generated by the inflow during the hydrologic scenario (the PMF) bounds the maximum water surface elevation generated by the most conservative input during the seismic failure scenario (½ PMF). Therefore the use of the full PMF was judged to be a conservative input for the seismic failure scenario due to its higher water surface elevation at the onset of failure and thus appropriate under the HHA methodology.

The breach width was conservatively assumed to equal the length of the flume on the west side because the largest breach flow is the results of the largest size breach. The development time was set to 0.1 hours (i.e., nearly instantaneous) because the shorter breach time results in a larger, more conservative breach flow (FERC 1993). The HEC-HMS model did not include analysis of rainfall-runoff because the NDCT Basin is contained by a perimeter concrete structure and thus does not have a natural contributory drainage area.

<u>UHS Basin</u>
The peak water surfaces in the UHS Basin for the ½ PMF and LIP were calculated using a mass balance approach. Outflows (discharges via water use, withdrawals, or blowoff pipes) were conservatively not considered. The UHS Cooling Tower Basin has a normal pool elevation of 876.0 feet MSL (Exelon 2009). There is typically minimal operational variation in the pool level. The surrounding grade is 874.0 feet USGS 1929 and the top of the basin wall (listed as the top of curb) is at 878.0 feet USGS 1929 Exelon 2008).

The USACE HEC-HMS computer model was used to calculate the basin failure outflow due to dam failure of the UHS Basin with the normal operating water surface elevation. The model was used to calculate the effects the sunny-day failure scenario only. The hydrologic and seismic failure scenarios for the UHS Basin were screened out.

The breach width was conservatively assumed to equal the length of the basin on the west side because the largest breach flow is the result of the largest size breach. The development time was set to 0.1 hours (i.e., nearly instantaneous) because the shorter breach time results in a larger, more conservative breach flow (FERC 1993). The HEC-

HMS model did not include analysis of rainfall-runoff because the UHS Basin is considered to fail under the sunny day failure scenario.

The dam failure outflow hydrograph calculated in HEC-HMS was input as an inflow hydrograph into the FLO-2D model developed in Calculation No. BYR13-FUK-06, "Byron Nuclear Generating Station: Local Intense Precipitation Analysis" (Exelon 2014f). The inflow hydrograph was distributed equally over the thirty FLO-2D grid cells adjacent to the west flume wall. The rainfall component (of the LIP model) was not included in this model as the rainfall was input as an inflow hydrograph from the HEC-HMS model.

3.4.3 Results

The attenuated dam breach outflow from upstream dams in the Rock River watershed is 48,350 cfs at Byron. The resultant peak water surface elevation from the combined dam breach peak outflow and PMF in the Rock River at the river screen house is 703.2 feet USGS 1929.

The maximum water surface elevation in the vicinity of the power block due to failure of the NDCT basin and flume under hydrologic dam failure scenario is 869.9 feet USGS 1929. The NDCT basin and flume is not a seismically qualified structure. However, seismic dam failure is bounded by the hydrologic dam failure.

The potential for on-site flooding due to failure of the NDCT basin and flume under sunny day conditions and the potential for site flooding due to failure of the UHS under PMF, ½ PMF and seismic, and sunny day conditions have been eliminated by inspection.

3.4.4 Conclusions

For the Rock River, the peak water surface elevation due to dam failure is 703.2 feet USGS 1929. The CLB indicates that the upstream and downstream dam failure would create minor flood waves which will dissipate before reaching the site. The reevaluated flood is below the CLB PMF stillwater elevation of 708.3 feet NGVD 1929. The reevaluated flood is also below the CLB combined event water surface elevation, including wind wave activity, of 703.39 feet USGS 1929. Additionally, in the unlikely event that the pumps are rendered inoperable by a flood level in excess of 703.7 feet USGS 1929, the onsite water wells will be powered from their respective Unit 1 ESF buses. These wells will then provide the makeup water for the essential service cooling towers.

The maximum water surface elevation in the vicinity of the power block due to failure of the NDCT under its seismic dam failure scenario is 869.9 feet USGS 1929. The failure of onsite impoundments was not considered in the CLB. However, this is bounded by the CLB LIP elevation of 870.9 feet NGVD 1929.

3.5 Combined Events Flood

NUREG/CR-7046, Appendix H states that the following alternative combinations should be evaluated to determine the highest flood water elevation at the site:

<u>H.1 Floods Caused By Precipitation Events</u>

- Alternative 1 – Combination of:
  - Mean monthly base flow
  - Median soil moisture

- Antecedent 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, cool-season rainfall
  - Waves induced by 2-year wind speed applied along the critical direction

- Alternative 3 – Combination of:
  - Mean monthly base flow
  - 100-year snowpack
  - Cool-season PMP
  - Waves induced by 2-year wind speed applied along the critical direction

<u>H.2 Floods Caused By Seismic Dam Failures</u>

- 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 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
  - 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

Combinations of events that include seiche, surge and tsunami (NUREG/CR-7046, H.4.2 Combinations) were not evaluated because these mechanisms were screened out in Calculation BYR13-FUK-07.

The three alternatives for H.1 combination were examined without waves induced by 2 year wind speed, in Calculation BYR13-FUK-03. The H.1 combinations including wind-wave activity and H.2 combinations for Rock River are analyzed in Calculation BYR13-FUK-09, Combined Events Analysis (Fukushima), (Exelon 2014i). The assumptions associated with Combined Events analysis are listed in Calculation BYR13-FUK-09.

3.5.1 Inputs

The inputs for the analysis are described below.

3.5.1.1 <u>Probable Maximum Water Surface Elevation for Rock River</u>

The hydrologic dam failure stillwater elevation of 703.2 feet USGS 1929 from Calculation BYR13-FUK-04, Byron Nuclear Generating Station: Dam Failures Analysis (Fukushima)" (Exelon, 2014d) is conservatively used in this analysis.

3.5.1.2 2-Year Wind Speed

The 2-year return period wind speed is calculated by applying the Gumbel Distribution to the fastest 2-minute wind speed data recorded at the National Climatic Data Center (NCDC) Station GHCND: USW00094822:

3.5.1.3 Fetch Length

The fetch length is measured using the digital elevation model for Ogle County (Illinois State Geologic Survey, 2013). Five wave runup cross sections were established at 10 degree intervals to capture the range of conservative fetch lengths at the Byron river screen house. Two additional fetches were established in the direction of Byron to determine if water could enter the site itself.

3.5.2 Methodology

The HHA approach described in NUREG/CR-7046 (NRC, 2011) was used for the evaluation of the effects of the combined-effects floods on the Rock River at Byron. The criteria for combined-effects floods are provided in NUREG/CR-7046, Appendix H.

Combinations of events that include seiche, surge and tsunami were not evaluated because these mechanisms were screened out in Calculation No. BYR13-FUK-07, Byron Nuclear Generating Station: Probable Maximum Surge, Seiche and Tsunami (Fukushima), (Exelon, 2014g).

3.5.2.1 2-Year Wind Speed

The 2-year annual recurrence interval wind speed was required for the coincident wind wave calculations as part of the combined-effects flood analysis per NUREG/CR-7046 (NRC, 2011). The fastest 10 meter altitude, 2 minute duration wind speed recorded at nearby National Climatic Data Center (NCDC) Global Historical Climatology Network-Daily (GHCND) stations was used. The Greater Rockford Airport, IL gage (GHCND USW00094822) was selected because it reported the fastest 2 minute duration wind speed in the vicinity of Byron. The 2 year return period wind speed was determined using the Gumbel Distribution. The 2-year return period, 2-minute wind speed was calculated to be 46 mph using the Gumbel Distribution.

3.5.2.2 Wind Speed Adjustments

The 2 year wind speed is adjusted following the guidance outlined in USACE Coastal Engineering Manual (USACE 2008) for level (measurement height), duration, and wind speed over water versus wind speed over land.

3.5.2.3 Wave Height and Period

The wave prediction application of the CEDAS-ACES Version 4.03 was used to determine the deepwater significant wave height and period. The wind duration sensitivity analysis shows that the sixty-six minute duration provided the most conservative deepwater wave heights for the river screen house and that twenty-two minute duration provided the most conservative wave height for Byron.

3.5.2.4 Wave Runup

The "Runup and overtopping on impermeable slopes" and the "Irregular wave runup on beaches" applications of the CEDAS-ACES Version 4.03 software program are used to develop wave runup based on empirical runup equations developed by Ahrens and Titus (USACE 1992).

3.5.3 Results

The wind generated wave runup resulting from a 2 year wind speed applied along the critical direction (per Appendix H of NUREG/CR-7046) on the Rock River at the Byron river screen house was calculated to be 4.2 feet. The critical combination is the Combination H.1, Alternative 2, the probable maximum snowpack combined with the 100-year, cool-season rainfall.

The probable maximum water elevation resulting from combined events on the Rock River at the Byron river screen house was calculated as the sum of the bounding flood (stillwater elevation 703.2 feet USGS 1929) and the 2 year wind wave runup (4.2 feet), which results in a water surface elevation of 707.4 feet USGS 1929.

3.5.4 Conclusions

The results of the dam failure calculation (ENERCON 2014d), indicate that floods caused by seismic dam failures (NUREG/CR-7046, Appendix H.2) are bounded by the PMF with coincident hydrologic dam failure on the Rock River at Byron and further evaluation of seismic dam failures is not necessary. Byron flood protection features do not require manual or operator actions in order for the feature to complete its intended flood protection function (Exelon 2012). Therefore, the warning time associated with seismic dam failure is not relevant.

The maximum water elevation, including runup at the river screen house (707.4 feet USGS 1929) exceeds the CLB water surface elevation with wind wave of 703.39 feet USGS 1929. However, the reevaluated water surface elevation is lower than the CLB PMF still water elevation of 708.3 feet USGS 1929. Additionally, in the unlikely event that the pumps are rendered inoperable by a flood level in excess of 703.7 feet USGS 1929, the onsite water wells will be powered from their respective Unit 1 ESF buses. These wells will then provide the makeup water for the essential service cooling towers.

3.6 Ice-Induced Flooding

Ice-induced flooding is analyzed in Calculation BYR13-FUK-05, Ice Induced Flooding (Fukushima) (Exelon, 2014e).

The HHA approach described in NUREG/CR-7046 (NRC 2011) was used for the calculation. As per NUREG/CR-7046, Appendix G, ice-induced events may lead to flooding at a site due to two scenarios:

1. Ice jams or dams that form upstream of a site that collapse, causing a flood wave;
2. Ice jams or dams that form downstream of a site that result in backwater flooding.

An evaluation of upstream and downstream structures and historical ice jam events near Byron was performed to demonstrate that ice jam flooding is bounded by the PMF at Byron. The assumptions associated with Ice-Induced flooding are listed in Calculation BYR13-FUK-05 (Exelon 2014e).

3.6.1 Inputs

The inputs for the analysis are described below.

3.6.1.1 Historical Ice Jam Data

The USACE ice jam database (USACE 2013a) was queried to obtain the record of ice jams that have occurred on Rock River. The period of record available was from 1915 through 2013.

3.6.1.2 Hydraulic Model for Rock River

The hydraulic model for Rock River is developed in Calculation BYR13-FUK-03, Probable Maximum Flood Analysis - Hydraulics (Fukushima) (Exelon 2014c). This model is used to determine maximum water surface elevations at Byron due to the PMF. The same hydraulic model is used to determine the water level at Byron due to collapse of an upstream ice jam, causing a flooding wave, or formation of a downstream ice jam, resulting in backwater flooding.

3.6.1.3 Mean Monthly Flow

The highest monthly mean flow from Calculation BYR13-FUK-02, Byron Nuclear Generating Station: Probable Maximum Flood Analysis- Hydrology (Fukushima) (Exelon 2014b) is used to determine flow inside the channel during both ice jam analyses. The highest monthly mean flow is the mean April flow recorded at USGS gage 05440700 Rock River at Byron, IL.

3.6.2 Methodology

3.6.2.1 Identify Historical Ice Jams

The USACE Cold Regions Research and Engineering Laboratory ice jam database (USACE 2013a) was queried to obtain the record of ice jams that have occurred on Rock River. The period of record available was from February 1915 through July 2013. The ice jam resulting in the largest river stage was selected from each of the following stream gages: USGS 05446500 Rock River near Joslin, IL, USGS 05440700 Rock River at Byron, IL, USGS 05443500 Rock River at Como, IL, and USGS 05437500 Rock River at Rockton, IL.

3.6.2.2 Calculate Ice Jam Thickness of Historic Ice Jam

The ice jam database presents the historic ice jam data as a combination of qualitative accounts and quantitative river stages and discharges resulting from ice-induced floods. The average discharge on the date of the historical ice jam was identified for the largest ice jam stage recorded in the historic ice jam database. The ice free gage height was determined by inspection of the USGS rating curve for the gage where the ice jam occurred.

The historical ice jam thickness was calculated by subtracting the ice free gage height from the historical river stage. Because the ice jam thickness was greater than all other historic ice jam stages, and therefore greater than all other ice jam thicknesses, no further ice thickness calculations were required. This is considered the controlling ice jam.
Ice Jam Thickness = historical river stage – ice free gage height

3.6.2.3 Calculate Water Surface Elevation from Upstream Ice Jam

Flooding at Byron due to an upstream ice dam breach was calculated. The County Route 2/U.S. State Highway 72 Bridge is the first significant structure upstream of Byron. The water surface elevation at the bridge due to the formation and subsequent failure of an upstream ice jam was calculated as the mean April water surface elevation at the ice jam location plus the height of the controlling ice jam.

The resultant water surface elevation due to the upstream ice jam elevation was directly translated to the site. This is a conservative assumption for modeling the effects of rapid failure of an upstream ice jam on the site.

3.6.2.4 Calculate Water Surface Elevation from Downstream Ice Jam

The water surface elevation at Byron due to backwater conditions resulting from a downstream ice jam was calculated. The Oregon Dam is a run of the river dam and is the first significant structure downstream of Byron. The elevation of the ice dam at Oregon Dam was calculated as the elevation of the spillway plus the thickness of the largest ice jam of record.

The water surface elevation at Byron due to ice jam backwater was calculated using the HEC-RAS model noted in Section 6.3. The geometry of the Oregon Dam in the HEC-RAS model was modified to represent an ice jam extending the full width of the cross section at the ice dam elevation. The mean April flow was then routed through the modified model to establish the water surface elevation at the Byron screen house.

3.6.3 Results

The largest historic ice induced flood was determined to be the result of the January 14, 2009 ice jam occurring near Joslin, Illinois. A maximum river stage of 30.0 feet was recorded with a calculated ice jam thickness of 20.4 feet.

The calculated water surface elevation due to the breaching of an upstream ice dam is conservatively calculated as 696.1 feet USGS 1929. The water surface elevation at the Byron river screen house caused by a downstream ice jam was calculated to be 692.2 feet USGS 1929.

3.6.4 Conclusions

In accordance with NUREG/CR-7046 (NRC 2011), ice-induced flooding was analyzed to calculate the resulting water surface elevation at Byron. Ice-induced flooding is not specifically included as a mechanism to be combined with other extreme events (Appendix H of NUREG/CR-7046) (NRC 2011).

The calculated water surface elevation due to the breaching of an upstream ice dam is conservatively calculated as 696.1 feet USGS 1929. This elevation is 3.1 feet below the PMF water surface elevation, which indicates that upstream ice-induced flooding is bounded by the PMF on the Rock River. The water surface elevation at the Byron river screen house caused by a downstream ice jam was calculated to be 692.2 feet USGS 1929, which is 7.0 feet below the PMF water surface elevation. Downstream ice induced flooding is bounded by the PMF on the Rock River.

The water surface elevation at the Byron river screen house caused by a downstream or an upstream ice jam was calculated to be below the limiting height of the essential service water pumps at elevation 703.7 feet USGS 1929.

3.7 Channel Migration

The channel migration possibility is analyzed in the Calculation BYR13-FUK-08, Channel Diversion (Fukushima) (Exelon 2014h).

NUREG/CR-7046 (NRC, 2011) notes that natural channels may migrate or divert either away from or toward the site. There are no well-established predictive models for channel diversion. Historical records and hydrogeomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander toward the site. The assumptions associated with Channel Migration are listed in Calculation BYR13-FUK-08.

3.7.1 Inputs

The inputs for the analysis are described below.

3.7.1.1 Historical Topographic Map and Aerial Photos

The 1939 and 2011 Aerial Photographs, as well as the 1983 and 1924 U.S. Geological Survey (USGS) Topographic maps of Byron are examined to illustrate general continuity of the river shore for that period.

3.7.2 Methodology

The evaluation approach is summarized in general terms below:

- Review historical records, geologic data, and Byron design information to assess whether Rock River exhibits the tendency to migrate toward the site.
- Review historic landslides in the vicinity of the site and review landslide potential for causing the channel to migrate toward the site.
- Evaluate present-day channel stabilization and maintenance measures in place to mitigate channel migration of Rock River.

3.7.3 Results

A review of historical data and site information indicates that the Rock River has not exhibited a tendency to meander toward the Byron site. Byron is approximately 2.0 miles from the river and is approximately 196 feet higher than the typical river elevation. Historic and site information also indicate that the river channel in the vicinity of the river screen house has been generally stable. Underlying geology consists of relatively shallow bedrock at the plant site. Channel diversion impacts at Byron are not anticipated to occur as a result of landslide because of the low landslide potential in the area, the relatively flat surrounding floodplain, and distance of the site to the river. The Rock River is not a maintained waterway; however, a channel maintenance system is in place to maintain the local channel configuration near the river screen house. Local bank stabilization features (sheet pile and riprap) are in place at the river screen house to protect against erosion and channel migration in the immediate vicinity.

3.7.4 Conclusions

Channel migration is not considered to be a potential contributor to flooding at Byron.

3.8 Error/Uncertainty

The Error/Uncertainty is analyzed in the Calculation BYR13-FUK-11, Error/Uncertainty Analysis (Fukushima), (Exelon 2014k). The analysis evaluates the errors and uncertainties associated with the Effects of LIP calculation. The LIP flood mechanism was selected for error/uncertainty analysis because it has the greatest potential to adversely affect safety-related equipment under the current flood re-evaluation process.

Some degree of error/uncertainty is unavoidable in the development of the deterministic hydrologic and hydraulic models used in the Byron flood reevaluation. The LIP flood mechanism was selected for error/uncertainty analysis because it produces the highest water surface elevation of the mechanisms studied under the current flood reevaluation process. However, none of the peak flood elevations calculated for the LIP (or any other flood mechanism) exceeds the current licensing basis. The assumptions associated with error/uncertainty analysis are listed in Calculation BYR13-FUK-11.

The error and/or uncertainties accompanying the effects of LIP analysis are:

- Error/uncertainty associated with the selected Manning's roughness coefficients
- Error/uncertainty associated with the ground surface topography

LIP error/uncertainty analysis was completed by performing a sensitivity analysis using the FLO-2D model developed as part of the LIP Calculation.

3.8.1 Inputs

3.8.1.1 Two-Dimensional Hydrodynamic Model

The FLO-2D model developed as part of the LIP Calculation (Exelon 2014f) was used.

3.8.1.2 LIP PMP Elevation:

Maximum LIP water surface elevation in the vicinity of the main building at Byron of 870.80 feet USGS 1929 (Exelon 2014f).

3.8.1.3 Range of Manning's n Parameter:

A Manning's roughness coefficient was estimated by correlating Manning's roughness coefficient specifically intended for use with two dimensional models with land cover selected from visual assessment of the aerial topography and site observations. Range of Manning's n parameter for paved areas is 0.02 to 0.05, shrubs are 0.30 to 0.40, and grass cover is 0.2 to 0.4 based on the FLO-2D Reference Manual.

3.8.2 Methodology

A sensitivity analysis was performed for the Manning's n parameter used in the FLO-2D model that was used in the LIP Calculation (Exelon 2014f).

The FLO-2D model included in the LIP Calculation (Exelon 2014f) generally utilized the mid-range values of the recommended ranges of Manning's n above. Specifically,

- 0.035 for the paved areas,
- 0.07 for open ground, and
- 0.30 for shrubs and grass areas.

The middle range of Manning's roughness coefficients used in the two dimensional models are generally the upper end of the range of values used in one dimensional models (Chow 1959). Note that the surface types "shrubs" and 'grass cover" were grouped together into a single surface type named "shrubs and grass areas". This surface type uses the middle end of the range values for grass cover.

The accuracy of the topographic survey was assessed by reviewing the LiDAR and Vertical Accuracy Report (Quantum Spatial 2014) produced in support of the site survey. The document includes a report of the contour interval accuracy as well as the RMSE vertical accuracy for select points.

A sensitivity analysis was performed by changing the Manning's n values included in the LIP FLO-2D model to reflect the low end range values, including:

- 0.02 for paved areas,
- 0.04 for unpaved areas, and
- 0.30 for shrub and grass areas. This surface type uses the middle end of the range of Manning's n values for grass cover of 0.30.

3.8.3 Results

3.8.3.1 Error/Uncertainty in Ground Surface Topography

The elevation data was prepared by photogrammetric methods using aerial photography. The topographic survey for Byron complies with National Map Accuracy Standards (NMAS) for a Horizontal Scale of 1"=100', a contour interval of two feet accuracy, and +/- 0.125 feet Root Mean Square Error (RMSE) vertical accuracy for spot elevations at well-defined points.

Uncertainty regarding onsite flood elevations is generally limited to the level of accuracy of the site survey. The nature of two dimensional flow models is such that the impact of potential inaccuracy in the elevation of any single grid element is generally mitigated by the surrounding grid elements.

3.8.3.2 Error/Uncertainty due to Manning's Roughness Coefficients

The sensitivity analysis indicates that the FLO-2D model is relatively insensitive to the Manning's n values.

3.8.4 Conclusions

The sensitivity analysis indicates that the FLO-2D model is relatively insensitive to the Manning's n values. The results are bounded by the CLB LIP values. For the two dimensional flow model, water surface uncertainty due to survey error of +/- 0.125 feet could occur.

3.9 Associated Effects

The associated effects for the flooding due to the LIP and Rock River PMF flooding due to the dam failure (both hydrologic and seismic) are determined in Calculation BYR13-FUK-10, Associated Effects Analysis (Fukushima) (Exelon 2014j). The assumptions associated with Associated Effects analysis are listed in Calculation BYR13-FUK-10.

The "Guidance for Performing the Integrated Assessment for External Flooding Guidance" (NRC, 2012) defines "flood height and associated effects" as follows:

"The maximum stillwater surface elevation plus the following factors:

- wind waves and run-up effects;
- hydrodynamic loading, including debris;
- effects caused by sediment deposition and erosion;
- concurrent site conditions, including adverse weather conditions;
- groundwater ingress; and
- other pertinent factors."

3.9.1 Inputs

- PMF with hydrologic dam failure maximum water surface elevation (703.2 feet USGS 1929), stage hydrograph, and velocity (1.07 feet per second) at the Byron river screen house from BYR13-FUK-09, Combined Events Analysis (Exelon 2014i).
- ½ PMF with seismic dam failure maximum water surface elevation (694.4 feet USGS 1929) and stage hydrograph, at the Byron river screen house from BYR13-FUK-09, Combined Events Analysis (Exelon 2014i).
- Wind-generated wave runup of 4.2 feet from BYR13-FUK-09, Combined Events Analysis (Exelon 2014i).
- Hydrostatic forces, hydrodynamic forces and maximum velocities during a LIP event from BYR13-FUK-06, Local Intense Precipitation Analysis (Exelon 2014f).

3.9.2 Methodology

Two locations were evaluated for "flood height and associated effects" including:

1) Power block area for effects of LIP.
2) Screen house area for effects of riverine PMF with hydrologic dam failure and effects of ½ PMF with seismic dam failure at the Byron river screen house.

Loading Resulting from LIP

Hydrostatic

Hydrostatic loads are those caused by water above or below the ground surface, free or confined. These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts. Hydrostatic pressure is equal in all directions and always acts perpendicular to the surface on which it is applied. Hydrostatic loads can be subdivided into vertical downward loads, lateral loads or vertical upward loads (uplift or buoyancy). Hydrostatic loads at the Byron main power block and building were derived from the FLO-2D model developed as part of the LIP Calculation (Exelon 2014f).

Hydrodynamic

Water flowing around a building (or structure) imposes loads on the building. Hydrodynamic loads, which are a function of flow velocity and structure geometry, include frontal impact on the upstream face, drag along the sides and suction at the downstream side. Hydrodynamic loads at the Byron main power block and building were derived from the FLO-2D model developed as part of the LIP Calculation (Exelon 2014f).

Hydrodynamic Loading at the Byron River Screen house

Water flowing around a building (or structure) imposes loads on the building. Hydrodynamic loads, which are a function of flow velocity and structure geometry, include frontal impact on the upstream face, drag along the sides and suction at the downstream side. Hydrodynamic loads calculated here used steady-state flow velocities (FEMA 2012).

Wave Impact Loading at the Byron River Screen house

Wave loads are those loads that result from water waves propagating over the water surface and striking a building (or other structure), including: a) waves breaking on any portion of the building; b) wave run-up striking any portion of the building; c) wave-induced drag and inertia forces; and d) wave-induced scour. Loads due to non-breaking waves are calculated as the hydrostatic and hydrodynamic loads. Loads due to broken waves are similar to hydrodynamic loads from flowing or surging water. The forces from breaking waves are the largest and most severe; therefore this load condition was used as the design wave load. The three breaking wave load conditions include: a) waves breaking against submerged small diameter piles; b) waves breaking against submerged walls; and c) wave slam, where the top of the wave strikes against a vertical wall. The term "wave slam" refers to the action of wave crest striking the elevated portion of a structure. Wave slam is only calculated for elevated structures.

Debris Impact Loading

<u>River Screenhouse</u>

Debris impact loads are imposed on a building (or structure) by objects carried by moving water. The loads are influenced by where the building is located in the potential debris stream, specifically if it is:

- immediately adjacent to or downstream from another building,
- downstream from large floatable objects, or
- among closely spaced buildings.

Impact loads at the water surface, for the river screen house, were calculated using the guidelines described in FEMA P-259 (FEMA 2012) and by considering debris weight recommended in ASCE/SEI-7-10 (ASCE 2010). ASCE/SEI-7-10 states that an average debris weight of 1,000 pounds is reasonable. For riverine floodplains, where large woody debris predominates, the debris weight can vary from 1,000 to 2,000 pounds. Also, in riverine areas subject to floating ice, debris weights typically range from 1,000 to 4,000 pounds (Exelon 2014k).

Groundwater Ingress

Safety related SSCs located below grade and/or below the maximum stillwater elevation are subject to flooding due to groundwater ingress. The following steps were used to determine the potential effects of groundwater ingress:

1. The site information, such as available drawings and the Flood Walkdown Reports, were utilized to identify such SSCs.
2. Protective measures against groundwater ingress were also identified.
3. The results of the riverine PMF calculation were used to contrast water elevations with those of safety related structures to evaluate the potential for groundwater ingress (Exelon 2014k).

Sediment Deposition and Erosion

High velocity flood flows may result in scour or erosion.

River Screen house

The velocity at the river screen house was taken from the Dam Failures Calculation (Exelon 2014d) and then compared with typical permissible velocities for riprap.

Main Power block at Byron

Maximum velocities at the power block area were taken from the LIP Calculation (Exelon 2014f) and then compared with typical permissible velocities for selected ground cover materials (Exelon 2014k).

Flood Duration

Flood duration can vary depending on many factors, including rainfall duration and intensity, watershed area and topography, and riverine characteristics.

River Screen house: PMF/Upstream Dam Failures

The stage hydrograph for the ½ PMF and PMF dam failure scenarios were calculated in HEC- RAS as part of the Dam Failure Calculation (Exelon 2014d). These hydrograph stillwater elevations, along with the wave runup height of 4.2 feet, were used to calculate the flood duration at the river screen house above the elevation of the limiting height of the essential service water makeup pumps at 703.7 feet USGS 1929 (Exelon 2014k).

Main Power Block at Byron: LIP

Stage hydrographs were calculated for select locations in the main power block in FLO-2D as part of the LIP Calculation (Exelon 2014f). The hydrograph that resulted in the longest duration of flooding was used to calculate the flood duration above the current licensing basis for on-site flooding mechanisms (870.9 feet USGS 1929).

Warning Time

Certain flood causing mechanisms, such as dam failures, are associated with little warning time.

River Screen house

The stage hydrograph for the ½ PMF and PMF dam failure scenarios were calculated in HEC-RAS as part of the Dam Failure Calculation (Exelon 2014d). These hydrograph stillwater elevations, along with the wave runup height of 4.2 feet, were used to calculate the warning time at the river screen house before flood waters reached the elevation of the limiting height of the essential service water makeup pumps at 703.7 feet USGS 1929 (Exelon 2014k).

Main Power Block at Byron

A local intense precipitation event has no appreciable warning time except those provided by a weather (precipitation) forecast.

3.9.3 Results

The results due the "flood height and associated effects" analysis are summarized below:

Associated Effects at the Main Power Block (Exelon 2014k):

- The maximum hydrostatic force at the main power block at Byron is 646 pounds per foot at the junction of the Radwaste/Service building and the Turbine building.
- The maximum hydrodynamic force at the main power block at Byron is 14.7 pounds per foot; near the northeast corner of the main power block.
- Groundwater ingress is not anticipated to adversely affect safety-related equipment at Byron because the buildings with external walls below grade are designed to be watertight up to the floor ground elevation (one foot above grade) and designed to resist the full hydrostatic head of groundwater.
- The flood does not exceed the CLB LIP elevation. Therefore, flood duration above CLB is considered 0 hours.

Associated Effects at the River Screen House (Exelon 2014k):

- Debris impact loading at the Byron river screen house is 3,424 pounds for ice debris and 1,712 pounds for wood debris.
- The hydrodynamic loading at the Byron river screen house is 3,248 pounds or 1.4 pounds per square foot (psf) acting on the northern face of the screen house.
- The wave impact loading at the Byron river screen house is 851,350 pounds or 307 psf acting on the northern face of the screen house.
- Sediment deposition and erosion is not anticipated to adversely affect the Byron river screen house. Additionally, in the unlikely event that the pumps are rendered inoperable by a flood level in excess of 703.7 feet USGS 1929, the onsite water wells will be powered from their respective Unit 1 ESF buses. These wells will then provide the makeup water for the essential service cooling towers.
- The flood duration associated with potential wave effects at the river screen house is 4.4 days for the PMF with hydrologic dam failure scenario. However, the duration of

the winds used to determine the wave height of 4.2 feet is sixty-six minutes. Stillwater elevations are below the elevation of safety-related equipment in the screen house.

- With wave effects, the warning time is 4.1 days for the PMF with hydrologic dam failure scenario at the river screen house.

3.9.4 Conclusions

The flood height and its associated effects analysis indicates that the hydrostatic force, hydrodynamic loads, groundwater ingress, and sediment transport are not anticipated to adversely affect safety-related equipment at the screen house and the power block area.

4. FLOOD PARAMETERS AND COMPARISON WITH CURRENT DESIGN BASIS

Per the March 12, 2012, 50.54(f) letter (NRC March 2012), Enclosure 2, the following flood-causing mechanisms were considered in the flood hazard reevaluation for Byron.

1. Local Intense Precipitation;
2. Flooding in Streams and Rivers;
3. Dam Breaches and Failures;
4. Storm Surge;
5. Seiche;
6. Tsunami;
7. Ice Induced Flooding; and
8. Channel Migration or Diversion.

Some of these individual mechanisms are incorporated into alternative 'Combined Effect Flood' scenarios per Appendix H of NUREG/CR-7046 (NUREG/CR-7046).

The March 12, 2012, 50.54(f) letter, Enclosure 2, requests the licensee to perform an integrated assessment of the plant's response to the reevaluated hazard if the reevaluated flood hazard is not bounded by the current design basis. This section provides comparisons with the current design basis flood hazard and applicable flood scenario parameters per Section 5.2 of JLD-ISG-2012-05 (NRC ISG-2012-05), including:

1. Flood height and associated effects
   a. Stillwater elevation;
   b. Wind waves and run-up effects;
   c. Hydrodynamic loading, including debris;
   d. Effects caused by sediment deposition and erosion (e.g., flow velocities, scour);
   e. Concurrent site conditions, including adverse weather conditions; and
   f. Groundwater ingress.

2. Flood event duration parameters (per Figure 6, below, of JLD-ISG-2012-05)
   a. Warning time (may include information from relevant forecasting methods (e.g., products from local, regional, or national weather forecasting centers) and ascension time of the flood hydrograph to a point (e.g. intermediate water surface elevations) triggering entry into flood procedures and actions by plant personnel);

b. Period of site preparation (after entry into flood procedures and before flood waters reach site grade);
c. Period of inundation; and
d. Period of recession (when flood waters completely recede from site and plant is in safe and stable state that can be maintained).

3. Plant mode(s) of operation during the flood event duration
4. Other relevant plant-specific factors (e.g. waterborne projectiles)

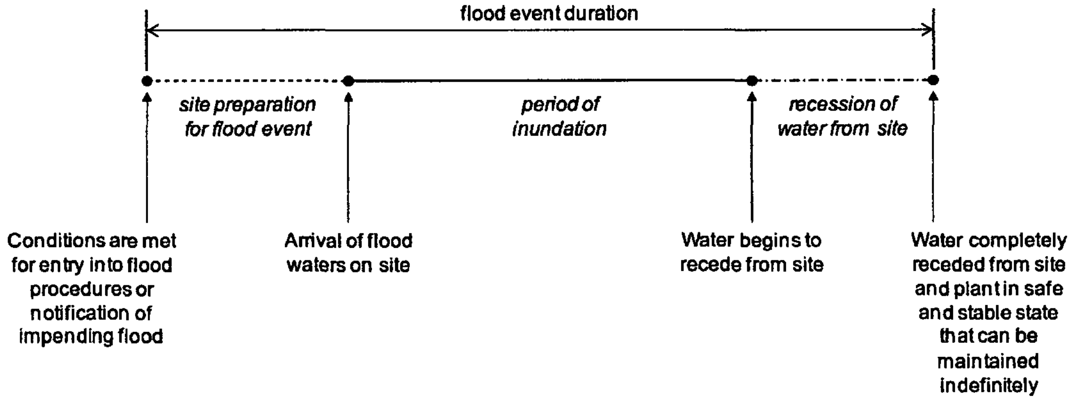

Figure 4.0.1 – Illustration of Flood Event Duration (NRC ISG-2012-05, Figure 6)

Per Section 5.2 of JLD-ISG-2012-05 (NRC ISG-2012-05), flood hazards do not need to be considered individually as part of the integrated assessment. Instead, the integrated assessment should be performed for a set(s) of flood scenario parameters defined based on the results of the flood hazard reevaluations. In some cases, only one controlling flood hazard may exist for a site. In this case, licensees should define the flood scenario parameters based on this controlling flood hazard. However, sites that have a diversity of flood hazards to which the site may be exposed should define multiple sets of flood scenario parameters to capture the different plant effects from the diverse flood parameters associated with applicable hazards. In addition, sites may use different flood protection systems to protect against or mitigate different flood hazards. In such instances, the integrated assessment should define multiple sets of flood scenario parameters. If appropriate, it is acceptable to develop an enveloping scenario (e.g., the maximum water surface elevation and inundation duration with the minimum warning time generated from different hazard scenarios) instead of considering multiple sets of flood scenario parameters as part of the integrated assessment. For simplicity, the licensee may combine these flood parameters to generate a single bounding set of flood scenario parameters for use in the integrated assessment.

For Byron, the following flood-causing mechanisms were either determined to be implausible or completely bounded by other mechanisms:

1. Surge, Seiche and Tsunami;
2. Ice Induced Flooding; and
3. Channel Migration or Diversion.
4. Seismic Dam Failure on Rock River is bounded by Hydrologic Dam Failure. The flood protection features do not require manual or operator actions. Therefore, warning time associated with sunny day or seismic dam failure is not relevant.
5. Onsite Impoundment failure is bounded by the LIP flood.

Byron was considered potentially exposed to the flood hazards (individual flood-causing mechanisms and/or combined-effects flood scenarios per Appendix H of NUREG/CR-7046) listed below. In some instances, an individual flood-causing mechanism (e.g. 'Flooding in Streams and Rivers') is addressed in one or more of the combined-effect flood scenarios.

1. Local Intense Precipitation
2. Combinations in Section H.1 of NUREG/CR-7046 (Floods Caused by Precipitation Events) for the Rock River

Tables 4.0.1 through 4.0.3 summarize the parameters for each flood hazard and provide comparisons with the current design basis flood.

Table 4.0.1 – Summary of Licensing Basis and External Flooding Study Parameters

Parameter	Current Licensing Basis	Reevaluation Study
	Value/Methodology	Value/Methodology
Probable Maximum Precipitation		
Methodology	HMR 33, USACE, U.S. Bureau of Reclamation	HMR 51 and HMR 52
Storm Duration	48 hours	72 hours
Cumulative PMP	34.28 (All-Season) 14.70 (Cool-Season)	15.1
Probable Maximum Flood on Rock River		
Nonlinear Basin Response	No	Yes
Hydrologic Model	None. Hand Calculations.	HEC-HMS
Total area	8170 sq.mi.	8,174 sq. mi.
Subbasins	3	4
Wind Wave Activity coincident with PMF on Lake		
Methodology	USACE "Shore Protection Manual of the Corps of Engineers, 1977	USACE Coastal Engineering Manual ANSI/ANS-2.8-1992
Wind speed	40 mph	46 mph
Wave parameters determination	Hand calculation	Hand calculation
Wave runup (ft)	2.77 (significant) and 4.71 (maximum) – Based on Combined Event Flood	4.2 feet
Local Intense Precipitation		
Methodology	HMR 33, U.S. Bureau of Reclamation	Site-specific LIP study
1-Hour LIP (inches)	17.6 (All-Season) 14.70 (Cool-Season)	13.6
Effects Local Intense Precipitation		
Methodology	Rational Method, Weir Flow, HEC-RAS Backwater Step Calculation	Hydrodynamic Modeling using FLO-2D Computer Software
Potential Dam Failure		
Dam Failure Scenarios Considered	Seismic	Hydrologic, Seismic, Sunny Day (On-Site Only)

Table 4.0.2 – Local Intense Precipitation

Flood Scenario Parameter		CLB	Reevaluated	Bounded (B) or Not Bounded (NB)
Flood Level and Associated Effects	1. Max Stillwater Elevation (ft. USGS 1929)	870.90	870.80	B
	2. Max Wave Run-up Elevation (ft. USGS 1929)	N/A	N/A	N/A
	3. Max Hydrodynamic / Debris Loading (lb/ft)	Not Determined	14.7	B
	4. Effects of Sediment Deposition/Erosion	N/A	N/A	B
	5. Concurrent Site Conditions	N/A	N/A	N/A
	6. Effects on Groundwater	N/A	N/A	N/A
Flood Event Duration	7. Warning Time (hours)	N/A	N/A	N/A
	8. Period of Site Preparation (hours)	N/A	N/A	N/A
	9. Period of Inundation (hours)	N/A	N/A	N/A
	10. Period of Recession (hours)	N/A	N/A	N/A
Other	11. Plant Mode of Operations	Any	Any	Any
	12. Other Factors	N/A	N/A	N/A

Additional notes, 'N/A' justifications (why a particular parameter is judged not to affect the site), and explanations regarding the bounded/non-bounded determination.

1. The reevaluated flood elevation is bounded by the current design basis.
2. Consideration of wind-wave action for the LIP event is not explicitly required by NUREG/CR-7046 and is judged to be a negligible associated effect because of limited fetch lengths and flow depths.
3. The hydrodynamic and hydrostatic loads are determined as force per unit length of structure (lb/ft). To determine the force for the entire structure the loads need to be multiplied by the structure length. The hydrodynamic and hydrostatic loads are bounded by the CLB. The debris load for the LIP event is assumed to be negligible due to the absence of heavy objects at the plant site and due to low flow velocity, the factors combination of which could lead to a hazard due to debris load.
4. The flow velocities due to the LIP event are determined to be below the suggested velocities (USACE 1984) at the plant area. Therefore, erosion is considered bounded by the current design basis.
5. High winds could be generated concurrent to a LIP event. However, manual actions are not required to protect the plant from LIP flooding so this concurrent condition is not applicable.
6. The majority of the plant area is paved or gravel and results in minimal infiltration, if any. Additionally, the event is a short-duration (1-hour precipitation) which limits the amount of soil infiltration.
7. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the LIP flood.
8. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the LIP flood.
9. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the LIP flood.
10. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the LIP flood.
11. The reevaluated peak flood elevation is bounded by the current design basis. Current plant operations and procedures will still govern.
12. There are no other factors, including waterborne projectiles, applicable to the LIP flood.

Table 4.0.3 – Combinations in Section H.1 of NUREG/CR-7046 (w/ hydrologic dam failure) – Screen House at Rock River

	Flood Scenario Parameter	CLB	Reevaluated	Bounded (B) or Not Bounded (NB)
Flood Level and Associated Effects	1. Max Stillwater Elevation (ft. USGS 1929)	708.3	699.20	B
	2. Max Wave Run-up Elevation (ft. USGS 1929)	N/A	707.4	B
	3. Max Hydrodynamic/Debris Loading	44,820 psf	307 psf	B
	4. Effects of Sediment Deposition/Erosion	N/A	See note	B
	5. Concurrent Site Conditions	Not Determined	See note	N/A
	6. Effects on Groundwater	N/A	See note	B
Flood Event Duration	7. Warning Time (days)	N/A	N/A	N/A
	8. Period of Site Preparation (hours)	N/A	N/A	N/A
	9. Period of Inundation (days)	N/A	N/A	N/A
	10. Period of Recession (hours)	N/A	N/A	N/A
Other	11. Plant Mode of Operations	Any	Any	Any
	12. Other Factors	N/A	N/A	N/A

Additional notes, 'N/A' justifications (why a particular parameter is judged not to affect the site), and explanations regarding the bounded/non-bounded determination.

1. .Reevaluated water surface elevation is bounded by the CLB PMF elevation of 708.3 ft NGVD29. Additionally, as indicated in the UFSAR, the onsite seismically qualified wells provide an alternate source of makeup water if pumps at the screen house are inoperable.
2. UFSAR utilizes lesser flood than the PMF for wind wave activity. However, reevaluated water surface elevation with wind-wave activity is bounded by the CLB PMF elevation of 708.3 ft NGVD29.
3. The hydrodynamic and debris loads are bounded by the CLB. Additionally, in an unlikely event that the river screen house is rendered inoperable, the onsite seismically qualified wells will provide the makeup water.
4. The peak velocity in the left overbank during the PMF with dam failure is 1.1 ft/sec, which is much less than the permissible velocity for 45 inch diameter riprap, protecting the screen house from scour. Additionally, in an unlikely event that the river screen house is rendered inoperable, the onsite seismically qualified wells will provide the makeup water.
5. High winds could be generated concurrent to a PMF. However, manual actions are not required to protect the screen house from river PMF so this concurrent condition is not applicable.
6. All critical equipment at the river screen house is elevated and therefore not subject to effects associated with groundwater ingress. The lowest floor elevation in the plant is also above the PMF. Stillwater elevation so effects of the PMF on groundwater surcharge will not affect the plant.
7. SSC's important to safety are currently protected by means of permanent/passive measure. Therefore, flood event duration parameters are not applicable to the Rock River PMF.
8. SSC's important to safety are currently protected by means of permanent/passive measure. Therefore, flood event duration parameters are not applicable to the Rock River PMF.
9. SSC's important to safety are currently protected by means of permanent/passive measure. Therefore, flood event duration parameters are not applicable to the Rock River PMF.
10. SSC's important to safety are currently protected by means of permanent/passive measure. Therefore, flood event duration parameters are not applicable to the Rock River PMF.
11. The reevaluated peak flood elevation is bounded by the current design basis. Current plant operations and procedures will still govern.
12. There are no other factors, including waterborne projectiles, applicable to the LIP flood.

5. REFERENCES

(ASCE 2010) American Society of Civil Engineers (ASCE), "Minimum Design Loads for Buildings and Other Structures", ASCE/SEI 7-10, 2010

(ANSI/ANS-2.8-1992) American Nuclear Society, American National Standard for Determining Design Basis Flooding at Power Reactor Sites, Prepared by the American Nuclear Society Standards Committee Working Group ANS-2.8, La Grange Park, Illinois, 1992

(Chow 1959) Chow, Ven Te, Open-Channel Hydraulics, McGraw-Hill Book Company, New York, 1959

(ENERCON 2013), ENERCON, LiDAR Survey for Byron, October 2013

(ESRI 2013), ESRI, World Imagery, modified: November 2013. Accessed: December 2013

(Exelon 1997a) Exelon, Essential Service Cooling Tower Section 1-1, S-249, April 1997.

(Exelon 1997b) Exelon, Essential Service Cooling Tower Key Plan & Details, S-255, April 1997

(Exelon 2008) Essential Service Cooling Tower Sections & Details, S-250, August 2008

(Exelon 2009) Exelon, Byron Operating Procedure, SX-12A1, Revision 5, October 15, 2009

(Exelon 2012) Exelon, "Flooding Walkdown Report, NRC Docket No. 50-454/50-455", November 2012

(Exelon 2014a) Exelon, Calculation BYR13-FUK-01, Probable Maximum Precipitation Analysis (Fukushima), Revision 0

(Exelon 2014b) Exelon, Calculation BYR13-FUK-02, Byron Nuclear Generating Station: Probable Maximum Flood Analysis- Hydrology (Fukushima), Revision 0

(Exelon 2014c) Exelon, Calculation BYR13-FUK-03, Probable Maximum Flood Analysis - Hydraulics (Fukushima), Revision 0

(Exelon 2014d) Exelon, Calculation BYR13-FUK-04, Dam Failure Analysis (Fukushima), Revision 0

(Exelon 2014e) Exelon, Calculation BYR13-FUK-05, Ice Induced Flooding (Fukushima), Revision 0

(Exelon 2014f) Exelon, Calculation BYR13-FUK-06, Local Intense Precipitation Analysis (Fukushima), Revision 0

(Exelon 2014g) Exelon, Calculation BYR13-FUK-07, Probable Maximum Surge, Seiche, and Tsunami (Fukushima), Revision 0

(Exelon 2014h) Exelon, Calculation BYR13-FUK-08, Byron Nuclear Generating Station: Channel Diversion (Fukushima), Revision 0

(Exelon 2014i) Exelon, Calculation BYR13-FUK-09, Combined Events Analysis (Fukushima), Revision 0

(Exelon 2014j) Exelon, Calculation BYR13-FUK-10, Associated Effects Analysis (Fukushima), Revision 0

(Exelon 2014k) Exelon, Calculation BYR13-FUK-11, Error/Uncertainty Analysis (Fukushima), Revision 0

(Exelon 2014l) Exelon, Calculation L-003859, Beyond Design Basis Site-Specific Local Intense Precipitation Analysis (Fukushima), Revision 0

(FEMA 2010a), Federal Emergency Management Agency, Flood Insurance Study – Lee County, Illinois, April 2010

(FEMA 2010b), Federal Emergency Management Agency, Flood Insurance Study – Ogle County, Illinois, December 2010

(FEMA 2012) Federal Emergency Management Agency, "Engineering Principles and Practices for retrofitting Flood-Prone Residential Structures", FEMA-P-259, January 2012

(FERC 1993) Federal Energy Regulatory Commission, Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 2 Selecting and Accommodating Inflow Design Floods, October 1993

(FLO-2D) FLO-2D Software, Inc., FLO-2D Pro Computer Software, 2013

(Froehlich 1995) Froehlich, D.C., Peak Outflow from Breached Embankment Dam., Journal of Water Resources Planning and Management, vol. 121, no. 1, p. 90-97., 1995

(Huff 1992) Huff, F.A., and Angel, J.R., Bulletin 71, Rainfall Frequency Atlas of the Midwest, Midwest Climate Center and Illinois State Water Survey, 1992

(HMR No. 33) U.S. Department of Commerce, Weather Bureau, U.S. Department of the Army, Corps of Engineers, Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridiain For Areas From 10 to 1000 square miles and durations of 6, 12, 24 and 48 hours, Hydrometeorological Report No. 33, Washington, D.C., April 1956

(HMR No.51) U.S. Department of Commerce, National Oceanic and Atmospheric Administration, U.S. Department of the Army, Corps of Engineers, Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No.51, Washington, D.C., June 1978

(HMR No.52) U.S. Department of Commerce, National Oceanic and Atmospheric Administration, U.S. Department of the Army, Corps of Engineers, Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, NOAA Hydrometeorological Report No.52, Washington, D.C., August, 1982

(HMR No.53) U.S. Department of Commerce, National Oceanic and Atmospheric Administration, U.S. Nuclear Regulatory Commission (NRC), Hydrometeorological Report No.53, Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, April 1980

(NOAA 2013a) National Oceanic and Atmospheric Administration (NOAA), Tsunami Data and Information website, National Geophysical Data Center, Natural Hazards, Available at http://www.ngdc.noaa.gov/hazard/tsu.shtml, Accessed August 2013

(NOAA 2013b) National Oceanic and Atmospheric Administration (NOAA), Hydrometeorological Design Studies Center, Precipitation Frequency Data Server (PFDS), Available at http://hdsc.nws.noaa.gov/hdsc/pfds, Accessed September 2013

(NOAA 2013c) National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center, Snow Climatology Data, http://www.ncdc.noaa.gov/ussc/index.jsp, Accessed February 2013

(NOAA 2013d) National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center (NCDC), Surface Data, Hourly Global, Available at http://cdo.ncdc.noaa.gov/pls/plclimprod/poemain.cdobystn?dataset=DS3505&StnList=72534014819, Accessed November, 2013

(NOAA 2013e) National Oceanic and Atmospheric Administration (NOAA), VERTCON- North American Vertical Datum Convesion, Available at http://www.ngs.noaa.gov/cgi-bin/VERTCON/vert_con.prl, Accessed October 2013

(NRC March 2012) U.S. Nuclear Regulatory Commission, Letter to Licensees, 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, March 12, 2012

(NRC RG 1.59) U.S. Nuclear Regulatory Commission, Regulatory Guide 1.59, Rev. 2, Design Basis Flood for Nuclear Power Plants, Washington, D.C., 1977

(NRCS 1985) Natural Resources Conservation Service (NRCS), Simplified Dam Breach Routing Procedure, Technical Release No. 66. September 30 1985.

(NRCS 2013) Natural Resources Conservation Service (NRCS), U.S General Soil Map (STATSGO) Database (http://soildatamart.nrcs.usda.gov/USDGSM.aspx) Accessed July 2013,

(NUREG-0800) U.S. Nuclear Regulatory Commission, NUREG-0800, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition – Site Characteristics and Site Parameters (Chapter 2), ML070400364, March 2007

(NUREG/CR-7046) U.S. Nuclear Regulatory Commission, NUREG/CR-7046, PNNL-20091, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, ML11321A195, November 2011

(NUREG/CR-6966) U.S. Nuclear Regulatory Commission, NUREG/CR-6966, Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America, Springfield, VA, National Technical Information Service, March 2009

(NRC ISG-2012-06) U.S. Nuclear Regulatory Commission, NRC JLD-ISG-2012-06, Guidance for Performing a Tsunami, Surge and Seiche Flooding Safety Analysis, Japan Lessons-Learned Project Directorate Interim Staff Guidance, Revision 0, June 2012

(NRC ISG-2013-01) U.S. Nuclear Regulatory Commission, NRC JLD-ISG-2013-01, Guidance For Assessment of Flooding Hazards Due to Dam Failure, Japan Lessons-Learned Project Directorate Interim Staff Guidance, Revision 0, July 2013

(Sargent & Lundy 1987) Sargent & Lundy, "Circulating Water Pump House Plan- Concrete Flume Sections & Details", S-379-E, July 1987

(UFSAR) Byron Power Station, Updated Final Safety Analysis Report (UFSAR), Revision 14

(USACE 1992) U.S. Army Corps of Engineers, Leenknecht, D., Szuwalski, A., "Automated Coastal Engineering System, Technical Reference",Version 1.07, September 1992.

(USACE 1998) U.S. Army Corps of Engineers, EM 1110-2-1406, Engineering and Design, Runoff from Snowmelt, Washington, DC 20314-1000, March 31, 1998

(USACE 2008) U.S. Army Corps of Engineers, Coastal Engineering Manual, EM 1110-2-1100 (Part II), August 1, 2008 (Change 2)

(USACE 2013a) U.S. Army Corps of Engineers, Ice Engineering Research Group, Cold Regions Research and Engineering Laboratory, available at http://icejams.crrel.usace.army.mil/, Accessed July 26, 2013

(USACE 2013b) U.S Army Corps of Engineers, National Inventory of Dams, available at http://geo.usace.army.mil/pgis/f?p=397:1:250834646188401::NO, Accessed November 19 2013

(USBR 1982) U.S. Bureau of Reclamation, Guidelines for defining inundated areas downstream from Bureau of Reclamation dams.", Reclamation Planning Instruction No. 82-11, June 15, 1982

(USGS 1966), "Effects of the March 1964 Alaska Earthquake on the Hydrology of South Central Alaska", United States Geologic Survey, 1964

(USGS 2013a) U.S. Geological Survey (USGS), National Elevation Dataset, available at http://viewer.nationalmap.gov/viewer/, Accessed: July 2013

(USGS 2013b) U.S. Geological Survey (USGS), Illinois Earthquake History, available at http://earthquake.usgs.gov/earthquakes/states/illinois/history.php, Accessed: November 2013

(Quantum Spatial 2014), "LiDAR and Vertical Accuracy Report – Byron Plant LiDAR Project," Quantum Spatial, January 15, 2014.

Enclosure 2

CD-R labeled:

Byron Nuclear Generating Station
Pertinent Site Data