Enclosure 1

Clinton Power Station
Flood Hazard Reevaluation Report
Revision 0

(63 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

Clinton Power Station

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Revision 0
Submitted Date: February 25, 2014

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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
USAR	Updated 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 Hazard Analysis
2. [NTTF] Recommendation 2.1: Flooding Reevaluation
3. [NTTF] Recommendation 2.3: Seismic Walkdown
4. [NTTF] Recommendation 2.3: Flooding Walkdown
5. [NTTF] Recommendation 9.3: Emergency Preparedness
6. Licensees and Holders of Construction Permits – Contact Information for Licensees

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 Clinton Power Station (CPS), 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 CPS, 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 the NRC staff's two phase process to implement Recommendation 2.1, and will be used to identify potential 'vulnerabilities' (See definition below).

For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard with the hazard evaluation.
Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them. The scope of the integrated assessment report will

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

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

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 Clinton Power Station is located 6 miles east of the city of Clinton, DeWitt County in central Illinois.

The cooling lake was formed by constructing an earth dam across Salt Creek 1,200 feet downstream from its confluence with North Fork and 3,300 feet upstream from Illinois State Route 10. The location of the dam is approximately 4 miles east of Clinton. The Salt Creek and North Fork fingers of the U-shaped lake extend 14 miles and 8 miles, respectively, upstream from the dam. The drainage area of the lake is 296 mi^2. The surface area of the lake is 4,895 acres (7.65 mi^2 - 2.6% of the drainage area) and the storage capacity is 74,200 acre-feet at a normal pool elevation of 690 feet mean sea level datum, USGS, 1929 adjustment (MSL).

CPS is located between the two fingers of the lake with a station grade elevation of 736 feet and plant floor elevation of 737 feet MSL (USAR). The present-day general location of the site is presented in Figure 2.1.1. The detailed site layout is presented in Figure 2.1.2.

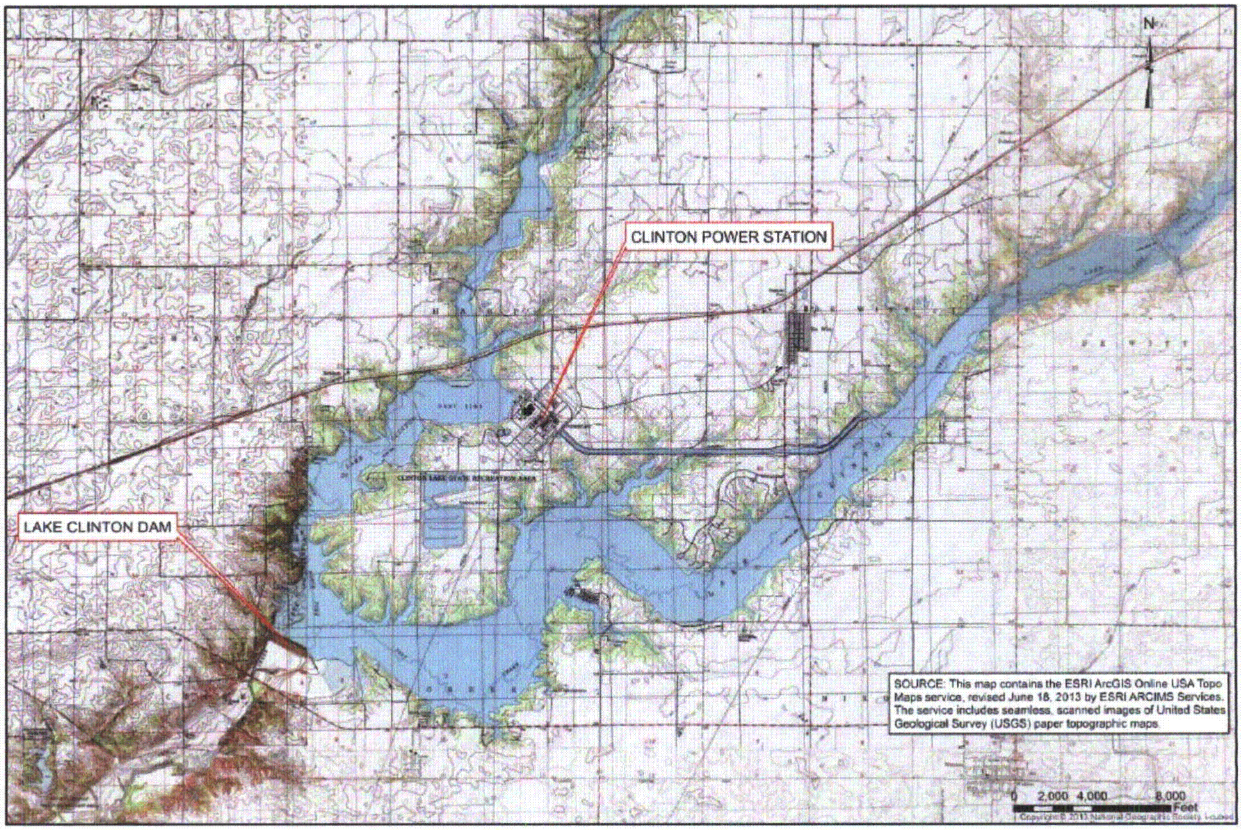

Figure 2.1.1 – Present-Day General Site Map

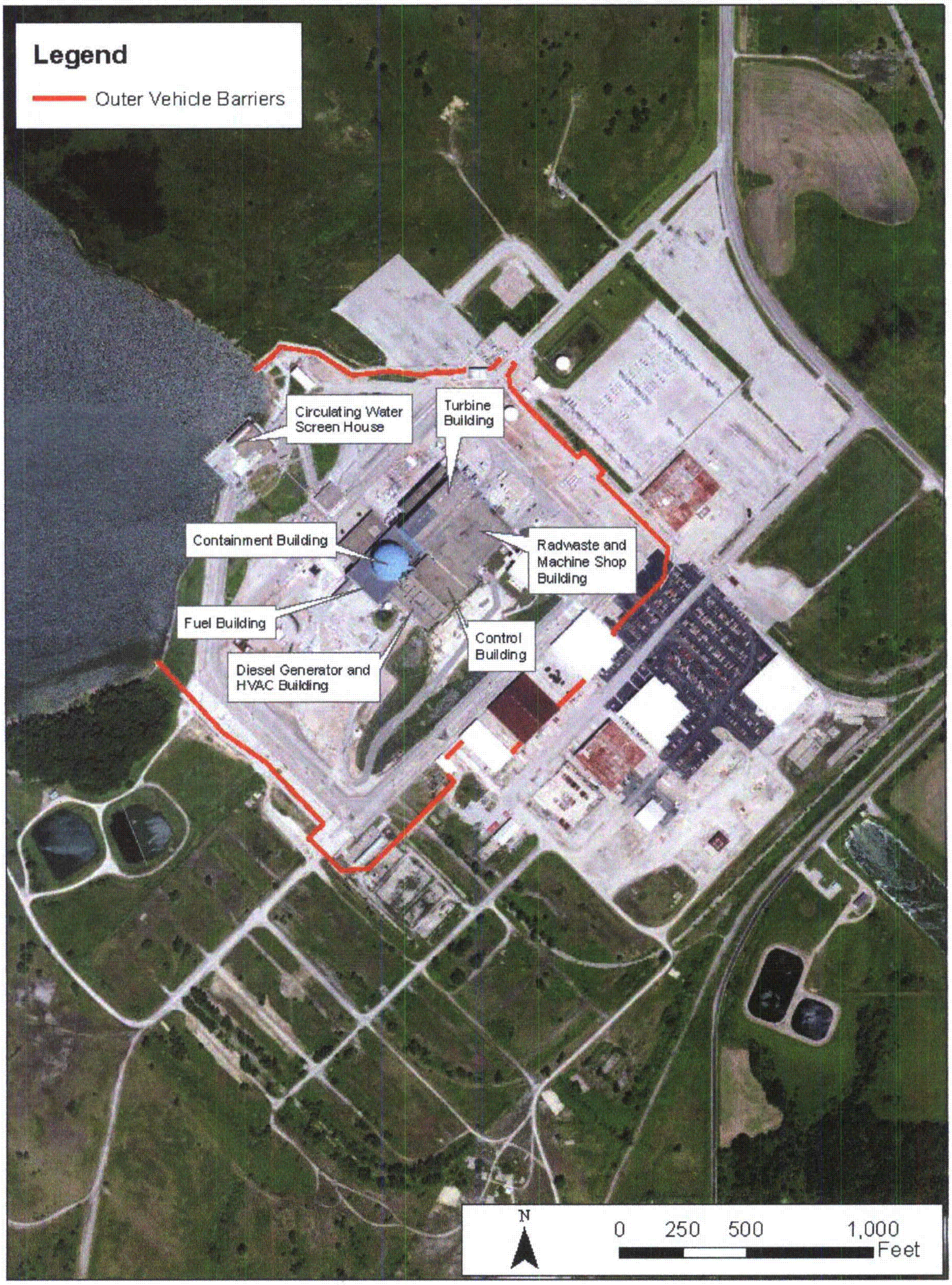

Figure 2.1.2 – Present-Day Detailed Site Layout

Safety-related elevations are as follows (USAR):

a. normal groundwater table at station site - 675.0 to 729.0 ft
b. fuel building
    1. top of base slab - 712.0 ft
    2. adjacent grade - 736.0 ft
    3. building grade floor - 740.0 ft
c. containment building
    1. top of base slab - 712.0 ft
    2. lowest hatch - 737.0 ft
d. auxiliary building
    1. top of base slab - 707.5 ft
    2. adjacent grade - 736.0 ft
    3. building grade floor - 737.0 ft
e. diesel-generator and HVAC building
    1. top of base slab - 702.0 ft and 712.0 ft
    2. adjacent grade - 736.0 ft
    3. building grade floor - 737.0 ft
f. control building
    1. top of base slab - 702.0 ft
    2. building grade floor - 737.0 ft
g. radwaste building
    1. top of base slab - 702.0 ft
    2. adjacent grade - 736.0 ft
    3. building grade floor - 737.0 ft
h. circulating water screen house
    1. top of base - 657.5 ft
    2. adjacent grade - 698.0 ft
    3. operating floor - 699.0 ft
    4. top of watertight enclosure around Seismic Category I equipment - 730.0 ft
i. cooling lake
    1. normal pool - 690.0 ft
    2. 100-year flood (at station site) - 697.0 ft
    3. 100-year flood (at main dam) - 697.0 ft
    4. probable maximum flood (at station site) 708.9 ft
    5. probable maximum flood (at main dam) - 708.8 ft
    6. 100-year drought (low water) - 682.3 ft
    7. crest of main dam - 711.8 ft
    8. crest of service spillway - 690.0 ft

9. crest of auxiliary spillway - 700.0 ft

j. ultimate heat sink
   1. bottom - 668.5 ft
   2. crest of submerged dam - 675.0 ft

k. shutdown service water system outlet structure
   1. invert of the pipes - 675.0 ft
   2. top of base - 671.0 ft.

The elevations at CPS are referenced to a mean sea level (MSL) datum, USGS 1929 adjustment (USAR). It is determined that CPS MSL vertical datum is the same as National Geodetic Vertical Datum of 1929 (NGVD 29).

The difference in vertical datum between NGVD29 and Mean Sea Level datum, USGS 1929 adjustment for CPS is equal to 0 feet, determined as the difference between Clinton Lake levels on United States Geological Survey (USGS) quadrangle map for DeWitt, Ill., dated 1979 (USGS 1979) and Exelon Transfer of Design Information (TODI) CPS-13-027 (Exelon TODI).

2.2 Current Design Basis

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

2.2.1 Effects of Local Intense Precipitation

The 48-hour Probable Maximum precipitation (PMP) for the site area equal to 33.6 inches was used as the LIP at the CLB. Rational method is used to convert rainfall to runoff. The flows over the tracks are calculated using the standard weir equation. The backwater effects were added to the head to obtain the water surface elevation near the plant building. The water surface elevations at the site are determined by step backwater calculation performed using the U.S. Army Corps of Engineers (USACE) HEC-RAS model (USAR). The CLB maximum water surface elevation around the plant is lower than the plant floor elevation of 737.0 ft NGVD29 except on the northern site of the plant over the portions enclosed by the curved tracks where the elevation would be about 737.2 ft NGVD29 (USAR).

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 16 in (USAR).

2.2.2 Flooding in Streams and Rivers

The only stream at the site proximity is the Salt Creek impounding of which resulted in the lake formation. Therefore, the flooding on streams and rivers considers only the PMF on the Salt Creek (USAR Subsection 2.2.4). The peak probable maximum flood flow of Salt Creek under natural river conditions is 112,927 cfs. The peak probable maximum flood flow into the lake is 175,615 cfs (USAR Subsection 2.4.3.4).

2.2.3 Dam Breaches and Failures

A postulated failure of the cooling lake dam will not result in the loss of water from the ultimate heat sink. In the Salt Creek basin, there are no existing or proposed dams upstream from the

Clinton Power Station; therefore flood waves induced from dam failures that affect the safety-related structures are considered impossible (USAR).

2.2.4 Probable Maximum Flood on Cooling Lake

The cooling lake is designed to provide cooling water to the station and to remove the design heat load from the circulating water before the water circulates back into the station. The cooling lake is designed to withstand the effects of a PMF.

The design precipitation consists of standard project storm (SPS) equal to 50% of the PMP of 48-hour duration followed by three dry days and then PMP of 48-hour duration. The cumulative PMP is equal to 25.2 in. The winter PMP is determined to be smaller than the August or all-season PMP.

Initial loss of 1.5 inches is considered for the antecedent storm. No initial losses are applied to the PMP event. Infiltration losses equal to 0.1 in/hr. are applied for both the antecedent and the PMP events. The rainfall is transformed into runoff using methods described in "Unit Hydrographs in Illinois" by W.D. Mitchell (USAR).

The maximum water level in the lake at the dam site was determined by routing the antecedent standard project flood (occurring 3 days prior to the PMF) and the PMF through the lake using the USACE (HEC) computer program "Spillway Rating and Flood Routing". The maximum water level obtained was elevation 708.8 feet with a peak outflow of 135,360 cfs passing through the spillways.

The maximum water level at the station was determined by making backwater calculations from the dam to the station along the North Fork finger of the lake, a distance of about 3-1/2 miles. The backwater computations were made using the USACE HEC computer program, "Water Surface Profiles", with a starting elevation of 708.8 ft NGVD29 which is the maximum water level at the dam during PMF. The backwater computations resulted in a maximum lake water surface elevation of 708.9 ft NGVD29 at the station. The station grade is 736 feet, and therefore flooding cannot affect the safety of the station (USAR).

The wave activity (wave runup and setup) due to PMF and the coincident 40-mph wind increases the maximum lake water surface elevation to 711 ft MSL at the dam (USAR). Top of the dam is at elevation 711.8 ft NGVD29.

The significant (33-1/3%) and maximum (1%) wave effects of a coincident 40-mph wind were superimposed on the probable maximum flood water level at the station. The estimated wave runup values are 2.95 feet and 4.85 feet for the significant waves and maximum (1%) waves, respectively. Superimposing the wave runup values on the probable maximum flood level at the station site resulted in a wave runup elevation of 711.9 feet for significant waves and elevation of 713.8 feet for maximum (1%) waves. The only station facility that would be affected by the CLB PMF is the circulating water screen house, which has a floor elevation of 699.0 ft NGVD29, but is protected to elevation of 714.0 ft NGVD29.

2.2.5 Storm Surge & Seiche

The CPS CLB considers flooding due to surges or seiches as not applicable for the site (USAR).

2.2.6 Tsunami

The CPS CLB considers flooding due to tsunami as not applicable for the site (USAR)

2.2.7 Ice Induced Flooding

An ice jam occurred along Salt Creek on February 11, 1959 raised the flood level by 2.7 feet. The streamflow records show that the maximum recorded ice jam effects were less than the maximum flood stage and discharge values observed during the period of record.

The average thickness of sheet-ice that could form in the lake area is estimated to be 10 inches. The inlet to the screen house is at elevation 670 feet, 5 feet below the design water level of the ultimate heat sink, giving a water depth of 12.3 feet for station operation during lake low water level conditions. The occurrence of an estimated ice thickness of 10 inches in the intake area when the water level is at elevation 675 feet would not block the flow into the screen house.

2.2.8 Channel Migration or Diversion

As described in the USAR, there is no historical evidence of channel diversion of Salt Creek and North Fork of Salt Creek upstream of the dam site. The dam site is located on the upper reaches of Salt Creek, 28 miles from its source. The topographic characteristics and geological features of the drainage basin indicate that there is no possibility for the occurrence of a landslide that will cut off the streamflow into the lake. The history of ice jam formation did not show evidence of flow diversion during winter months.

2.2.9 Low water Considerations

The minimum lake water level for the 100-year drought is elevation 682.3 feet. The minimum design operating level for the circulating water pumps and the plant service water pumps is elevation 677 feet and elevation 672 feet, respectively. The cooling water supply for normal station operation will not be affected by the 100-year drought condition. The circulating water system is designed to operate with a minimum lake water surface elevation of 677 ft NGVD29. This level is 5.3 feet below the low lake water level during a 100-year drought and therefore, the station cooling water supply will not be affected by a drought as severe as a 100-year drought.

In the event of the occurrence of a drought more severe than a 100-year drought that will bring down the lake water level to elevation 677 ft NGVD29, station shutdown operations will be followed with the ultimate heat sink supplying water for the shutdown service water system. The ultimate heat sink is a submerged pond within the cooling lake formed by the construction of a submerged dam. The top of the submerged dam is at elevation 675 ft NGVD29, which is the design water level for the ultimate heat sink. The performance of the ultimate heat sink will not be affected by low flow conditions in the streams.

2.2.10 Combined Effect Flood

Floods due to combinations of flooding events and their effects are not considered in the CPS CLB (USAR).

2.3 Flood Related Changes to the License Basis

CPS is considered a dry site and is physically protected from all flood hazards described in the CLB. There are no flood related changes to CLB.

2.4 Changes to the Watershed and Local Area since License Issuance

The Clinton Lake watershed has a total area of 296 square miles which includes the area of the lake equal to 7.65 square miles (USAR). Based on aerial images, the changes to the watershed include commercial development within the watershed area, which is a very small percentage of the overall watershed area.

The CPS USAR indicates that there are no existing or proposed dams upstream from CPS; therefore, flood waves induced from dam failures that affect the safety-related structures are considered impossible (USAR, Section 2.4.2.2).

Currently four dams are identified in the CPS watershed (upstream). The failure of the identified dam is examined as part of this reevaluation study.

The changes to the local plant area 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 flooding effects of a PMF on Salt Creek and a LIP on the plant area are the design bases for flood protection of all station safety-related facilities. The considerations for selecting the PMF on Salt Creek as the design flood are discussed in Subsection 2.4.2.2 of the USAR. The effects of the PMF and a coincident wind wave activity on the lake at the station site are discussed in Subsection 2.4.3 of the USAR.

The maximum wave runup elevation at the station site is 713.8 feet, produced by a sustained 40 mph overland wind acting on the PMF still water elevation of 708.9 feet. The station grade elevation of 736 feet is 22.2 feet above the maximum wave runup level and 27.1 feet above the PMF water level. The safety-related facilities in the station area would not be affected by the PMF conditions in the lake. The only station facility that would be affected by the PMF is the circulating water screen house, for which the flood protection requirements are discussed in this section.

Flood protection for the safety-related systems and components in the circulating water screen house are provided to elevation 714 feet. The following protection measures are adopted to waterproof the compartments housing the safety-related systems and components:

a. Water stops are provided in all construction joints up to elevation 714 feet.

b. Water seal rings are provided for all penetrations in exterior walls below elevation 714 feet.

c. Watertight doors are provided for all doorways located on both the entrance walls and the internal walls of the SSW pump rooms which are below elevation 714 feet. The watertight doors leading into the shutdown service water pump compartments are normally kept closed. Watertight doors SDI-9 and SDI-10 are normally kept closed, with SDI-9 chained on the interior side of the compartments and SDI-10 chained on the exterior side of the compartment. Administrative procedures require that plant personnel tour the circulating water screen house periodically to ascertain that systems, equipment, and structures are functioning properly (Q&R 240.04).

d. A hatch is provided on the roof of the essential service water pump structure (elevation 730 feet) for access during PMF.

The maximum water surface elevation due to a local PMP is 736.8 feet in the immediate station area where safety-related facilities are located. With the station floor elevation at 737 feet, the safety-related facilities would not be affected by the local PMP.

The eroded area south of the screen house was first graded and shaped to approximately a 3-to-1 slope. The revetment mat is composed of a double layer of industrial grade nylon fabric which is anchored to the slope by the use of a 2-foot deep trench. Into this mat, a sand/cement mortar is pumped to complete the installation of the revetment on the slopes above the normal pool elevation water line.

3. SUMMARY OF FLOOD HAZARD REEVALUATION

Flooding hazards from various flood-causing mechanisms are evaluated for CPS 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 data.

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

The methodology used in the flooding reevaluation study performed for CPS 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

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

3.1 Effects of Local Intense Precipitation

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 the NUREG/CR-7046.

The LIP and the effects of the LIP, which are the resulting impacts from the flood water surface elevations and flow depths, are determined in Calculation IP-S-0282, Fukushima - 2.1 Flooding Re-evaluation for Local Intense Precipitation Analysis (Exelon 2014f). The effects of the LIP are computed for the safety-related structures at CPS. The assumptions associated with LIP are listed in Calculation IP-S-0282.

3.1.1 Inputs

The inputs for the analysis are described below.

3.1.1.1 Local Intense Precipitation

The LIP at CPS is performed in Calculation IP-S-0277, Fukushima - 2.1 Flooding Re-evaluation for Probable Maximum Precipitation Analysis (Exelon 2014a). The LIP estimates are derived using the generalized hydrometeorological study (HMR No. 51 and HMR No. 52). Point (1-sq.mi.) PMP depths for the CPS are equal to 18.2 inches and 27 inches for 1-hour and 6-hour, respectively. The rainfall hyetograph distribution used is based on Figure B-5 of NUREG/CR-7046 (NUREG/CR-7046), which suggests front loaded storm.

3.1.1.2 Ground Surface Topography

Site topography was based on LiDAR data (ENERCON 2013) for the power block and adjacent areas provided in GIS format supplemented by a digital elevation model (DEM) for Dewitt County, Illinois from the National Elevation Dataset (managed by the USGS National Geospatial Program (USGS 2013b). 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 2013b).

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 volume conservation model. The FLO-2D model simulates open channel flow through a numerical approximation of the shallow water equations. Flood wave progression over the flow domain is controlled by topography and resistance to flow. Flood routing in two dimensions is accomplished through a numerical integration of the equations of motion and the conservation of fluid volume.

A two-dimensional model is appropriate and better suitable model compared to a one-dimensional model to simulate the overland flow conditions at the site, which are sheet flow and shallow open channel flow. The two-dimensional model determines the flow direction based on the ground topography when in the one-dimensional model the flow direction has to be assigned. The two-dimensional model uses a grid to represent the ground surface. Each grid element is treated as a computational cell and the hydraulic relationships are determined

for each cell depending on the hydrologic and hydraulic properties of the cell itself and the surrounding cells. The ground is closely represented in the two-dimensional model because *each grid element is assigned a corresponding ground surface elevation, roughness coefficient*, and, when applicable, reduction factor(s) to account for obstructions (buildings, walls, etc.).

The steps applied to model the LIP event at CPS in the FLO-2D are described as follows:

- Create a grid system using ground surface topographical data;
- Assign properties and pertinent conditions such as computational boundary and outflow elements;
- Specify roughness coefficients (Manning's coefficients) corresponding to the site's land cover type (e.g. concrete, grass, water);
- Identify obstructions completely blocking water flow (i.e. buildings and/or structures);
- Identify obstructions diverting water flow (i.e. security wall barriers);
- Assign precipitation inflow to the model;
- Perform the FLO-2D computation;
- Analyze the results produced by the FLO-2D.

Following the guidance outlined in NUREG/CR-7046 the 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-hour, 1-sq.mi. LIP is equal to 18.2 inches, and the 6-hour 10-square mile PMP is equal to 27.0 inches for CPS (Exelon 2014f).

The project computational boundary is based on the location of site, anticipated location of model boundary conditions, the local drainage area anticipated to influence discharge from or to CPS, and the area anticipated to experience unconfined flow conditions. The local drainage area for CPS was delineated based on CPS topographic information (ENERCON 2013 and USGS 2013b).

The model computational boundary domain includes the Unit 2 excavation, which is located southeast of the main building. The Unit 2 excavation is approximately 38 feet deep. The excavation is maintained as dry by CPS under normal conditions.

The computational domain of the FLO-2D models extend approximately 700 feet west from the power block into Lake Clinton, 2,600 feet south from the power block near Dewitt Road, approximately 2,300 feet east from the power block near the intersection of Dewitt Road with Wren Road, and approximately 2,800 feet north from the power block to Wren Road. The boundaries were aligned with the general topography of the area such that the local site drainage area is entirely within the model limits

The elevations at CPS USAR are referenced to a mean sea level (MSL) datum, USGS 1929 adjustment, which is determined to be the same as NGVD29 vertical datum. The elevations in the topographic survey (LiDAR) are referenced to North American Vertical Datum of 1988 (NAVD88). The vertical datum difference between NAVD88 and NGVD29 is obtained using VERTCON (NOAA 2013e). Based on VERTCON, the vertical datum difference is equal to 0.23 ft (NGVD29 = NAVD88 + 0.23 ft).

3.1.3 Results

Maximum LIP flood elevations next to the main building in the power block area generally range from 710.1 feet (NGVD29) at the southeast side of the main building (near the Unit 2 excavation area) to 736.8 ft (NGVD29) at the northeast side of the main building.

Note that localized high ground (ground elevation of 736.9 ft NGVD29) near the northeast side of the main building results in a localized maximum water surface elevation of 737.0. However, the maximum depth of flow is only 0.1 feet; and therefore, this elevation is not considered representative at the building. Maximum velocities range from 0.2 feet per second (fps) to 20 fps, with the highest velocity occurring at the slopes to the Discharge Flume at the southeast end of the Plant. Significant velocity values up to 15 fps locally are shown in the Unit 2 excavation area due to the steep slopes. Maximum flow depth in the vicinity of the CPS main building ranges from 0.1 to 1.7 feet, with the highest depth occurring at the south end of the main building. Depths up to 17 feet are shown in the Unit 2 excavation area southeast of the main building.

3.1.4 Conclusions

1. The maximum water surface elevations at all locations on the site due to the LIP at CPS result from a PMP depth of 18.2 inches within an hour and 27.0 inches within 6 hours.
2. The maximum water surface elevation in the vicinity of the main building in the power block area is 736.8 feet, NGVD29 at the northeast side of the main building.
3. The maximum water surface elevation in the Unit 2 Excavation resulting from the LIP is 714.2 ft (NGVD29).

The maximum reevaluated water surface elevation due to the LIP at CPS equal to 736.8 ft NGVD29 does not exceed the CLB maximum LIP water surface elevation equal to 736.8 ft NGVD29. The maximum reevaluated water surface elevation due to the LIP is below the station floor elevation of 737 ft NGVD29.

3.2 Probable Maximum Flood on Lake Clinton

The probable maximum flood is the hypothetical flood (peak discharge, volume and hydrograph shape) that is considered to be the most severe reasonable possible, based on comprehensive hydrometeorological application of probable maximum precipitation and other hydrologic factors favorable for maximum flood runoff such as sequential storms and snowmelt.

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

Flooding in Rivers and Streams

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 IP-S-0277, Fukushima - 2.1 Flooding Re-evaluation for Probable Maximum Precipitation Analysis (Exelon 2014a).

The PMF flow hydrograph for the Lake Clinton, Salt Creek and the North Fork are determined in Calculation IP-S-0278, Fukushima-2.1 Flooding Re-evaluation for Probable Maximum Flood Analysis-Hydrology, (Exelon 2014b). The PMF water surface elevations for the Lake Clinton, Salt Creek and the North Fork are determined in Calculation IP-S-0279, Fukushima-2.1 Flooding Re-evaluation for Probable Maximum Flood Analysis-Hydraulics, (Exelon 2014c).

The results of the wind wave activity coincident with the PMF along Lake Clinton, North Fork and Salt Creek are determined in IP-S-0286, Fukushima - 2.1 Flooding Re-evaluation for Combined Event Analysis (Exelon 2014j). The assumptions associated with PMF on Lake Clinton are listed in Calculations IP-S-0277, IP-S-0278, and IP-S-079.

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 CPS 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 2013a) for the CPS location.

3.2.1.3 Hourly Dew Point Temperature and Wind Speed Data

Hourly Wind Speed and Dew point data at selected Global Hourly station provided by National Climatology Data Center (NCDC) (NOAA 2013b)

3.2.1.4 Snow Data

Using "Median and Extreme Daily Snow Cover by Month" at one selected station 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). Extreme snowpack depths were taken from the GHCN Farmer City, IL Station (NOAA 2013d). Snow-water equivalent (SWE) data is not available for the Lake Clinton watershed. In lieu of site-specific data, SWE (ratio) was considered to be 0.5 for October to April as a conservative estimate based on observed SWE data for other parts of the country (NRCS 2012).

3.2.1.5 Mean Monthly Baseflow

Baseflow was calculated for the North Fork and Salt Creek sub-watersheds based on USGS Surface-Water Monthly Statistics data for USGS Stream Gage 05578500 on Salt Creek near Rowell, Illinois

3.2.1.6 Soil Data

The Natural Resources Conservation Service (NRCS) data is used (NRCS 2013).

3.2.1.7 Surface Roughness Coefficients

Manning's n values for Salt Creek, North Fork of the Salt Creek, and Lake Clinton were developed using orthoimagery photos and using published guidance for selection of Manning's Roughness Coefficients based on "Open-Channel Hydraulics" (Chow 1959).

3.2.1.8 Ground Surface Topography

Site topography was based on LiDAR data (ENERCON 2013) for the power block and adjacent areas provided in GIS format supplemented by DEM information for Dewitt County, Illinois from the National Elevation Dataset (managed by the United States Geological Survey (USGS) National Geospatial Program (USGS 2013b). 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 2013b).

3.2.2 Methodology

The maximum water surface elevation along Lake Clinton, North Fork and Salt Creek are determined using the USACE HEC-RAS computer software. The detailed description of the methods utilized in the PMF analyses is presented below.

3.2.2.1 Alternative 1 Precipitation Input

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

The all-season PMP estimates for CPS 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.

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 2013a) for the CPS location.

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 snowmelt estimate conservatively used average monthly maximum dewpoint values. Monthly maximum dewpoint data points were extracted from recorded hourly data, for cool-season months (November through March) at one selected station and then used to calculate an average monthly maximum for each month. Dewpoint data was considered only for those periods coincident with rainfall to reflect realistic values for rain-on-snow periods. Note that the temperature of saturated air in the snowmelt equation is measured at 3 meters (10 feet) above the snow surface. However, according to NOAA's National Climatic Data Center, the temperature of saturated air is measured approximately 6 feet above the ground. As a conservative approach, the temperature of saturated air input in the snowmelt calculations is the temperature values measured by the climatological gage station (i.e., the 6 feet above ground level is typically slightly warmer than the 10 feet above ground level).

Wind velocity was calculated using historical climatological data (NOAA 2013a) available from the same gage used for the dewpoint calculation. The snowmelt calculation used average monthly wind velocity values. The average wind velocity was calculated based on recorded hourly data, for cool-season months (November through March) at one selected station for each month. The months of October and April were not included as months that could generate significant snowmelt due to the low value of recorded maximum monthly snow cover.

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 from the historical observations to the magnitude of 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 CPS 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 cool-season monthly PMPs are calculated using HMR No. 53 to develop the seasonal PMP depths for the 72-hour, 10 square mile storm by month of

occurrence. The HMR No. 53 does not provide guidance for watershed areas greater than 10 square miles. Thus, the seasonal variation of the PMP (i.e., all-season PMP/seasonal PMP ratio) is considered to be constant for other watershed sizes to calculate the seasonal PMP depths for the watershed and sub-watersheds based on the 10-mi^2 PMPs presented in HMR No. 51 and HMR No. 53.

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 as the input to the energy budget equation, dew point temperatures and wind speeds, are determined from the historical data obtained from the meteorological stations in close proximity to CPS.

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 CPS.

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 CPS watershed is delineated using the USGS StreamStats (USGS 2013a). The watershed and sub-watersheds delineation is verified using USGS topographic maps (USGS 1996).

3.2.2.5 Infiltration Loss Rate

The initial loss is zero when the sub-watershed is completely saturated. Initial losses are higher if dry conditions prevail. Initial loss represents the precipitation depth prior to the onset of runoff and is used only for the calibration/verification process. During the PMF, initial losses are set to zero.

Constant loss rate describes the infiltration characteristics of the subwatershed soils. The constant loss rate was initially calculated by the following process:

1. The NRCS State Soil Geographic Data Base (STATSGO) for the United States was downloaded from NRCS (NRCS 2013). The nation-wide soil database extent was trimmed to conform to the delineated sub-watersheds. The soil database contains information for each delineated soil type (or classification), represented by a GIS shape file.
2. The hydrologic soil group (HSG) classification (A, B, C, or D from lowest runoff potential to highest runoff potential) was determined from the soil database (NRCS 2013). Conservative runoff potential was selected for soil types having more than one HSG (i.e., D for soils classified A/D). Distinct shapefiles were created in ArcMap 10.1 for each HSG for each of the subwatersheds.
3. The minimum initial infiltration rate (Pilgrim 1993) for each HSG classification was conservatively utilized. Area-weighted constant infiltration rates for each sub-watershed were calculated for each soil group within each watershed.

Constant loss rates were developed using an "iterative process" process during the calibration and verification of the watershed runoff model

3.2.2.6 Unit Hydrograph

Synthetic unit hydrographs are calculated for two sub-watersheds based on published information by the Illinois Institute of Natural Resources (Singh 1981). Instantaneous rainfall-runoff transformation is applied to the Reservoir Watershed (lake area).

The calculated UH for each sub-watershed was adjusted by increasing the peak discharge by one-fifth and decreasing the time-to-peak by one third as per NUREG/CR-7046.

3.2.2.7 HEC-HMS Model Calibration

The approach for modeling the hydrology of a gaged portion of a watershed is to utilize observed USGS stream gage data to calibrate the HEC-HMS model through optimization of initial input parameters. Observed reservoir water surface elevations were used at the Lake Clinton Dam in lieu of observed stream flow data within the watershed, which was not available. These recorded USGS reservoir water surface elevation supplemented by reservoir water level records maintained by Exelon and associated National Climate Data Center (NCDC) rainfall data were used as inputs for the watershed model. The observed USGS water surface elevation data were also used to calculate initial model input parameters such as the starting water surface elevation in the reservoir at the onset of candidate storms for the calibration process. The observed USGS water surface elevation data were also used to verify the model after the completion of the calibration process.

3.2.2.8 HEC-HMS Dam Modeling

The Lake Clinton Dam was included in the HEC-HMS model. The Lake Clinton Dam is an earthfill dam with a maximum height of 65 feet and a length of 3,040 feet that was constructed to create Lake Clinton for CPS. The top of the dam is at elevation 711.8 feet NGVD29. The dam is owned and operated by Exelon. The dam includes a service spillway and an auxiliary spillway. The service spillway is an uncontrolled concrete ogee type semicircular in plan, with a crest length of 175 feet at elevation 690 feet NGVD29. The ogee weir discharges to an 80-foot wide concrete chute and into a stilling basin. The auxiliary spillway was designed to pass floods more severe than the 100-year flood. The spillway is an open-cut type with a crest of 1,200 feet and a crest elevation of 700 feet NGVD29. The dam outlet works is primarily used to release a minimum flow of 5 cfs to Salt Creek (i.e., creek downstream of dam) for environmental purpose. The discharge capacity of the outlet works with the lake at normal pool (690 feet NGVD29) with all gates fully open is 170 cfs (USAR). Note that the outlet works were not considered in the dam discharge calculations as a conservative measure, and because the outlet works are generally not operated during floods.

The elevation-storage data for the lake and elevation-discharge information for the dam that were incorporated in the HEC-HMS model were based on information included in figures from the Clinton USAR. The elevation-discharge curves for the service and auxiliary spillways, and the table with the total elevation-discharge data for the dam were verified by a separate calculation. A back calculation for the service and auxiliary spillways weir discharge coefficients of the elevation-discharge data from the USAR shows that the service spillway weir discharge coefficients accounts for tailwater effects (i.e., weir discharge coefficient decreases from elevation 700 feet NGVD29 through elevation 710 feet NGVD29).

The elevation-discharge curves for the service and auxiliary spillways, and the table with the total elevation-discharge data for the dam consider that the spillways are free from debris blockage; therefore, the spillways approaches, weirs, and discharge areas are considered to be free from debris blockage. The service spillway is not expected to have its discharge capacity decreased due to debris blockage based on its uncontrolled nature, which does not include a structure (i.e., gate, stoplogs, or a bridge over the weir crest) that could trap debris and the anticipated significant depth of flow (18 feet) during the PMF which would likely remove any debris from the spillway. The auxiliary spillway channel, which is typically dry, is unlikely to form debris blockage because of its shallow approach channel, its crest elevation 10 feet above the normal pool elevation, and its 1,200-ft width (or weir length).

3.2.2.9 Temporal Distribution

The temporal order of the PMP was considered. 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).

3.2.2.10 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. 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 cooling pond and dam are modeled using reservoir storage routing. The HEC-HMS model produces results for the water surface elevation in the cooling pond and flow hydrograph for Salt Creek and North Fork for future use in the hydraulic model.

3.2.2.11 Hydraulic Modeling

Hydraulic modeling for Salt Creek, the North Fork and Lake Clinton is performed using USACE HEC-RAS computer software. Cross sections are designated along the Salt Creek and the North Fork and the geometry for the channel, the dam and the bridges obtained using ArcGIS 10.1 computer software. The channel geometry is created using the ground surface topography developed via LiDAR data acquisition (ENERCON 2013). The HEC-RAS unsteady-state model is used to analyze the flow hydrographs obtained from the HEC-HMS hydrologic model to determine the water surface elevation at each cross section.

3.2.3 Results

The maximum PMF water surface elevation at CPS site is 708.0 ft NGVD29. The maximum PMF water surface elevation at the Lake Clinton dam is 707.9 ft NGVD29. These maximum PMF water surface elevations do not include the effects due to the wind activities. The maximum water surface elevations due to the wind-generated wave are discussed in Section 3.4, Combined Events.

3.2.4 Conclusions

The CLB maximum still-water elevation in the lake due to the PMF at CPS site location is equal to 708.9 ft NGVD29. The reevaluated maximum still-water elevation in the lake due to the PMF at CPS site location is equal to 708.0 ft NGVD29. The reevaluated maximum water surface elevation is 0.9 ft below the CLB maximum water surface elevation.

3.3 Probable Maximum Storm Surge and Seiche

The probable maximum storm surge (PMSS) and probable maximum seiche (PMS) are analyzed in Calculation IP-S-0284, Fukushima - 2.1 Flooding Re-evaluation for Probable Maximum Surge and Seiche Analysis (Exelon 2014h).

The storm surge analysis 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 at Clinton Lake. 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." The CPS is not a coastal location; however, the Clinton Lake could be subjected to storm surge (wind setup and wave setup) due to severe wind storms. The assumptions associated with PMSS are listed in Calculation IP-S-0284.

3.3.1 Inputs

The inputs for the analysis are described below.

3.3.1.1 Cooling Lake Bathymetry (Lake Bottom Topography)

The cooling lake bottom topography (bathymetry) was conducted by USGS.

3.3.1.2 Ground Surface Topography

Site topography was based on LiDAR data (ENERCON 2013) for the power block and adjacent areas provided in GIS format supplemented by DEM information for Dewitt County, Illinois from the National Elevation Dataset (managed by the United States Geological Survey (USGS) National Geospatial Program; (USGS 2013b). A single DEM for use in the Delft3D model was developed in ArcMap 10.1 from the LiDAR data (ENERCON 2013) and from the DEM data (USGS 2013b).

3.3.1.3 Atmospheric Forcing (Wind)

A constant over-water wind speed equal to 100 mph is used to initiate a storm surge/seiche event at CPS cooling lake. The wind speed is selected conservatively following the guidance outlined in the ANSI/ANS-2.8-1992.

3.3.1.4 Initial Lake Water Level

Normal pool elevation of 690 ft NGVD29 (USAR) equivalent to 210.45 meters NAVD88, was used as model initial (pre-storm) water level.

3.3.1.5 Lake Bottom Roughness Coefficient

A uniform lake bottom roughness Chézy coefficient of 65, representing a smooth sandy bottom, obtained from the Delft3D-FLOW User Manual (Deltares 2011), was used in the model. Calculations were performed to verify this estimation (Exelon 2014g).

3.3.1.6 Hourly wind data

Hourly and sub-hourly, wind data from several meteorological stations were collected, however the longest complete hourly wind speed, wind direction, wind gust, and pressure data were available from Midway International Airport which lies approximately 120 miles NNE of the CPS. The other nearby meteorological stations has data available for shorter time measurement intervals steps (20 min). However, these stations have data available only for shorter time duration from 2005-2013) and therefore were not used.

3.3.2 Methodology

The probable maximum storm surge on the CPS cooling lake is simulated by applying the probable maximum wind storm caused by a 100-mph overland wind speed. The probability of occurrence of the probable maximum seiche was analyzed.

The wind speed is assumed to be perfectly aligned in the critical fetch direction.

3.3.2.1 Storm Surge

Storm surge is the rise in offshore water elevation cause principally by the sheer force of the wind due to hurricanes, extra-tropical storms, or squall lines acting on the water surface. Storm surge is the product of four processes: pressure setup, wind setup, wave setup, and incident wave runup. Clinton Lake is too small in size to be affected by moving pressure gradients; therefore, pressure term was not used. The incident wave runup is determined separately in Calculation IP-S-0286, Fukushima - 2.1 Flooding Re-evaluation for Combined Events Analysis (Exelon 2014j).

The Probable Maximum Storm Surge (PMSS) results from a combination of meteorological parameters of a probable maximum hurricane or probable maximum wind storm. According to ANSI/ANS-2.8-1992, for the area of the Great Lakes region, the probable maximum surge and seiche should be calculated from the Probable Maximum Winds Storm (PMWS). ANSI/ANS-2.8-1992 further indicates that parameters of the PMWS should be determined by a meteorological study. However, in lieu of a study for the Great Lakes, the following may be used:

1. Set maximum over-water wind speed at 100 mph (~ 45 m/s).
2. Set lowest pressure within the PMWS to 950 mbar.
3. Apply a most critical, constant translational speed during the life of the PMWS.
4. Assume the wind speeds over water vary diurnally from 1.3 (day) to 1.6 (night) times the overland speed.
5. Assume that all winds blow 10 degrees across the isobars over the water body.

The Delft3D computer model was used to evaluate the PMSS on CPS cooling lake. The Delft3D model is set up following the ANSI/ANS-2.8-1992 guidance and is run to simulate 9-hours of real time computation with water level outputs at intervals of 1-minute. The model simulation time does not represent a storm duration time, rather the model simulation time (9-hours) extends long enough for the model to reach dynamic equilibrium at a "steady state" maximum wind and wave setup. As such, the storm surge still water level, and applied wind were held constant throughout the model simulation.

3.3.2.2 Storm Seiche

Seiches may originate from a number of different surface and atmospheric disturbances, of which wind is thought to be the most important. Enclosed basins have certain natural

frequencies of seiche, depending on the geometry of the water boundaries and the bathymetry of water depths. NUREG/CR-7046 states that amplified seiche or resonance occurs when an atmospheric disturbance's period (mainly wind, seismic) matches the natural fundamental lake period.

JLD-ISG-2012-06 noted that "for water bodies with variable bathymetry and irregular shorelines seiche periods and water surface elevation should be determined by numerical modeling." Due to the configuration of Clinton Lake and the presence of berms located in Clinton Lake, the storm surge and seiche evaluation was performed using the Delft3D numerical model.

The HHA approach described in NUREG/CR-7046 was used to determine whether a seiche can result in significant flooding at CPS. This approach involves:

1) Determination the natural oscillation periods of external forces such as extra-tropical storms
2) Evaluation of the natural period of the lake
3) Comparison of Step-1 and Step-2 oscillation periods to determine if resonance is possible
4) If resonance is possible, compute the potential seiche amplitude by applying the external forcing at the resonance period of the lake.

3.3.2.3 Natural Oscillation Periods of External Forces (Wind Storm Frequency/Period)

Spectral analyses for the three major historical non-convective wind storm events were performed by applying a Fast Fourier Transform (FFT) in Microsoft Excel to identify the fundamental frequencies in the wind record.

3.3.2.4 Natural Period of Clinton Lake

The natural period of a lake is primarily a function of its geometry and basin depth and is independent of external forcing mechanisms. Seiche periods can be extracted from observations, modeled, or calculated by spectral analyses or by Merian's formula. Merian's Formula provides a method for estimating the natural periods for the seiche modes in enclosed and semi-enclosed basins. When using Merian's formula, certain simplifying assumptions are made, namely that the lake is a rectangular basin of uniform depth and seiche motions to be confined to the longitudinal axis of the lake. The traditional theory of free oscillations of elongated basins (Merian's formula) is based on the "channel hypothesis" which approximates the difficult two-dimensional problem with a simple one-dimensional problem by integrating over cross sections and assuming that no cross oscillations occur. However, if the rectangular basin has significant width as well as length (e.g., Clinton Lake), both horizontal dimensions affect the natural period of the water body, the one dimensional empirical seiche equation may provide an inaccurate result of the lake seiche period. Therefore, a Delft3D hydrodynamic model was also used to calculate the maximum seiche water level and fundamental period for Clinton Lake.

The 2D hydrodynamic model takes into account the 2D dimensionality of the lake and variable bathymetry.

3.3.2.5 Wind Input for Numerical Simulations

The maximum over-water wind speed for the surge (wind set-up) and seiche analysis was set to 100 mph (~ 45 m/s in the Delft3D model). The Clinton Lake is relatively small in size to be affected by moving pressure gradients; therefore, a pressure term was not used for surge and seiche modeling. Based on the geometry of the lake and the prevailing wind direction the effects of winds from 90°, 180°, 205° (for maximum fetch) and 270° according to the nautical direction were analyzed.

3.3.2.6 Seiche Water Level Modeling

Delft3D was used to investigate any oscillating motions of a seiche at Clinton Lake due to a wind forcing. The model was run to simulate 9-hours of real time with water level outputs at intervals of 1-minute. The model was forced by a constant 100 mph (45 m/s in the Delft3D model) for 4.5 hours, and the forcing wind was removed for the remaining simulation time in order to model the "free oscillation" (once the storm has passed).

For wind-induced seiches, the height of the initial seiche wave will be equivalent to the highest storm surge unless there is resonance. Resonance is the amplification of the seiche water level. Resonance occurs when the dominant frequencies of the external forcing match the fundamental frequencies of the basin.

3.3.2.7 Period of the Seiche Wave

Spectral analyses were performed by applying a FFT (in Microsoft Excel) to the Delft3D time series water level data, similar to the spectral analysis of the wind speed time series, to identify the fundamental frequencies of Clinton Lake.

3.3.2.8 Seismic frequency

Reviews of historical earthquake and seismic records from Seismic sources were determined from the U.S. Geological Survey (USGS) Earthquake Hazards Program hazard fault database (USGS 2013c), and Federal Emergency Management Agency (FEMA) hazard maps (FEMA 2013) were performed to understand the magnitude and the period of seismic activity in the region. Additionally, data from nearby geologic environments was analyzed to determine the potential amplitude (and existence) of potential seismic seiches.

3.3.2.9 Modeling Method Overview

The methodology used to determine the PMSS and wave characteristics (significant wave height and peak wave period) during probable maximum surge event included the following steps:

1. Prepare model grid of the Clinton Lake with the necessary grid resolution to perform storm surge and wave computations.
2. Prepare bathymetry and near site topographic data and interpolate depth files at computational grid points.
3. Prepare model parameters.
4. Set-up a coupled WAVE and Delft3D-FLOW numerical model that accounts for the two dimensionality of the reservoir, wind, topographic, and bathymetric characteristics of the lake and inundation area (if any).
5. Determine the surge (wind and wave set-up) still-water elevation at near site locations.
6. Determine the wave characteristics at near site locations.

3.3.2.10 Delft3D Computational Grid

For the Clinton Lake the grid resolution varies between approximately 7 meters in the areas of interest to 45 meters at the NE end of the reservoir, as shown in Figure 6-5. This gives a total of 620 M-points and 539 N -points. The grid resolution of the lake model was selected to be as fine as possible, subject to computer computational time currently available.

3.3.2.11 6.5.15 Initial Water Level Condition

ANS 2.8,1992 (Reference 36) requires the lesser of the 100-year water level or the maximum controlled water level to be applied as the initial water level in a storm surge model. Measured water level data were not available to calculate the 100-year lake water level at Clinton Lake. Therefore, the normal pool elevation of 690.0 ft NGVD29 (or 210.45 meters-NAVD88 in the Delft3D model), was used as initial water level.

3.3.2.12 Wave Propagation/Transformation

As per ANSI/ANS-2.8-1992, controlling offshore incident waves shall be transformed to the site taking into account shoaling, refraction, diffraction, and reflection. The SWAN, which is a part of the Delft3D modeling package, accounts for (refractive) propagation due to current and depth and represents the processes of wave generation by wind, dissipation due to whitecapping, bottom friction and depth-induced wave breaking and non-linear wave-wave interactions (both quadruplets and triads) explicitly. Wave blocking by currents is also explicitly represented in the model.

3.3.3 Results

3.3.3.1 Surge Water Levels and Wave Characteristics

The Delft3D modeling results show that a westerly wind blowing along the longest fetch direction of the cooling pond (east to west or 90 degrees) causes the largest surge, 0.52 meters, at CPS, and the maximum wave height observed is 1.79 meters. The pre-storm starting water lake level used was the normal pool elevation, i.e. elevation of service spillway of the dam.

Wave characteristics such as significant wave heights and peak wave periods associated with the critical wind storm in the vicinity of the CPS are determined and used as inputs in a separate calculation IP-S-0286, Fukushima - 2.1 Flooding Re-evaluation for Combined Events Analysis, (Exelon 2014j) to evaluate wave run-up effects with combined flooding scenarios.

The most critical condition occurs in front of the Intake Screen House with a water surface elevation of 691.34 feet NAVD88, or 691.57 ft NGVD29, and a significant wave height of 1.93 meters, or 6.33 ft when the wind blows from West to East (270°).

3.3.3.2 Seiche Assessment and modeling

Although the CPS is located on a small, closed basin, not directly influenced by the Great Lakes, but due to the proximity to the Great Lakes it is expected that the strongest non-convective storms affecting the Great Lakes would be responsible for the largest seiche and surge activity. A spectral analysis of wind data for three historical non-convective historical events was performed by applying a Fast Fourier Transform. The spectral analysis shows a dominating fundamental period of 85 and 256 hours for November 1975; 73 and 256 hours for January 1978; and 102 and 171 hours for the November 1998 wind

storm. The fundamental period of Clinton Lake is determined to be equal to 128 minutes, or 2.1 hours.

The Delft3D model results show no observable seiche-like oscillation of the water level, suggesting the absence of resonance with the wind forcing. For this reason, seiche-related phenomenon is not expected to be the controlling flood event at CPS site. It is not expected that under a periodic wind forcing that an amplified seiche would occur, therefore, the maximum water elevations correspond to those of the surge elevations.

Data from nearby geologic environments were analyzed to determine potential of seismic generated seiches. In general, the CPS region is relatively aseismic. The required level of seismic activity to generate an observable seiche is an earthquake magnitude greater than magnitude 6.5 (USGS 2013c), which is absent from the region. Therefore, seiche due to seismic and earthquake activity is not a probable flooding scenario for CPS.

3.3.4 Conclusions

The results of the dam failure calculation (Exelon 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 surface elevation in the lake due to the probable maximum storm surge and seiche, without wave runup, is equal to 691.57 ft NGVD29. The plant grade is at elevation of 736 ft NGVD29. Therefore, the probable maximum storm surge and seiche alone do not result in a flooding hazard for CPS due to a significant available margin equal to 44 ft.

The wave runup is determined separately in Calculation IP-S-0286, Fukushima - 2.1 Flooding Re-evaluation for Combined Events Analysis, (Exelon 2014j) and added to the still-water elevations.

3.4 Dam Failure

The dam failure analysis is performed in Calculation IP-S-0280, Fukushima - 2.1 Flooding Re-evaluation for Upstream Dam Failures Analysis (Exelon 2014d). The assumptions associated with Dam Failure analysis are listed in Calculation IP-S-0280.

3.4.1 Inputs

The inputs for the analysis are described below.

3.4.1.1 Location of Upstream Dams

The location of the dams located in the upstream watershed of CPS is obtained from the National Inventory of Dams (USACE NID) and from the State of Illinois Dam database received via email correspondence (IDNR 2013).

3.4.1.2 Upstream Dam Storage Information

The maximum storage for the upstream dam is obtained from the National Inventory of Dams (USACE NID).

3.4.1.3 Lake Clinton PMF

The PMF on the lake was determined in Calculation IP-S-0278, Fukushima - 2.1 Flooding Re-evaluation for Probable Maximum Flood Analysis-Hydrology (Exelon 2014b).

The HEC-HMS hydrologic model developed for the PMF analysis was used to calculate the one-half PMF maximum water surface elevation at CPS.

3.4.2 Methodology

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 was used for the calculation. The method of analysis is summarized in general terms below:

1. Upstream dam failures may result from a hydrologic event (i.e. PMF), a seismic event, or embankment failure due to piping through the embankment (sunny-day failure). The sunny-day dam failure mode is usually characterized by the absence of a concurrent extreme flood. As per Appendix D of NUREG/CR-7046, the PMF and the one-half PMF scenarios discussed below bound the sunny day failure mode, because upstream reservoir levels used in the calculations were higher (i.e., coincident with top of dam) and coincident river flows are also larger (equal to the PMF for hydrologic event and one-half PMF for the seismic event).
2. The dams in the Lake Clinton watershed were identified based on the NID's database, State of Illinois dams' database, and Lake Clinton watershed delineation.
3. The "Volume Method" described in the NRC's Guidance for Assessment of Flooding Hazards Due to Dam Failure (NRC 2013) was used to calculate the increase in the Probable Maximum Flood (PMF) peak water surface elevation of Lake Clinton due to upstream dam failures (i.e. hydrologic event).
4. The one-half PMF peak water surface elevation of Lake Clinton was calculated and used with the "Volume Method" described in the NRC's Guidance for Assessment of Flooding Hazards Due to Dam Failure (NRC 2013) to calculate the increase in the one-half PMF peak water surface elevation of Lake Clinton due to upstream dam failures (i.e. seismic event).

3.4.2.1 Query Dam Databases

Two databases were used for identifying dams in the Lake Clinton watershed including:

1. National Inventory of Dams (NID) (USACE NID)
2. State of Illinois Dams (IDNR 2013)

The NID database (USACE 2013) provides basic dam information such as height, storage capacity, and latitude and longitude.

The State of Illinois Dams' database is not publically available online (i.e., website). Latitude and longitude information for selected dams were obtained from the Illinois Department of Natural Resources (IDNR) through email correspondence (IDNR 2013).

3.4.2.2 Identifying Dams in Lake Clinton Watershed

A geo-referenced file was created using the ArcGIS software based on latitude and longitude information contained in the NID database, which was used for identifying dams in the Lake Clinton watershed.

The State of Illinois Dams' database (IDNR 2013) was used to validate the location of the dams from the NID database.

3.4.2.3 Calculate the Maximum Elevation at CPS Due to Upstream Dam Failures

The effect of upstream dam failures on the maximum water surface elevation of Lake Clinton at CPS was calculated based on the "Volume Method" described in the NRC's Guidance for Assessment of Flooding Hazards Due to Dam Failure (NRC 2013). The Volume Method is the first of four simplified modeling methods described in NRC, 2013 that can be applied sequentially in an HHA-type gradation of conservatism.

The Volume Method consists of:

1. Estimate and sum the storage volume for all upstream dams ("inconsequential" dams are excluded from the NID database) in the watershed, using the maximum storage volume reported in the NID (i.e. corresponding to the top of the dam).
2. NRC 2013 suggests that the 500-year flood is used to capture antecedent flood conditions at a site. This calculation conservatively uses the PMF elevation as the starting water surface elevation for the hydrologic event and the one-half PMF for the seismic event.

3.4.2.4 Hydrologic Upstream Dam Failures

The increase in water surface elevation due to hydrologic upstream dam failures at CPS was calculated by applying the total storage volume for the upstream dams to the Lake Clinton surface area corresponding to the PMF peak elevation. The flood elevation at CPS was calculated by adding the increase in water surface elevation due to hydrologic upstream dam failures to the PMF peak elevation at CPS determined in the PMF Hydraulics Calculation (Exelon 2013c). This approach is judged to be appropriate based on the PMF hydraulic model results for CPS included in Calculation IP-S-0279 (Exelon 2014c), which indicates that Lake Clinton functions as a level pool (i.e., water surface elevation at CPS is approximately the same as the water surface elevation at Lake Clinton Dam).

3.4.2.5 Seismic Upstream Dam Failures

The one-half PMF is used as the riverine flooding conditions for the seismic upstream dam failures simulation. The one-half PMF peak water surface elevation of Lake Clinton was calculated using the HEC-HMS model developed as part of the PMF Hydrology Calculation (Exelon 2014b). The one-half PMF peak water surface elevation of Lake Clinton was calculated by multiplying a 0.5 factor to the sub-watersheds discharges and reservoir direct precipitation in HEC-HMS for the controlling scenario (all-season) identified in the PMF Hydrology Calculation (Exelon 2014b).

The depth increase in water surface elevation due to seismic upstream dam failures at CPS was calculated by applying the total storage volume for the upstream dams to the Lake Clinton surface area corresponding to the calculated one-half PMF peak elevation. The

flood elevation at CPS was calculated by adding the depth increase in water surface elevation due to seismic upstream dam failures to the one-half PMF peak elevation at CPS.

3.4.3 Results

The depth increase in water surface elevation of Lake Clinton at CPS due to hydrologic upstream dam failures is 0.5 feet. The maximum water surface elevation of Lake Clinton at CPS due to upstream dam failures during the PMF is 708.5 feet NGVD29.

The depth increase in water surface elevation of Lake Clinton at CPS due to seismic upstream dam failures is 0.7 feet. The one-half PMF maximum water surface elevation of Lake Clinton is 702.9 feet NGVD29. The maximum water surface elevation of Lake Clinton at CPS due to upstream dam failures during the one-half PMF is 703.6 feet NGVD29.

3.4.4 Conclusions

The CPS USAR indicates that there are no existing or proposed dams upstream from CPS; therefore, flood waves induced from dam failures that affect the safety-related structures are considered impossible (USAR, Section 2.4.2.2). Currently four dams are identified in the CPS watershed (upstream).

The maximum water surface elevation in the lake due to hydrologic dam failure is equal to 708.5 ft NGVD29, which is 27.5 ft below the plant grade elevation equal to 736.0 ft NGVD29. Therefore, hydrologic dam failure does not result in a flooding hazard for CPS.

The maximum water surface elevation in the lake due to seismic dam failure is equal to 703.6 ft NGVD29, which is 32.4 ft below the plant grade elevation equal to 736.0 ft NGVD29. Therefore, seismic dam failure does not result in a flooding hazard for CPS.

The results of the dam failure calculation (ENERCON 2014d), indicate that floods caused by seismic dam failures are bounded by the PMF with coincident hydrologic dam failure and further evaluation of seismic dam failures is not necessary. CPS flood protection features do not require manual or operator actions in order for the feature to complete its intended flood protection function (USAR). Therefore, the warning time associated with seismic dam failure is not relevant.

The revaluated flood is bounded by the CLB combined event flooding (H.1, H.2, and H.4.2 flooding combinations). As discussed in UFSAR, the cooling lake dam is designed to withstand the effects of PMF. Therefore, the reevaluated flood is not expected to cause a loss of UHS. Additionally, the UHS is a submerged pond formed by submerged dam across the North Fork channel. UFSAR indicates that higher velocities over the submerged dam due to failure of the cooling lake dam (during PMF), does not impact submerged dam and available UHS.

3.5 Combined Events Flood

NUREG/CR-7046, Appendix H.4 states that the following combinations of flood causing events provide an adequate design basis for shore and streamside locations:

<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

H.2 Floods Caused By Seismic Dam Failures

• Alternative 1 – Combination of:
  - A 25-year flood
  - A flood caused by dam failure resulting from a safe shutdown earthquake (SSE), and coincident with the 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
  -

H.4.1 Shore Location

Combination of:
- Probable maximum surge and seiche with wind-wave activity
- 100-year or maximum controlled level in water body, whichever is less

H.4.2 Streamside location

Alternative 1 – Combination of:

- The lesser of one-half of the Probable Maximum Flood (PMF) or the 500-year flood
- Surge and seiche from the worst regional hurricane or windstorm with wind-wave activity
- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

Alternative 2 – Combination of:

- PMF in the stream
- A 25-year surge and seiche with wind-wave activity
- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water

Alternative 3 – Combination of:

- A 25-year flood in the stream
- Probable maximum surge and seiche with wind-wave activity
- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

As indicated in Section 3.4.4, the stillwater surface elevation due to the seismic dam failure is bounded by the hydrologic dam failure. Therefore, combinations listed in H.2 are bounded by H.1 and are not evaluated further.

The H.4.1 combination is analyzed in Calculation L-003858, Beyond Design Basis Storm Surge and Seiche Analysis & Tsunami Screening for Lake (Fukushima) (Exelon 2014f).

The H.4.2 combinations are analyzed in Calculation L-003861, Beyond Design Basis External Flooding Combined Events Analysis (Fukushima), (Exelon 2014h). Additionally, Calculation L-003861 determined the effects of the wind wave activity coincident with the PMF on the lake (waves induced by 2-year wind speed applied along the critical direction). The assumptions associated with Combined Events analysis are listed in Calculation IP-S-0286.

3.5.1 Inputs

The inputs for the analysis are described below.

3.5.1.1 Probable Maximum Surge and Seiche

The probable maximum surge and seiche are determined in Calculation IP-S-0284, Fukushima - 2.1 Flooding Re-evaluation for Probable Maximum Surge and Seiche Analysis (Exelon 2014h).

3.5.1.2 2D Hydraulic Model for Lake

The 2-Diamensional hydraulic model (Delft3D) for the lake is developed in Calculation IP-S-0284, Fukushima - 2.1 Flooding Re-evaluation for Probable Maximum Surge and Seiche Analysis (Exelon 2014h). This model is used to determine the probable maximum surge. The same model is used to determine other wind generated events on the lake for the H.4.2 combinations described above.

3.5.1.3 2-Year Wind Speed

The 10-meter (measurement height), 2-year annual recurrence interval wind speed is required for the coincident wind wave calculations as part of the combined-effects flood analysis as per NUREG/CR-7046. Data for 10-meter, 2-minute duration wind speeds from nearby National Climatic Data Center (NCDC) Station Global Historical Climatology Network-Daily (GHCND) stations, was used as the statistical basis for generating 2-year annual recurrence interval wind speed at the site. The two minute wind speed is an average speed for the most recent two-minute period prior to the observation time. This is considered the sustained wind speed for routine surface observations and is the most readily available data provided by the NCDC (NOAA 2013b). The Springfield Abraham Lincoln Capitol Airport, IL was selected because it reported the fastest 2-minute duration wind speed when compared to other stations in the vicinity of CPS.

Conversion of the raw data to the 2 hour wind speed was done using the following steps:

1. The 2-minute wind speed data from NCDC Station was used. The period of record for this station was from 1995 to 2013, approximately 18 years. The station is located at the Springfield Abraham Lincoln Capitol Airport in Illinois, approximately 50 miles Southwest of CPS.
2. The greatest wind speed from each year during the period of record was selected. The annual maximum wind speeds were sorted in descending order. The Gumbel Distribution, a Generalized Extreme Value (GEV) Distribution (Maidment 1993), was used to calculate the 2 year recurrence wind speed.

The used observation station is located on flat ground with no obstruction from trees and buildings. Due to its proximity to CPS and similar terrain features, the Springfield Abraham Lincoln Capitol Airport is representative of the 2-minute duration wind speed at CPS.

3.5.1.4 Worst Regional Windstorm – Maximum Velocity Wind Speed

The fastest 10-meter, 2-minute duration wind speed from nearby National Climatic Data Center (NCDC) Station Global Historical Climatology Network-Daily (GHCND) stations, was used to determine the worst regional windstorm for CPS (NOAA 2013b). The Springfield Abraham Lincoln Capitol Airport was selected because it reported the fastest 2-minute duration wind speed in the vicinity of CPS. The worst regional hurricane or windstorm wind speed is utilized for combination H.4.2, Alternative 1.

3.5.1.5 25-YearWind Speed

A 25-year surge and seiche is assumed to be caused by a 25-year wind. A 25-year wind speed (4% exceedance probability) is determined using the regional NOAA climate maps. The NOAA Climate Maps provide annual wind speeds for the exceedance probabilities of 1%, 5%, and 10%. The 25-year wind data is based on hourly average and mean monthly wind speeds. Due to the size of Lake Clinton, the optimum duration to generate a surge/seiche is less than an hour. The 4% exceedance probability wind speed value is determined based on the known exceedance probability values using a second order polynomial regression function in Microsoft Excel. The calculated 4% exceedance probability (25-year) wind speed is used as a forcing mechanism in Delft-3D to model a 25-year surge and seiche.

3.5.2 Methodology

The analysis of the combined events is performed based on the guidelines outlined in ANSI/ANS-2.8-1992 and NUREG/CR-7046. The wind wave parameters and effects are determined using the guidance and methodology outlined in the USACE Coastal Engineering Manual EM 1110-2-1100 (USACE 2008). The detailed methodology used in the analysis is presented below.

3.5.2.1 Wind Speed Adjustments

The wind speeds are adjusted following the guidance outlined in USACE Coastal Engineering Manual (CEM), Part II, Chapter 2 (USACE 2008) for level (elevation), duration, and overland to overwater wind speeds.

3.5.2.2 Flooding on Lake Clinton Modeling

The HEC-HMS model developed as part of the PMF hydrology calculation (Exelon 2014b) was utilized to calculate the ½ PMF, 25-year flood, and the 500-year flood peak water

surface elevation of Lake Clinton. PMF hydraulic model results for CPS included in Calculation No. IP-S-0279 (Exelon 2014c), indicate that Lake Clinton functions as a level pool (i.e., water surface elevation at CPS is approximately the same as the water surface elevation at Lake Clinton Dam). Therefore, the resulting flood elevations determined using HEC-HMS were utilized for the ½ PMF, 25-year flood, and the 500-year flood.

3.5.2.3 One-Half PMF

The all-season PMF with the four heaviest 6-hour increments of the PMP in a 24-hour sequence, where the peak six-hour increment falls on the seventh position, is the critical PMF scenario for Lake Clinton as determined in the PMF-Hydrology calculation (Exelon 2014b). This PMF scenario was therefore used to determine the ½ PMF estimates by multiplying the all-season PMF discharges in HEC-HMS by ½.

3.5.2.4 500-Year Flood

The 500 year flood was developed using precipitation frequency estimates in NOAA Atlas-14 precipitation frequency data (NOAA 2013a). NOAA Atlas-14 contains precipitation frequency (PF) estimates for recurrence intervals from one year to one thousand years. Durations range from 5-minutes to 6-days. For the 500 year flood, the mean value for the 500 year recurrence interval and a 24-hour duration was selected.

The temporal distributions are cumulative percentages of precipitation and duration at various percentiles of duration. Four scenarios representing the quartiles where (in the temporal distribution) the most precipitation occurs are available from NOAA Atlas 14. The first quartile case is where the greatest percentage of the total precipitation falls during the first quarter of the time period, i.e., the first 1.5 hours of a 6 hour period, the first 3 hours of a 12-hour period, etc. The second, third, and fourth quartiles are for cases were the most precipitation falls in the second, third, or fourth quarter of the time period. Linear interpolation was performed to calculate the cumulative percentage of precipitation at 2 hour intervals (between the published 6 hour intervals).
The HEC-HMS model (Exelon 2014b) was used to calculate the 500-year flood and the resulting water surface elevation. A sensitivity analysis was performed in HEC-HMS to determine which of the four quartiles with the greatest percentage of total precipitation resulted in the highest peak water surface elevation of Lake Clinton for the 500-year flood.

3.5.2.5 25-Year Flood

The 25-year flood was developed in the same way the 500-year flood, using NOAA Atlas 14 precipitation frequency data (NOAA 2013a).

3.5.2.6 Combined Event Modeling

The probable maximum surge and seiche are analyzed in Calculation No. IP-S-0284, Fukushima – 2.1 Flooding Re-evaluation for Probable Maximum Surge and Seiche Analysis (Exelon 2014h), where a comprehensive 2-dimentional hydrodynamic was developed (Delft3D).

The Delft3D computer software was used for the hydrodynamic modeling of the probable maximum surge and seiche on Lake Clinton. The same model is utilized in this calculation to analyze the surge and seiche for the maximum regional windstorm, a 25-year surge and seiche, and to model the wind waves generated by the 2-year wind speed for the combined event scenarios H.4.1 through H.4.2.

3.5.2.7 Wind-Wave Effects

Lake Clinton at CPS would be susceptible to the formation of wind-generated waves. The calculation methodology includes the following steps:

1. Calculate the straight line fetch in the critical direction;
2. Calculate wave height and period using the CEDAS-ACES Version 4.03 wave prediction application, including transformation of 2-minute wind speed to critical final wind speed duration;
3. Determine the wave runup using the CEDAS-ACES Version 4.03 wave runup and overtopping on impermeable slopes application.

3.5.2.8 Determine the Straight Line Fetch

Due to the irregular geometry of Lake Clinton near CPS, five wave runup cross sections were established to capture the range of conservative fetch lengths at the CPS screen house. The screen house is potentially susceptible to flooding during combined events. The building grade floors at other safety related structures are at elevation 737 feet NGVD29 and not along the shoreline of CPS.

The over-water fetch lengths were calculated using the probable maximum water elevation from the HEC-RAS model as described in Calculation No. IP-S-0279, Fukushima – 2.1 Flooding Re-evaluation for Probable Maximum Flood Analysis - Hydraulics (Exelon 2014c) in conjunction with the National Elevation Dataset (NED) 1/3 arc second DEM for the vicinity of CPS (USGS 2013b). The water surface elevation at the site during the PMF was used in establishing the outline of the flooded area. Fetch lengths were calculated using the measure tool in ArcGIS.

3.5.2.9 Development of the Wave Height and Period

CEDAS-ACES Version 4.03, developed by the U.S. Army Engineer Waterways Experiment Station, includes an application for determining wave growth over open-water and restricted fetches in deep and shallow water.

The deep water wave growth formulas were utilized. The simplified wave growth formulas predict deep water wave growth in accordance to fetch and duration-limited criteria. These formulas are bounded (at the upper limit) by the estimates for a fully developed spectrum and accredit wave growth importance to the fetch length. The following variables were developed as input to the program to calculate wave height and period:

1. The elevation, duration, observation type, and speed of the observed wind speed
2. Duration of the final wind speed
3. The air-sea temperature difference
4. Latitude of the Observed Wind Speed
5. Wind fetch length
6. Wind fetch option determined from bathymetry

The 2-minute duration, 2-year return period wind speeds calculated earlier were used as the basis for initial wave development. A wind speed sensitivity analysis was performed in CEDAS-ACES to determine the most critical duration for wave growth. The CEDAS-ACES program uses a standard wind speed transformation to adjust from one duration to another for a specific return period event.

3.5.2.10 Development of the Wave Runup

The "Runup and overtopping on impermeable slopes" application of the CEDAS-ACES v4.03 software program is based on empirical runup equations developed by Ahrens and Titus (USACE 1992).

Near-shore slopes were estimated from the NED DEM for the vicinity of CPS (USGS 2013b). Record elevations of the screen house from the CPS USAR were utilized for the appropriate geometries in conjunction with the site DEM.

3.5.3 Results

1. The wind generated wave runup resulting from a 2-year wind speed applied along the critical direction (per Appendix H.1 of NUREG/CR-7046) on Lake Clinton at the CPS screen house due to riverine combined events was calculated to be 2.4 feet.
2. The maximum wind generated wave runup due to combined events alternatives is equal to 8.5 feet, resulting from the H.4.2 Alternative 1 scenario: the lesser of one-half of the PMF or the 500 year flood, surge and seiche from the worst regional hurricane or windstorm with wind-wave activity, and the lesser of the 100-year or the maximum controlled water level in the enclosed body of water. The wave height due to H.4.2 Alternative is higher than the wave height from H.1 combination. However, the maximum reevaluated water surface elevation with wave height is bounded by the CLB water surface elevation with wave-height.
3. The bounding still-water elevation for riverine flood events is 708.5 ft NGVD29 at the screen house. This flood level results from the hydrologic dam failure in conjunction with the PMF flooding event, as described in Calculation No. IP-S-0280, Fukushima – 2.1 Flooding Re-evaluation for Dam Failures Analysis (Exelon 2014d).
4. The probable maximum water elevation resulting from combined events on Lake Clinton at the CPS screen house caused by a precipitation event (all-season PMF) plus dam failure (H.1 combination), and is the sum of the bounding flood event (still water elevation 708.5 feet NGVD29) and the 2-year wind wave runup (2.4 feet), which results in a water elevation of 710.9 feet (NGVD29). This water elevation bounds the top elevation of the 1-percent wave height for all scenarios.

3.5.4 Conclusions

The maximum combined events flood elevations reported in the USAR including significant and 1-percent wave actions are 711.9 feet NGVD29 and 713.8 feet NGVD29, respectively (USAR). The maximum combined events flood elevations at the CPS screen house resulting from this flood re-evaluation do not exceed the CLB values and is below the plant grade elevation of 737 ft NGVD29.

The floods caused by seismic dam failures (NUREG/CR-7046, Appendix H.2) are bounded by the PMF with coincident hydrologic dam failure (NUREG/CR-7046, Appendix H.1).

Flooding due to combinations listed in H 4.2 (NUREG/CR-7046) was not considered as part of the CLB. However, the reevaluated flood from H4.2 scenarios is bounded by H.1 scenario. Therefore, the reevaluated flood from H.1 scenario provides the controlling water surface elevation for CPS.

3.6 Ice-Induced Flooding

The ice-induced flooding is analyzed in the Calculation IP-S-0281, Fukushima - 2.1 Flooding Re-evaluation for Ice Effects (Exelon 2014e).

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 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 CPS was performed to demonstrate that ice jam flooding is bounded by the Probable Maximum Flood (PMF) at CPS, which produces a water surface elevation of 708.0 ft NGVD29 (Exelon 2014c). The assumptions associated with Ice-Induced flooding are listed in Calculation IP-S-0281.

3.6.1 Inputs

The inputs for the analysis are described below.

3.6.1.1 Historical Ice Jam Data

The U.S. Army Corps of Engineers (USACE) ice jam database (USACE 2013) was queried to obtain the record of ice jams that have occurred on Salt Creek. The period of record available was from 1920 through March 2013.

3.6.1.2 Hydraulic Model for Salt Creek, North Fork and Lake Clinton

The hydraulic model for Salt Creek, North Fork and Lake Clinton is developed in Calculation IP-S-0279, Fukushima - 2.1 Flooding Re-evaluation for Probable Maximum Flood Analysis - Hydraulics, (Exelon 2014c). This model is used to determine maximum water surface elevations at CPS due to the PMF. The same hydraulic model is used to determine the water level at CPS 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 FEMA 100-year flood discharge

Flooding due to collapse of an upstream ice jam or backwater flooding due to a downstream ice jam is conservatively assumed to occur during a FEMA 100-year discharge event. The 100-year discharges from the FEMA Flood Insurance Study (FEMA 2007) for Salt Creek and North Fork Salt Creek are used.

3.6.1.4 Bridge Geometry

The bridge geometry is obtained from the bridge construction drawings provided by the Illinois Department of Transportation (IDOT), (IDOT 1976).

3.6.1.5 Lake Clinton Dam auxiliary spillway geometry

Invert elevation: 700 feet NGVD29 (USAR)
Weir length: 1,200 feet (USAR)

3.6.2 Methodology

3.6.2.1 Identify Historical Ice Jams

The U.S. Army Corps of Engineers (USACE) Cold Regions Research and Engineering Laboratory ice jam database (USACE 2013) was queried to obtain the record of ice jams that have occurred on Salt Creek. The period of record available was from 1920 through March 2013. The ice jam that resulted in the highest ice thickness in Salt Creek was selected for use as the ice jam forming on the upstream and downstream structures.

3.6.2.2 Calculate Ice Jam Thickness of Historic Ice Jam

The database typically reports river stage relative to a local gage datum. The reported river stage was converted into a height above normal river level (at the stream gage station noted in the ice jam database). The river elevation during the ice jam event was calculated by adding the reported river stage to the "zero" stream gage elevation (gage datum). The zero gage elevation was determined based on information from the USGS.

River Elevation during Ice Jam = Zero Gage Elevation + Reported River Stage

To calculate the thickness of the ice jam, it is necessary to estimate what the river elevation would have been without the ice jam. This river elevation was estimated based on the daily flow at the gage the day before the ice jam event to avoid possible ice-induced influences at the stream gage. USGS did not report the stage for the stream gage for the dates around the ice jam event of interest. However, the annual peak flow record at the stream gage does provide a list of peak flow rates and corresponding stage. The daily flow at the gage the day before the ice jam was then compared to the nearest corresponding annual peak flow and its corresponding reported river stage. This river stage was then selected as the river elevation the day before the ice jam (i.e., the river elevation without the ice jam). The thickness of the ice jam was then calculated as follows:

Ice Jam Thickness = River Elevation during Ice Jam – River Elevation day before Ice Jam

3.6.2.3 Calculate Water Surface Elevation from Upstream Ice Jam

Since CPS is located in the middle of two creeks, the North Fork of Salt Creek and Salt Creek, the first significant stream obstruction upstream of the site in each body of water was utilized as the location of a hypothetical upstream ice jam. The upstream structure of CPS located in the North Fork of Salt Creek is the U.S. State Highway 54 Bridge, and the upstream structure located in the Salt Creek is the County Highway 14 Bridge. The hypothetical ice jam was conservatively assumed to be the same elevation as the deck of each bridge.

The peak water surface elevation at CPS due to an upstream ice jam failure was calculated as the reported FEMA 100-year flood elevation (FEMA 2007) at CPS combined with the height of the peak flood wave from the ice jam failure (depth). The peak flood wave depth caused by the failure of the ice dam and release of impounded water was conservatively translated directly to CPS without attenuation. As the deck elevation of the County Highway 14 Bridge is higher than the deck elevation of the U.S. State Highway 54 Bridge, the collapse of an Ice Jam at County Highway 14 would envelope the results of a collapse of an Ice Jam at U.S. State Highway 54 Bridge. Only the collapse of an Ice Jam at County Highway 14 is considered.

3.6.2.4 Calculate Water Surface Elevation from Downstream Ice Jam

The Lake Clinton Dam downstream of CPS was selected for the location of a downstream ice jam. The downstream ice jam was conservatively assumed to block the service spillway entirely (i.e., no flow through the service spillway occurs); therefore, flow during the ice jam event is assumed to pass only through the 1,200-foot long auxiliary spillway. The depth of flow over the auxiliary spillway was calculated based on the broad-crested weir equation.

The water surface elevation in Lake Clinton at CPS conservatively assumed a coincident flow during the ice jam equal to the reported FEMA 100-year flood discharge at Lake Clinton Dam (FEMA 2007). The Lake Clinton water surface elevation was then calculated as the depth of flow over the auxiliary spillway plus the invert elevation of the auxiliary spillway weir.

3.6.3 Results

The calculated peak flood elevations resulting from the collapse of an ice jam at County Highway 14, upstream of CPS, is equal to 704.0 ft NGVD29. The calculated peak flood elevations resulting from backwater due to an ice jam at the Lake Clinton Dam, downstream of CPS, is equal to 702.4 ft NGVD29. The results of ice-induced flooding are bounded by the PMF peak elevation of Lake Clinton, 708.0 ft NGVD29.

3.6.4 Conclusions

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

The historic ice-induced flood was calculated to be the result from the February 11, 1959 ice jam occurring in Rowell, Illinois. A river stage of 24.84 ft was recorded with a calculated ice jam thickness of 4.8 feet.

The flood wave effects from an upstream ice jam result in a water surface elevation of 704.0 ft NGVD29. This elevation is 4 feet below the PMF elevation of 708.0 ft NGVD29, which indicates that ice-induced flooding is bounded by the PMF on Lake Clinton.

The water surface elevation at CPS caused by a downstream ice jam was calculated to be 702.4 ft NGVD29, which is 5.6 feet below the PMF elevation of 708.0 ft NGVD29.

The results from the reevaluation study match the CLB for the ice effects. As described in the USAR the flooding due to the ice effects is determined to be bounded by the PMF on the lake.

3.7 Channel Migration

The channel migration possibility is analyzed in the Calculation IP-S-0285, Fukushima - 2.1 Flooding Re-evaluation for Channel Migration and Diversion (Exelon 2014i).

NUREG/CR-7046 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 towards the site. The assumptions associated with Channel Migration are listed in Calculation IP-S-0285.

3.7.1 Inputs

The inputs for the analysis are described below.

3.7.1.1 Historical Topographic Map

The 1979 and 2011 U.S. Geological Survey (USGS) Topographic maps of the CPS are examined to illustrate general continuity of the lake shore for that period.

3.7.1.2 CPS USAR

Information regarding the site geology, evidence of landslide induced channel diversions near CPS, maintenance of the North Fork channel between CPS and Route 54 and the construction of the discharge flume are examined to determine whether there is any evidence of erosion due to surface faulting, landslide diversion, failure of the North Fork cannel near the site or failure of the discharge flume.

3.7.2 Methodology

The evaluation approach is summarized in general terms below:

- Review historical records, geologic data, and CPS design information to assess whether the Salt Creek and / or the North Fork exhibit the tendency to migrate towards the site.
- Review historic landslides in the vicinity of the site and review landslide potential for causing the channel to migrate towards the site.
- Evaluate present-day channel stabilization and maintenance measures in place to mitigate channel migration of the Salt Creek and / or North Fork.
- Evaluate if the Discharge Flume can fail and result in diversion of water towards the site.

3.7.3 Results

A review of historical data and site information indicates that the Salt Creek and the North Fork have not exhibited a tendency to meander towards CPS. Channel diversion impacts at CPS are not anticipated to occur as a result of landslide due to the lack of susceptible topographic and geological features (Godt 1997). During extremely large floods, such as the PMF, the resulting flow velocities are anticipated to be lower than if the river was free flowing due to backwater effects from the Lake Clinton Dam, which reduces the potential for erosion. Riprap has been placed as a slope protection on the lake shore at North Fork near CPS to mitigate against potential shoreline erosion. The discharge channel is unlikely to fail during extreme floods based on its design and its installed erosion protection, and even if it fails the flow of water would flow away from the plant toward the lake. If the Lake Clinton Dam were to fail, channel migration would be mitigated by the submerged pond (Ultimate Heat Sink) in the North Fork channel

3.7.4 Conclusions

Channel migration is not a potential contributor to flooding at CPS.

3.8 Tsunami

Screening for tsunami sources input to this calculation was performed in calculation Fukushima – 2.1 Flooding Re-evaluation for Tsunami and Coastal Surge Screening Analysis (IP-S-0283) (Exelon 2014g). It was determined that there are no credible tsunami sources credible to generate surge and seiche activity. The assumptions associated with tsunami screening are listed in Calculation IP-S-0283.

3.8.1 Inputs

The inputs for the analysis are described below.

3.8.1.1 Historical Tsunami Data

The Global Historical Tsunami Database (NOAA 2013f), 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 NGDC tsunami-source-event database is global in extent with information dating from 2000 Before Common Era (B.C.E.) to the present.

3.8.2 Methodology

The Hierarchical Hazard Assessment HHA approach described in Section 2 of NUREG/CR-6966 (NUREG/CR-6966) considers the following three steps for assessing tsunamis hazard:

1. Is the site region subject to tsunamis?
2. Is the plant site affected by tsunamis?
3. What are the hazards posed to safety of the plant by tsunamis?

3.8.2.1 Regional Survey Review

The Global Historical Tsunami Database (NOAA 2013f), 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 CPS, extending from 34° to 46° N Latitude and 82° to 95° W, an approximately 630,000 square mile area.

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 2013c) is reviewed to determine the largest earthquake in Illinois history.

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) subaerial that are initiated above the water and impact the water body during their progression or fall into the water body.

The geographical areas where subaerial 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.

3.8.3 Results

Eight events were recorded in the area examined using the NGDC survey. The maximum run-up associated with a historical tsunami was 3.0 feet due to a 1954 hurricane.

The USGS Illinois Earthquake History website (USGS 2013c) shows that, historically, 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.

CPS is located within an area considered to have a low landslide incidence (Godt 1997). Furthermore, the cross section at river mile 14.41 on the North Fork corresponds to the CPS location. The maximum slope calculated from the cross section is less than 25 degrees on the eastern side of the North Fork branch of Lake Clinton. Thus, given a landslide, its velocity would be limited due to the low-angle slope. As a result, the lake bed in the vicinity of CPS is judged unlikely to generate a subaqueous landslide which would result in a tsunami-like wave that could affect the CPS site.

3.8.4 Conclusions

A screening analysis was performed to assess the potential for tsunami. Tsunami is not considered to be a potential contributor to flooding at CPS. The screening analysis for tsunami concluded that while there is a potential for tsunami in the region, the site screening for tsunami concluded that flooding of CPS due to tsunami is not anticipated. Due to the inland location of CPS, coastal storm surge on oceans or the Great Lakes is not considered to be a potential contributor to flooding at the site.

3.9 Error/Uncertainty

The Error/Uncertainty is analyzed in the Calculation IP-S-0289 Fukushima – 2.1 Flooding Re-evaluation for Error/Uncertainty Analysis (Exelon 2014m). The analysis evaluates the errors and uncertainties associated with the Effects of LIP calculation. The flood due to the LIP event at CPS is the controlling flood hazard mechanism because it results in the highest water surface elevations at the site.

The other flooding mechanisms result in lower water surface elevations compared to the controlling flood hazard. Therefore, the error/uncertainty calculation for them is not performed. The assumptions associated with error/uncertainty analysis are listed in Calculation IP-S-0289.

The maximum water surface elevation at Lake Clinton determined in the Combined Effects Calculation (Exelon 2014j) was based on the hydrologic (i.e., PMF) dam failure stillwater elevation, which incorporated a significant amount of conservatism such as the direct transposition of the combined maximum storage of upstream dams into Lake Clinton (Exelon 2014d; "Volume Method" - first and most conservative step on the HHA approach described in NUREG/CR-7046). Additionally, the non-linearity adjustments performed on the sub-watersheds unit hydrographs of the Lake Clinton hydrologic model developed as part of the PMF Hydrologic Calculation (Exelon 2014b) increase conservatism to the PMF calculation. Therefore, the conservatism in the riverine flood mechanism calculation bounds the uncertainties in the inputs. No sensitivity runs were performed as model is not sensitive to these minor changes.

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 PMP temporal distribution
- Error/uncertainty associated with the topographical survey used to develop the ground surface elevations in the computational model used to evaluate flooding due to the LIP (FLO-2D model) based on FLO-2D (FLO-2D) manning's n-values.

3.9.1 Inputs

1. FLO-2D model developed as part of the LIP Calculation IP-S-0282, Fukushima - 2.1 Flooding Re-evaluation for Local Intense Precipitation Analysis (Exelon 2014f).
2. Range of Mannings' n Parameter for paved areas is 0.02 to 0.05, shrubs is 0.30 to 0.40, and grass cover is 0.20 to 0.4.

3.9.2 Methodology

3.9.2.1 Manning's n Parameter Sensitivity Analysis

A sensitivity analysis was performed for the Manning's n parameter used in the FLO-2D model created in the LIP Calculation (Exelon 2014f). The high and low ends of the recommended range of Manning's n for each surface type (e.g., paved area) used in the FLO-2D model are evaluated in the FLO-2D model. The recommended range for Manning's n parameter for the surface types included in the FLO-2D model are as follows:

"Asphalt or concrete" used for paved areas:	0.02 – 0.05;
"Shrubs and forest litter, pasture" used for shrubs and grass areas:	0.30 – 0.40;
"Average grass cover" used for shrubs and grass areas (high end only):	0.20 – 0.40.

The FLO-2D model included in the LIP Calculation (Exelon 2014f) conservatively utilized the high end values of the recommended ranges of Manning's n-value (0.05 for the paved areas and 0.40 for shrubs and grass areas).

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 of 0.02 for paved areas and 0.30 for shrub and grass areas. Note that the surface types "shrubs and forest litter, pasture" and "average grass cover" were grouped into a single surface type named "shrubs and grass areas" in the LIP analysis (Exelon 2014f). This surface type uses the high end of the range of Manning's n value of 0.40. However, the low end values of the range of Manning's n are different for these two surface types (0.20 for "average grass cover" and 0.3 for "shrubs and forest litter, pasture"). Therefore, as a conservative approach, the higher (0.30) of the two low end range values was selected for the grouped surface type "shrubs and grass areas."

3.9.2.2 PMP Temporal Order Sensitivity Analysis

The 6-hour PMP temporal order used as input into the LIP Calculation (Exelon 2014b) is based on Figure B-5 of NUREG/CR-7046 regulatory guideline (NRC, 2011). Note that the PMP hyetographs used in the sensitivity analyses simulations developed in this calculation includes the first hour only of the 6-hour PMP hyetograph used in the LIP simulation (Exelon 2014b). This approach is appropriate because the LIP maximum water surface elevation occurs in the first hour of the simulation.

Three PMP temporal distributions were analyzed in the sensitivity analysis. The 1-hour PMP consists of twelve 5-minute increments of precipitation (e.g., 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12) in which the values denote the order of rainfall intensity, with 1 being the maximum increment and 12 being the minimum increment. The temporal order was varied by manually arranging the temporal distributions to develop the 1-hour PMP hyetographs with the following identification:

- Distribution "A": Used in the LIP Calculation (Exelon 2014f) is based on Figure B-5 of NUREG/CR-7046 regulatory guideline: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12; with 1 being the maximum rainfall increment and 12 being the minimum rainfall increment.
- Distribution "B": Reverse order of Distribution A (i.e., 12, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2, 1);
- Distribution "C": Shift the maximum 5-min increment to the middle of the 1-hour period (period 30-35-min); shift the second maximum 5-min increment after the maximum and the third maximum 5-min increment before the maximum; continue this process for the subsequent 5-min increments (i.e., 11, 9, 7, 5, 3, 1, 2, 4, 6, 8, 10, 12).

3.9.2.3 Topographic Survey Accuracy

The elevation data was prepared by photogrammetric methods using aerial photography. The topographic survey for CPS comply with National Map Accuracy Standards (NMAS) for a Horizontal Scale of 1"=100' and a contour interval of two feet accuracy and +/- 0.184 feet Root Mean Square Error (RMSE) vertical accuracy for spot elevations, at well-defined points. The methodology of the topographic survey was aerial LIDAR mapping of the site with control points for calibration (Quantum Spatial 2014).

Uncertainty regarding onsite flood elevations is generally limited to the level of accuracy of the site survey. The nature of the 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. However, the water surface uncertainty due to survey error of +/- 0.18 feet could occur.

3.9.3 Results

The sensitivity analysis performed for the Manning's n-value resulted in zero difference in the computed water surface elevations (Exelon 2014m).

The sensitivity analysis performed for the temporal distribution of the LIP input resulted in a maximum difference in the computed water surface elevations of 0.2 ft.

3.9.4 Conclusions

The sensitivity analyses indicate that the FLO-2D model is most sensitive to the PMP temporal order, while relatively insensitive to the Manning's n values.

3.10 Associated Effects

The associated effects for the flooding due to the LIP and flooding due to the dam failure (both hydrologic and seismic) are determined in Calculation IP-S-0287 Fukushima - 2.1 Flooding Re-evaluation for Associated Effects Analysis (Exelon 2014k). The assumptions associated with Associated Effects analysis are listed in Calculation IP-S-0287.

3.10.1 Inputs

The inputs for the analysis are described below.

1. Hydrologic Dam Failures maximum water surface elevation (708.5 feet NGVD29), overbank flow velocities, and associated hydrograph at CPS (Exelon 2014d).
2. Seismic Dam Failures water surface elevation (703.6 feet NGVD29) and associated hydrograph at CPS (Exelon 2014d).
3. Maximum flood velocities during the local intense precipitation event (Exelon 2014f).
4. Site safe shutdown elevation of 696.0 feet NGVD29 (CPS, 2012b; Attachment 3).
5. Wave runup (2.4 feet) and deep water wave height (2.2 feet) for controlling combined event scenario, H.2 alternative at the CPS screen house (Exelon 2014j).
6. Screen house deck elevation of 699 feet NGVD29 based on the CPS USAR (Clinton USAR).

3.10.2 Methodology

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC, 2011) was used for the calculation.

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" including flood event duration and warning time

Two locations were evaluated for "flood height and associated effects" including:
1) Power block area for effects of Local Intense Precipitation (LIP);
2) Screen house area for effects of riverine Probable Maximum Flood (PMF) in Lake Clinton.

The calculation methodology includes the following steps:
i. Flooding Resulting from LIP at CPS
ii. Riverine Flooding at the CPS Screen House

Sediment Deposition and Erosion

Maximum velocity at Lake Clinton is considered negligible (approximately 0.2 feet/second), near the screen house based on the hydrologic dam failure results included in the Dam Failure Calculation (Exelon 2014d). According to the NRCS soils database, the soils in the vicinity of the CPS screen house are hydrologic soil group C, and each are composed of approximately 30% of sand, silt, and clay. The group C soils primarily consist of sandy clay

loam. They have low infiltration rates when wetted and consist mainly of soils with a layer that impedes downward movement of water. The permissible velocity for sandy loam is 1.75 ft/sec (NRCS 2014). Therefore, it is not anticipated that erosion and scour at the CPS screen house will be of concern during the hydrologic dam failure coincident with the PMF.

The highest velocity around the main building and power block at CPS due to the LIP was calculated to be 10.9 feet/second on the southeast side of the main building adjacent to the Unit 2 excavation. Flow velocities were calculated to be upwards of 20 feet/second near the discharge flume at CPS (Exelon 2014f).

The majority of the area surrounding the main power block is concrete. The maximum permissible mean velocity threshold for concrete areas is greater than 18 feet/second. Therefore, scour and erosion around the main power block is not expected. While there are changes in flow velocity direction on the eastern side of the main building with flow velocities up to 10.9 ft/second, deposition is not expected to occur on the paved and concrete surfaces. Generally, the non-erodible nature of the site cover will limit the extent of sediment generation. The topography around the discharge flume is generally composed of grass. Short native grass has a permissible velocity of three to four feet/second. While velocities at the discharge flume exceed this threshold and erosion could occur, erosion in this area is unlikely to affect the function of the CPS main power block.

3.10.3 Results

The results due the "flood height and associated effects" analysis (Exelon 2014k) are summarized below:

- The hydrostatic force around the main building at CPS was calculated to be 94.6 lb/ft due to the LIP flooding event.
- The hydrodynamic loads at the main building and power block due to the LIP were calculated to be 2 pounds per foot on the north side of the building.
- Groundwater ingress is not anticipated to adversely affect safety-related equipment at CPS as the buildings with external walls below grade are designed to be watertight up to the floor elevation (one foot above grade) and designed to resist the full hydrostatic head of groundwater.
- The hydrostatic force at the CPS screen house above the operating floor elevation was calculated to be 2,816 lb/ft all around due to the hydrologic dam failures flooding event.
- The hydrostatic force at the CPS screen house for the combined events (H.4.2, Alternative 1 scenario) above the operating floor elevation was calculated to be 165 lb/ft all around the screen house due to the surge/seiche flooding event.
- The hydrodynamic loads at the northwest face of the CPS screen house due to hydrologic dam failures were calculated to be 0.47 lb/ft.
- Wave impact loading (static and dynamic) due to the dam failure flood event and wind generated waves (the H.1 combination) at the northwest face of the CPS screen house was conservatively calculated to be 2,036 lb/ft.
- Wave impact loading (static and dynamic) due to the surge/seiche flood event (H.4.2, Alternative 1 scenario) at the northwest face of the CPS screen house was conservatively calculated to be 8,568 lb/ft.
- Sediment Deposition and erosion is not anticipated to adversely affect the CPS screen house and main building.

- The upstream PMF with hydrologic dam failure flood duration at the site safe shutdown elevation of 696.0 feet NGVD29 was determined to be approximately 92 hours and 79 hours during the upstream seismic dam failure during the ½ PMF.
- The warning time to the site safe shutdown elevation of 696.0 feet NGVD29 was conservatively determined to be 42.5 hours for the PMF with upstream hydrologic dam failure and 48 hours for the upstream seismic dam failure during the ½ PMF.

3.10.4 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 CPS screen house and main building.

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 CPS.

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)

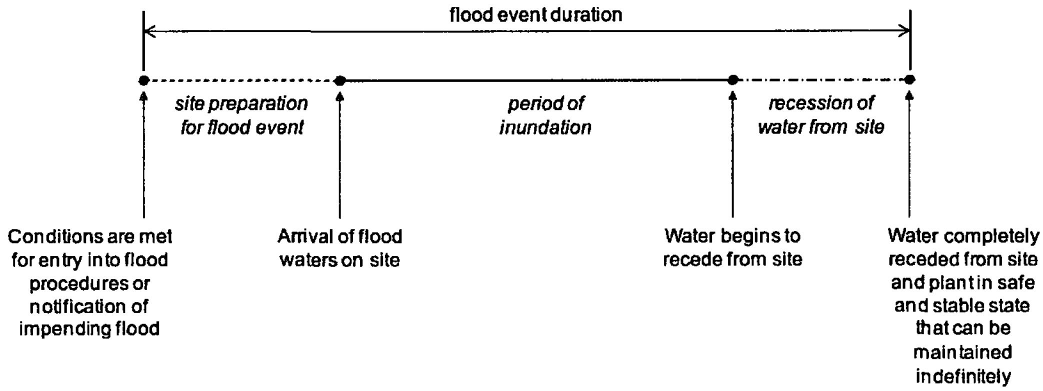

Figure 4.0.1 - Illustration of Flood Event Duration (NRC ISG-2012-05, Figure 6)

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

1. Seiche;
2. Tsunami;
3. Ice Induced Flooding; and
4. Channel Migration or Diversion
5. Seismically-Induced Dam Failure (Combination H.2)
6. Combined-Effect Flood Combination H.4.1

CPS 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 Lake Clinton (including hydrologic dam failure)

3. Combinations in Section H.4.2 of NUREG/CR-7046 (Floods along the Shores of Enclosed Bodies of Water, Streamside Location) for Lake Clinton.

The tables below 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 Value/Methodology	Reevaluation Study Value/Methodology
Probable Maximum Precipitation		
Methodology	HMR 33, USACE EM1110-2-1411	HMR 51 and HMR 52
Storm Duration	48 hours	72 hours
Cumulative PMP (inches)	25.2	27.2
Probable Maximum Flood on Lake		
Nonlinear Basin Response	No	Yes
Hydrologic Modeling	Synthetic UH	Synthetic UH
Total area (sq.mi.)	296	293.8
PMF Inflow into lake (cfs)	175,615	170,000
Lake Modeling Methodology	Stage-Storage	Stage-Storage
Discharge	Service and auxiliary spillways	Service and auxiliary spillways
Wind Wave Activity coincident with PMF on Lake		
Wind speed	40 mph	42.9 mph
Wave parameters determination	Hand calculation	2D Hydrodynamic Model (Delft3D)
Local Intense Precipitation		
Methodology	HMR 33	HMR 51 and HMR 52
LIP Duration (hours)	48	1
LIP (inches)	33.6	18.2
Effects Local Intense Precipitation		
Methodology	Rational Method	Hydrodynamic Modeling using FLO-2D Computer Software
Probable Maximum Surge and Seiche		
Model	Not Evaluated	2-Dimensional Hydrodynamic modeling (Delft3D computer software)
Combined Events (precipitation events on the lake combined with surge and seiche events)		
Methodology	Not Evaluated	Lake elevation – HEC-HMS Surge/Seiche – Delft3D

Table 4.0.2 – Local Intense Precipitation

	Flood Scenario Parameter	CDB	Reevaluated	Bounded (B) or Not Bounded (NB)
Flood Level and Associated Effects	1. Max Stillwater Elevation (ft. MSL)	736.8	736.8	B
	2. Max Wave Run-up Elevation (ft. MSL)	N/A	N/A	N/A
	3. Max Hydrodynamic / Debris Loading (lb/ft)	Not Determined	2 lb/ft	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	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

Notes for corresponding parameter:

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 design basis maximum tornado wind load. 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. Additionally, the water depth around the buildings due to LIP is shallow (in the order of 1 foot).
4. The flow velocities due to the LIP event are determined to be below the suggested velocities (USACE 1984) for the ground cover type (concrete and grave) at the plant area. Therefore, the erosion is not a plausible hazard for CPS.
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. Due to the sandy clay loam (Type C) soils, it is expected that infiltration of precipitation and groundwater seepage would likely be minimal. 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

<table>
<tr><th colspan="2">Flood Scenario Parameter</th><th>CDB</th><th>Reevaluated</th><th>Bounded (B) or Not Bounded (NB)</th></tr>
<tr><td rowspan="6">Flood Level and Associated Effects</td><td>1. Max Stillwater Elevation (ft. MSL)</td><td>708.9</td><td>708.5</td><td>B</td></tr>
<tr><td>2. Max Wave Run-up (Including Setup) Elevation (ft. MSL) (significant/maximum)</td><td>711.9/713.8</td><td>710.9/712.04</td><td>B</td></tr>
<tr><td>3. Max Hydrodynamic and Debris Loading (lb)</td><td>See Note</td><td>8,568 lb/ft /640 lb</td><td>B</td></tr>
<tr><td>4. Effects of Sediment Deposition/Erosion</td><td>See Note</td><td>See Note</td><td>B</td></tr>
<tr><td>5. Concurrent Site Conditions</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
<tr><td>6. Effects on Groundwater</td><td>See Note</td><td>See Note</td><td>B</td></tr>
<tr><td rowspan="4">Flood Event Duration</td><td>7. Warning Time (hours)</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
<tr><td>8. Period of Site Preparation (hours)</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
<tr><td>9. Period of Inundation (hours)</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
<tr><td>10. Period of Recession (hours)</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
<tr><td rowspan="2">Other</td><td>11. Plant Mode of Operations</td><td>Any</td><td>Any</td><td>Any</td></tr>
<tr><td>12. Other Factors</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
</table>

Notes for corresponding parameter:

1. None
2. None
3. As indicated in the USAR, the pressure distribution due to the waves is a combination of hydrostatic and hydrodynamic components and the exposed safety-related structures are designed to withstand these effects. The re-evaluated significant and 1-percent wave height and runup are less than the CLB. Therefore, the hydrodynamic loads at the screen house due to H4.1 combinations are bounded by CLB. The wave impact load of 8,568 lb/ft at the screen house includes breaking wave dynamic and hydrostatic wave load. The debris load due to wooden and ice debris are 320 lb and 640 lb respectively.
4. Maximum velocity at Lake Clinton, near the screen house is negligible, approximately 0.2 ft/sec. The soils around lake screen house are sandy loam and maximum permissible velocity for sandy loam is 1.75 ft/sec. Therefore, the flow velocities due to H.1 combination, including dam failure, are determined to be below the permissible velocities (USACE 1984) for the ground cover type (sandy loam) at the screen house. Additionally, the lake UHS basin is maintained by procedure Q&R 240.2-Sedimentation Monitoring Program.
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. The stillwater level is bounded by the current design basis stillwater level. Therefore, impact to groundwater ingress is considered to be bounded
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 PMF 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 PMF 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 PMF 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 PMF flood.
11. The reevaluated peak flood elevation is bounded by the current design basis. Current plant operations and procedures will still govern.
12. N/A

Table 4.0.4 – Combinations in Section H.4.2 of NUREG/CR-7046

	Flood Scenario Parameter	CDB	Reevaluated	Bounded (B) or Not Bounded (NB)
Flood Level and Associated Effects	1. Max Stillwater Elevation (ft. MSL)	708.9	701.18	B
	2. Max Wave Run-up (Including Setup) Elevation (ft. MSL) (significant/maximum)	711.9/713.8	709.8/713.3	B
	3. Max Hydrodynamic and Debris Loading (lb/ft)	See Note	2,036 lb/ft	B
	4. Effects of Sediment Deposition/Erosion	See Note	See Note	B
	5. Concurrent Site Conditions	N/A	N/A	N/A
	6. Effects on Groundwater	See Note	See Note	B
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

Notes for corresponding parameter:

1. Flooding due to combinations listed in H 4.2 (NUREG/CR-7046) was not considered as part of the CLB. The reevaluated flood from H.4.2, Alternative 1 scenario is bounded by H.1 scenario listed in table 4.0.2.
2. The maximum water surface elevation due to both significant and maximum wave runup is bounded by CDB for PMF scenario, as listed in Table 4.0.3.
3. As indicated in the USAR, the pressure distribution due to the waves is a combination of hydrostatic and hydrodynamic components and the exposed safety-related structures are designed to withstand these effects. The re-evaluated significant and 1-percent wave height and runup are less than the CLB. Therefore, the hydrodynamic loads at the screen house due to H.4.2 combinations are bounded by CLB.
4. Maximum velocity at Lake Clinton, near the screen house is negligible. The soils around the lake screen house are sandy loam and maximum permissible velocity for sandy loam is 1.75 ft/sec. Therefore, the flow velocities due to H.4.2 combination are determined to be below the permissible velocities (USACE 1984) for the ground cover type (sandy loam) at the screen house. Additionally, the lake UHS basin is maintained by procedure Q&R 240.2-Sedimentation Monitoring Program.
5. High winds could be generated concurrent to this combined effect flood. However, manual actions are not required to protect the screen house from river PMF so this concurrent condition is not applicable.
6. The stillwater level is bounded by the current design basis stillwater level. Therefore, impact to groundwater ingress is considered to be bounded.
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 PMF 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 PMF 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 PMF 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 PMF flood.
11. The reevaluated peak flood elevation is bounded by the current design basis. Current plant operations and procedures will still govern.
12. N/A

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Enclosure 2

CD-R labeled:

Clinton Power Station
Pertinent Site Data