**Enclosure 1**

Braidwood Generating Station
Flood Hazard Reevaluation Report
Revision 0

(49 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**

# Braidwood Generating Station

**35100 South Route 53**
**Braceville, Illinois**

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Revision 0
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# Acronyms and Abbreviations

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

# 1. PURPOSE

## 1.1 Background

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

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

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

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

On behalf of Exelon Generation Co. (Exelon) for the Braidwood Generating Station (BGS), 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 re-evaluated 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 BGS, 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 re-evaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the re-evaluated 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 re-evaluated flood causing mechanisms at the site. Provide an assessment of the current design basis flood elevation to the re-evaluated 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 re-evaluated 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 Braidwood Generating Station is located in the southwestern portion of Will County, 2 miles southwest of the City of Braidwood, Illinois.

Condenser water is cooled by water from an essential service cooling pond formed a part of the closed cooling system. The surface area of the cooling pond at its normal pool elevation of 595 ft mean sea level (MSL) is 2,475 acres (UFSAR Subsection 2.4.1.1). The pond is impounded by constructed exterior dikes with the top of the dike elevation varying from 600.0 ft to 602.5 ft MSL (Exelon 1976). Makeup water for the cooling pond is pumped from the Kankakee River. Blowdown water is discharged from the plant by pipeline to the outfall structure and then to the discharge flume and released into the Kankakee River.

The terrain around the plant site is relatively flat, with ground surface elevations varying from 595 ft to 605 ft MSL. The plant grade and floor elevations are 600.0 ft MSL and 601.0 ft MSL respectively (UFSAR). All SSC's at the site are at or above the plant grade elevation of 600.0 ft MSL.

The plant grade elevation (600.0 ft MSL) is 5 ft above the normal pool elevation in the cooling pond equal to 595.0 ft MSL (UFSAR). The site plant grade elevation is 21 ft above the mean water level in the Lake Michigan equal to approximately 579 ft MSL (USACE 2014).


Figure 2.1.1 – Present-Day General Site Map and Topography

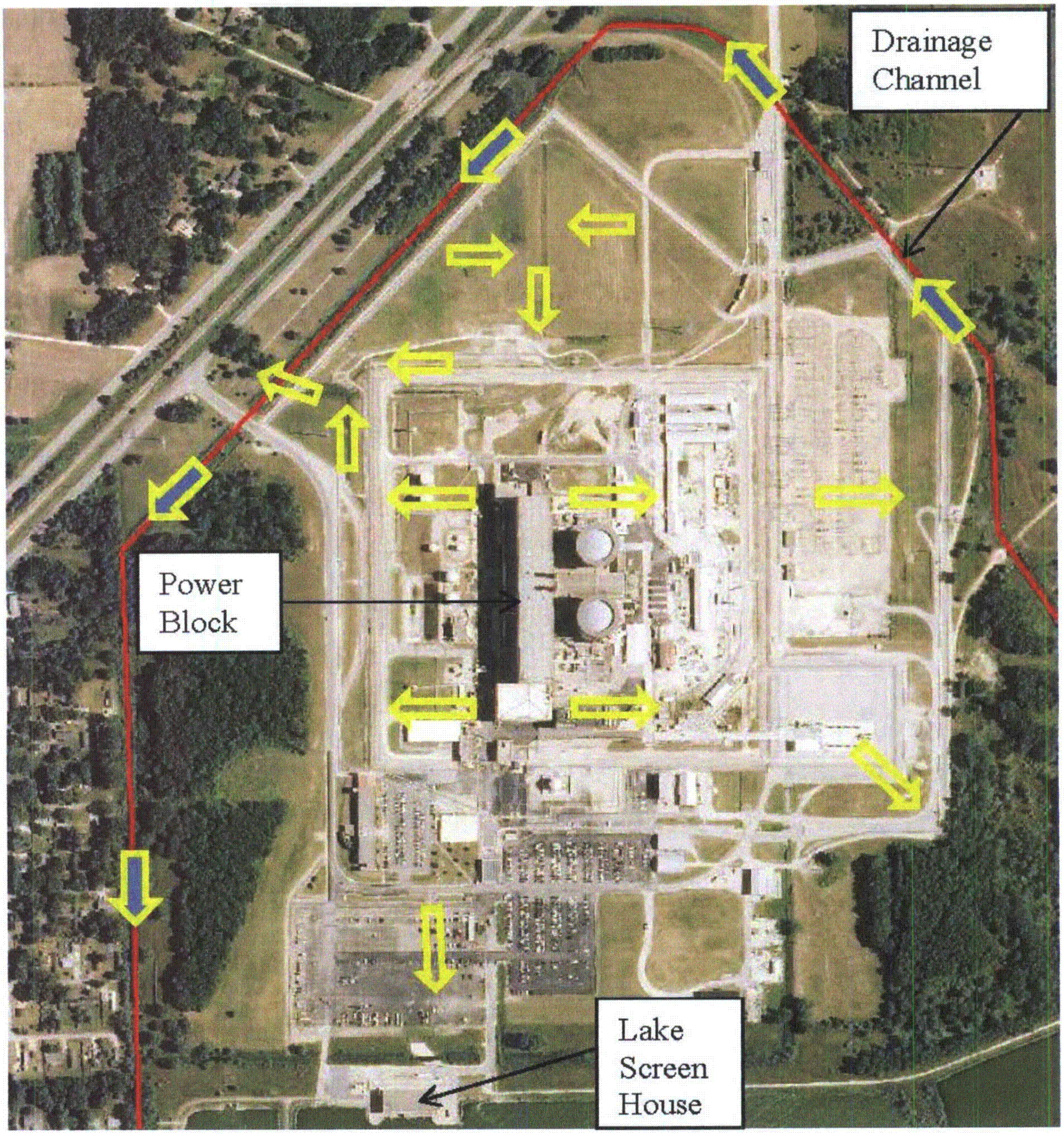


Figure 2.1.2 – Present-Day Site Layout with Flow Directions

## 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 BGS current licensing basis (CLB).

### 2.2.1 Effects of Local Intense Precipitation

The 48-hour PMP at the site equal to 31.9 in is used as the LIP for the CLB. The 1-hour LIP is equal to 17.8 in with the peak 5-minute intensity rainfall equal to 5.98 in. The CLB assumes that infiltration losses are negligible and the site drainage system is not functioning. The peripheral roads and railroads will act as broad-crested weirs to pass site runoff in the event of an LIP at the plant site, with a coefficient of discharge of 2.64. The precast concrete barriers along the outside of the security fence are considered in the analysis. The rational method is used to convert rainfall to runoff with the time of concentration calculated. The coefficient of runoff is conservatively assumed to be 1.0. The water surface elevations at the site are determined by performing a step backwater calculation. The CLB maximum water surface elevation at the site due to the LIP event is equal to 601.85 ft MSL at the east side of the plant (UFSAR).

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 (UFSAR).

### 2.2.2 Flooding in Streams and Rivers

The station is "floodproof" or "dry" with regard to a postulated PMF in the Kankakee River, since the plant floor elevation 600.0 ft MSL is 37.9 ft higher that the stillwater PMF plus wave runup elevation of 562.1 ft MSL (UFSAR). Coincident wind-wave on other rivers, such as the Mazon River, would amount to 1 or 2 feet at most added to the PMF elevation (UFSAR). The stillwater PMF plus wind-wave (assumed to be 2 feet) elevation of 584 ft MSL in the Mazon River is 16 ft below the plant grade elevation.

### 2.2.3 Dam Breaches and Failures

The station is "floodproof" or "dry". The nearest upstream dam on the Kankakee River is at Kankakee about 15 miles from the river screen house. The dam is 12 ft high with a normal pool elevation of 595 ft MSL. Failure of the dam would create minor flood waves that would dissipate before reaching the site area. The nearest downstream dam is at Wilmington, approximately 5 miles from the river screen house. The dam is 11 ft high with a crest elevation of 530.5 ft MSL. A rock ledge across the river 7,700 ft upstream of the dam maintains a pool elevation of 534 ft during low flows. Thus, failure of this dam due to flood flows or seismic disturbance would not affect safety-related portions of the plant.

The maximum water level in the pond is 1.83 ft below the plant grade elevation of 600.0 ft MSL. Failure of the pond dikes would not affect safety-related facilities.

Since the cooling pond is used as the cooling source and not the Kankakee River, plant safety is not affected by postulated blockage or by any other concurrent flooding condition on the Kankakee River.

### 2.2.4 Probable Maximum Flood on Cooling Pond

At normal pool, the pond has a water surface elevation of 595 ft MSL and a surface area of 3.87 square miles. The drainage area of the pond is 5.3 square miles (UFSAR).

The design precipitation consists of one-half of the maximum 6-hour PMP followed by a 3 day dry period and then the PMP of 48-hour duration. The cooling pond PMP is equal to 31.9 in. Initial loss of 0.25 in and a constant loss rate of 0.05 in is applied over the total drainage area. Rainfall-runoff is assumed to be instantaneous due to the very low portion of the drainage area occupied by land surface cover.

The water surface elevation at the pond is determined using reservoir elevation-storage routing. The discharge from the pond occurs through both a 200-foot wide broad-crested spillway. The maximum still-water surface elevation due to the PMF is equal to 598.17 ft MSL. The wave activity (wave runup and setup) due to PMF and the coincident 40-mph wind increases the maximum water surface elevation in the pond to 602.34 ft MSL. In order to provide protection for this extreme event, the exterior dike adjacent to the lake screen house was built 2.5 ft higher than the other locations. The top elevation of this dike is 602.5 ft MSL (Exelon 1976).

### 2.2.5 Storm Surge and Seiche

The BGS CLB states that surge and seiche flooding as not being possible because there is no large body of water near the site (UFSAR).

### 2.2.6 Tsunami

The BGS CLB states that tsunami flooding is not possible because the site is not near a coastal area (UFSAR).

### 2.2.7 Ice Induced Flooding

The flooding caused by ice jams is considered in the CLB. For the Kankakee River, ice induced flooding is determined to raise the water surface near the intake to a maximum elevation of 555 ft. The major tributary closest to the plant is the East Fork of the Mazon River, which lies about 1 mile southwest of the site at its closest point. Because of this distance from the site, there will be no adverse effects on safety-related facilities due to ice in the river and subsequent flooding (UFSAR).

### 2.2.8 Channel Migration or Diversion

There is no historical or topographical evidence indicating that flow in the Kankakee River can be diverted away from its present course. Upstream ice jams will not divert flow completely, since they do not prevent overbank or subsurface flow (UFSAR).

Channel migration of the Mazon River was not discussed in the UFSAR.

### 2.2.9 Low water Considerations

Low flows in the Kankakee River cannot affect safety-related facilities of the plant, since the UHS is independent of flows in the river (UFSAR).

### 2.2.10 Combined Effect Flood

Floods due to combinations of flooding events and their effects do not appear to be considered in the BGS CLB (UFSAR).

### 2.3 Flood Related Changes to the License Basis

There have been no flood-related changes to the licensing basis and no flood protection changes (including mitigation) since the licensing basis.

### 2.4 Changes to the Watershed and Local Area since License Issuance

There are no noticeable changes applied to the Kankakee River and Mazon River watersheds since the issuance of the license based on revisions made to the UFSAR.

### 2.5 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

Based on the design basis PMF for the Kankakee and Mazon Rivers, Crane and Granary Creek and the Cooling Pond and the LIP, the flood elevations are below the elevation that would affect safety-related facilities (UFSAR) so no flood protection or mitigation measures are credited in the CLB. Below-grade penetration seals are credited in the CLB with providing protection against groundwater ingress at groundwater levels equal to plant grade.

## 3. SUMMARY OF FLOOD HAZARD REEVALUATION

Flooding hazards from various flood-causing mechanisms are evaluated for BGS 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 engineering practices.

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

The methodology used in the flooding reevaluation study performed for BGS 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 BGS flood hazard reevaluation study.

## 3.1 Effects of Local Intense Precipitation

Local intense precipitation 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 at BGS is determined in the Calculation BRW-13-FUK-0170, LIP PMP Analysis (Fukushima), (Exelon 2014f). The effects of the LIP result from the flood water surface elevations, flow velocities and impact loads. The effects of the LIP at the safety-related facilities are computed for the safety-related structures at BGS. All assumptions for the calculation are in Calculation BRW-13-FUK-0170 (Exelon 2014f).

### 3.1.1 Inputs

The inputs for the analysis are described below.

#### 3.1.1.1 Local Intense Precipitation

The site-specific study for the LIP at BGS is performed for the calculation BRW-13-FUK-0170 (Exelon 2014f). Following the HHA approach the point-precipitation LIP estimates are first derived using the generalized hydrometeorological study (HMR No. 52). The generalized LIP estimates are very close to the LIP estimates used in the CLB.

#### 3.1.1.2 Ground Surface Topography

The ground surface elevations are collected via Light Detection and Ranging (LiDAR) data acquisition performed in November 2013 (Aerometric 2013). The pertinent features at the site, including buildings, structures, roads, railroad tracks, parking lots, and wall barriers, and the land features, including water, areas with trees and shrubbery, are also depicted from the LiDAR data. Additional ground features at the site are collected during a site ground survey performed by Aerometric in October 2013. The ground survey collected additional information for the relevant features at the site including survey benchmarks, door locations and elevations, additional wall barriers, and storm drain culverts. The LiDAR data is processed to produce data in various required formats using ESRI ArcGIS software in Calculation BRW-13-FUK-0174 (Exelon 2014j).

#### 3.1.1.3 Manning's Roughness Coefficients

Manning's surface roughness coefficients are used in the analysis. The roughness coefficients are selected based on the land cover type identified using aerial topographical survey information (Aerometric 2013), available aerial imagery.

### 3.1.2 Methodology

The Effects of LIP analysis uses a two-dimensional (2D) hydrodynamic model, FLO-2D. FLO-2D is a volume conservation model. The FLO-2D model simulates open channel and overland 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 a more suitable model compared to a one-dimensional model to simulate the overland flow conditions at the site that is dominated by sheet flow and shallow open channel flow. The two-dimensional model determines the flow direction based on the ground topography while in a one-dimensional model the flow direction has to be assigned. A one-dimensional model, such as an unsteady-state HEC-RAS model, is capable to utilize similar computational approaches as the FLO-2D model (equations of motion and volume conservation). However, because the flow direction is initially assigned, the model forces water flows in the assigned general direction rather than determining the direction. The flow path in the one-dimensional model is represented by geometrical cross sections along the assumed flow direction. On the other hand, the two-dimensional FLO-2D model uses a grid layout to represent the ground surface. Each grid element is treated as a computational cell and flow is dispersed along the grid boundary in four directions. The ground is more accurately 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.).

Following the guidance outlined in NUREG/CR-7046, runoff losses are ignored. The roof rainfall is conservatively assumed to be contributing to the overland runoff instead of staying within the parapet walls.

The sub-surface and surface drainage system at the site is assumed to be non-functional at the time of the LIP event. The site's surface is typically paved over the majority of the areas. The expected debris on the site is typically small (e.g. grass, leaves, gravel) and would not cause blockage of openings between security barriers.

The 1-hour, 1-sq.mi. LIP is equal to 17.84 inches and the peak intensity 5-minute LIP is equal to 6.0 inches for BGS (Exelon 2014f). Consistent with the example in Appendix B of NUREG/CR-7046 (NUREG/CR-7046), the front loaded temporal distribution is the expected rainfall distribution for the PMP; however, five temporal distributions are examined for the PMP LIP event at BGS – front, one-third, center, two-thirds, and end loaded rainfall.

The elevations at BGS are referenced to MSL vertical datum as stated in the UFSAR (UFSAR). The ground surface topography and the site-related features are referenced to North American Vertical Datum of 1988 (NAVD88). From the calculation BRW-13-FUK-0177 (Exelon 2014m), the vertical datum difference is equal to MSL – NAVD88 = 0.27 ft.

### 3.1.3 Results

The LIP with the end temporal distribution is determined to be the critical scenario for BGS. The results of the flooding due to LIP at BGS are included in Table 3.1.3. The locations of doors at the safety-related facilities were determined based on a site investigation and shown in Figure 3.1.3. The locations of on-site vehicle barrier systems (VBS) and concrete security barriers are shown in Figure 3.1.4. The maximum water surface elevation at the safety-related doors is 601.67 ft MSL at Containment Building #2. The resulting water surface elevations at the doors leading to the safety-related buildings are presented in Table 3.1.3.

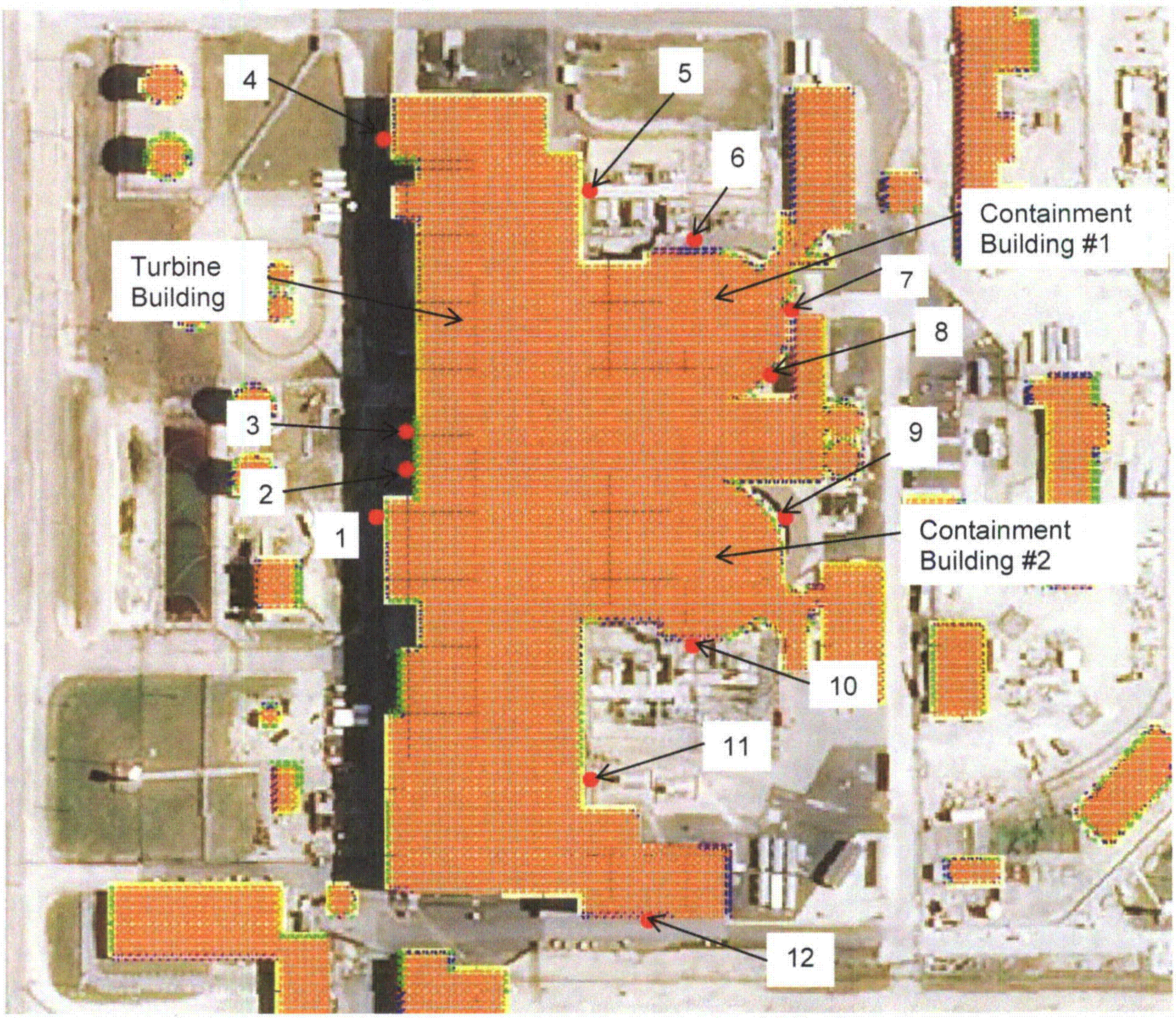


Figure 3.1.3 – Safety-Related Facilities Door Locations (Exelon 2014m)

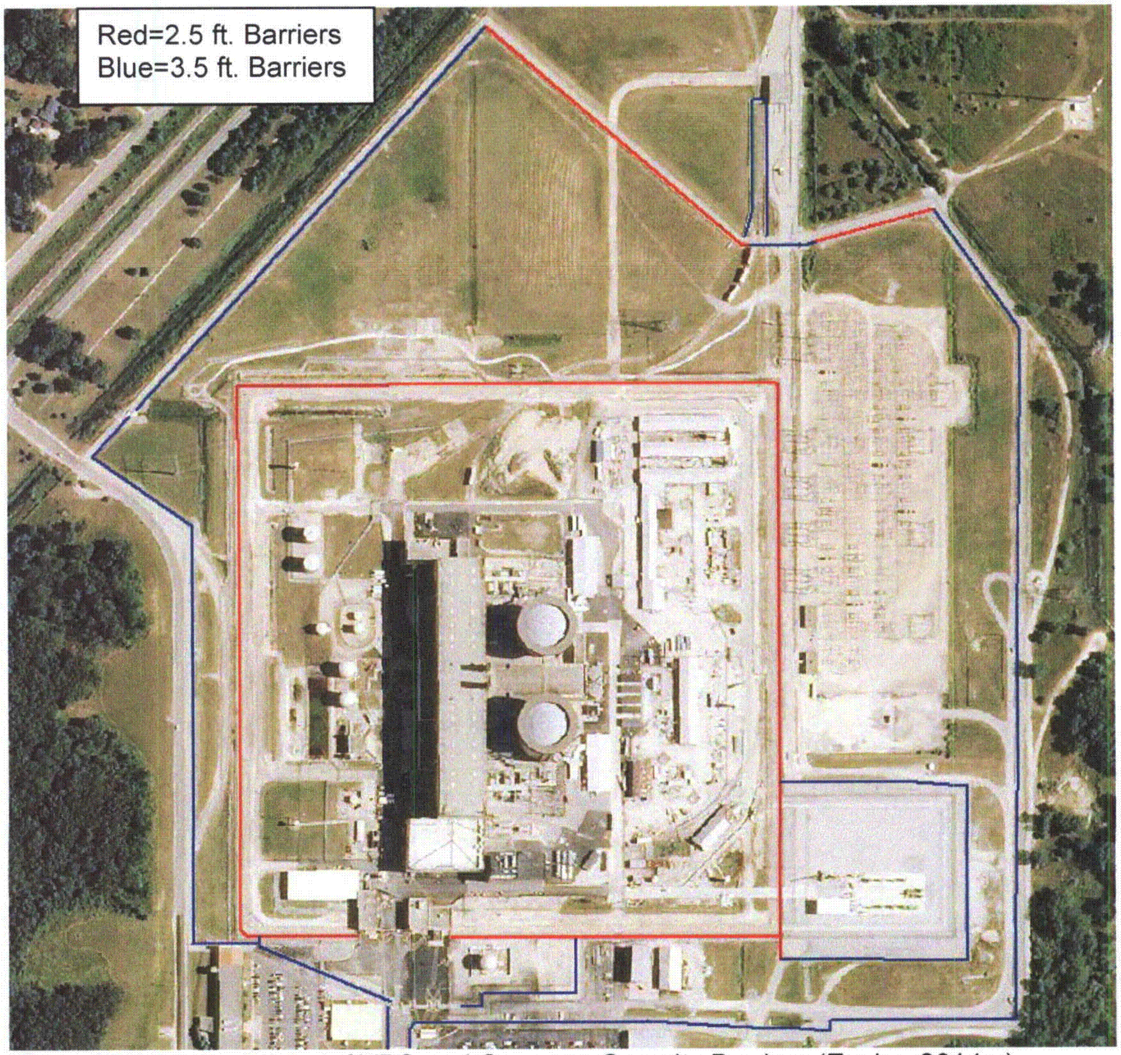

Figure 3.1.4 – Locations of VBS and Concrete Security Barriers (Exelon 2014m)

**Table 3.1.3 – Results from LIP Flood at Door Locations**

| Door Number | Building | Max Water Surface Elevation | Impact Load |
|---|---|---|---|
| | | ft, MSL | lb/ft |
| 1 | Turbine Building | 601.31 | 0.12 |
| 2 | Turbine Building | 601.31 | 0.01 |
| 3 | Turbine Building | 601.31 | 0.40 |
| 4 | Make-Up Building | 601.32 | 0.16 |
| 5 | Turbine Building | 601.47 | 0.25 |
| 6 | Containment Building #1 | 601.51 | 0.00 |
| 7 | Containment Building #1 | 601.55 | 0.19 |
| 8 | Containment Building #1 | 601.55 | 0.07 |
| 9 | Containment Building #2 | 601.61 | 0.00 |
| **10** | **Containment Building #2** | **601.67** | 0.12 |
| **11** | **Turbine Building** | **601.67** | 0.01 |
| 12 | Radwaste Building | 601.59 | 0.82 |

### 3.1.4 Conclusions

Although the grade floors for the safety related structures is at 601 ft MSL, the openings to all buildings housing safety-related facilities are protected by 15 in (1.25 ft) steel barriers up to elevation 602.25 ft MSL. The results presented in Table 3.1.3 indicate that the water surface elevations are below the protection level at safety-related structures at all locations.

The UFSAR indicates that the design basis for the effects of LIP is at 601.85 ft MSL. The re-evaluated flood level of 601.67 ft MSL is 0.18 ft below the design basis elevation.

The impact loading is the hydrodynamic force (lb) per unit water depth (ft) calculated by the FLO-2D model and is a function of velocity and depth at a given time during the model simulation. The maximum impact load at the doors of the safety-related facilities is 0.82 lb/ft at the Radwaste Building.

## 3.2 Probable Maximum Flood in Rivers and Streams

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. For BGS, the Kankakee River and Mazon River with Granary Creek are the pertinent river systems evaluated in the UFSAR and re-evaluated for this report.

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:

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

Alternative 1 – Combination of:

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

Alternative 2 – Combination of:

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

Alternative 3 – Combination of:

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

The precipitation input for Alternative 1, the all-season PMF, is determined in Calculation BRW-13-FUK-0165, All-Season Probable Maximum Precipitation Analysis (Fukushima), (Exelon 2014a).

The precipitation input, including snowmelt, for Alternatives 2 and 3, the cool-season PMF, are determined in Calculation BRW-13-FUK-0166, Cool-Season Precipitation and Snowmelt Analysis (Fukushima), (Exelon 2014b). All assumptions for the calculations are listed in Calculation BRW-13-FUK-0165, All-Season Probable Maximum Precipitation Analysis (Fukushima), (Exelon 2014a) and Calculation BRW-13-FUK-0166, Cool-Season Precipitation and Snowmelt Analysis (Fukushima), (Exelon 2014b).

### 3.2.1 Inputs

The inputs for the analysis are described below.

#### 3.2.1.1 <u>All-Season PMP</u>

The all-season PMP estimates for BGS 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 <u>100-Year Rainfall</u>

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

#### 3.2.1.3 <u>500-Year Rainfall</u>

The 500-year rainfall estimates are obtained from NOAA Precipitation Frequency Data Server (NOAA 2013b) for the BGS location.

#### 3.2.1.4 Maximum Dew Point Temperatures

The dew point temperatures are used as an input to the energy budget equation to calculate snowmelt rate. The dew point temperatures are obtained from the meteorological stations in close proximity to BGS (NOAA 2013d). The hourly data is processed in Microsoft Excel and the maximum dew point temperatures for three consecutive days observed at a general location of BGS are identified.

#### 3.2.1.5 Maximum Wind Speeds

The wind speeds are used as an input to the energy budget equation to calculate snowmelt rate. The wind speeds are obtained from the meteorological stations in close proximity to BGS (NOAA 2013d). The hourly data is processed in Microsoft Excel and the maximum wind speeds for three consecutive days observed at a general location of BGS are identified.

#### 3.2.1.6 Snow Data

The historical snow depth data are obtained from the meteorological stations located in close proximity to BGS (NOAA 2013c). The snow density data for the BGS location is obtained from NOAA Interactive Snow Information website (NOAA 2013e).

#### 3.2.1.7 Mean Monthly Baseflow

The BGS watershed does not have streamflow gages that could be used to determine the base flows. Additionally, there is no available methodology for the state of Illinois to determine the baseflow arithmetically. Therefore, the baseflow is determined based on the records at the stream flow gages located in close proximity to BGS.

#### 3.2.1.8 Soil Data

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

#### 3.2.1.9 Land Use Land Cover Data

The USGS Land Use Land Cover (LULC) data is utilized (USGS 2013b).

#### 3.2.1.10 Surface Roughness Coefficients

Manning's roughness coefficients are obtained based on the topographical survey and available aerial imagery for BGS using the guidelines outlined in the Open-Channel Hydraulics (Chow 1959).

#### 3.2.1.11 Hydrologic Unit Code Watershed Boundaries

For the Mazon River, the initial approximation of individual delineation of the watersheds is taken from hydrologic unit code (HUC) boundaries (USGS 2013c) that are publicly available from the USGS. These boundaries are further refined and adapted for use in the re-evaluation by visual inspection of the ground surface topography.

#### 3.2.1.12 Ground Surface Topography

The ground surface elevations are collected via publicly available LiDAR data performed by the Illinois Height Data Modernization program (Illinois 2004). Cross sections for the hydraulic analysis is taken from 3-meter Digital Elevation Model (DEM) publicly available from the USGS and accessed in August 2013.

### 3.2.2 Methodology

The PMF flow hydrographs for the Mazon River and Granary Creek are determined using USACE HEC-HMS computer software. The maximum water surface elevations for the Mazon River and Granary Creek are determined using the USACE HEC-RAS computer software. The hydrologic analysis for the Kankakee River was completed for the Dresden Flood Hazard Re-evaluation Report (FHRR) as described in section 3.2.2.1. The detailed description of the methods utilized in the PMF analyses for the Mazon River and Granary Creek are presented below.

#### 3.2.2.1 Kankakee River

The PMF flows for the Kankakee River were determined for the Dresden FHRR (Exelon 2013). Examination of the HEC-HMS models for the Kankakee River showed that the Center distribution from storm center KnKSC16 provides the highest peak flow at the Wilmington Dam from Alternative 2 snowmelt analysis. This flow hydrograph is used for the determination of the PMF for the Kankakee River.

#### 3.2.2.2 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 (lesser of 40% PMP or 500-year rainfall).

The all-season PMP estimates for the Mazon River and Granary Creek 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.

The 500-year rainfall estimates are obtained from NOAA Precipitation Frequency Data Server (NOAA 2013b) for both the Mazon River and Granary Creek. The estimates of 40% PMP and 500-year rainfall are compared and the 500-year rainfall is a smaller rainfall event and is used for the Alternative 1 PMF.

As a summary, the Alternative 1 precipitation event consists of a 72-hour 500-year rainfall, followed by a 3-day dry period and then 72-hour all-season PMP.

#### 3.2.2.3 Alternative 2 Precipitation Input

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

The 100-year rainfall estimates are obtained from NOAA Precipitation Frequency Data Server (NOAA 2013b) for both the Mazon River and Granary Creek. The all-season rainfall values are adjusted to represent the cold-season rainfall values using the methodology outlined in the Rainfall Frequency Atlas of the Midwest (Huff 1992).

The snowmelt rate is calculated using the energy budget equation for the rain-on-snow condition following the guidance outlined in USACE EM 1110-2-1406, *Runoff from Snowmelt* (USACE 1998). The dew point temperatures used as the input to the energy budget equation are determined from the historical data obtained from the meteorological stations in close proximity to BGS. The dew point temperatures are determined for the cool-season months October and April since October has the highest cool-season to all-season

PMF ratio and April since it is the month with the pertinent combination of highest dew point temperatures, wind speed and available snowpack for snowmelt. The wind speed typical for the region during the cool-season period is used as an input for the energy-budget equation to determine the snowmelt rates from the probable maximum snowpack.

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.

#### 3.2.2.4 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 BGS location are determined using the charts provided in the hydrometeorological Report No.53 (HMR No. 53) for the cool-season months October and April since October has the highest cool-season to all-season PMF ratio and April since it is the month with the pertinent combination of highest dew point temperatures, wind speed and available snowpack for snowmelt..

The snowmelt rate is also determined for October and April 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 BGS. The maximum dew point temperatures and maximum wind speeds observed at each cool-season months are used as an input to the energy budget to determine the snowmelt rates from a 100-year snowpack.

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 BGS.

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.

#### 3.2.2.5 Infiltration Loss Rate

The NRCS Curve Number (CN) method described by Technical Release 55 (TR-55) (NRCS 1986) is used to estimate the losses due to infiltration. The Antecedent Moisture Conditions (AMC) variable is used to set the pre-storm saturation potential. For the re-evaluation, AMC III was used as it assumes saturated conditions before the storm as a conservative parameter. The LULC and hydrologic soil group data are used to develop the area-weighted CN values for each sub-basin. The infiltration loss rates are applied to Alternatives 1, 2 and 3.

#### 3.2.2.6 Unit Hydrograph

The Snyder unit hydrograph defined is used as the basis to transform rainfall to runoff. The Snyder method is applicable for watersheds between 10 and 10,000 square miles and is based on a set of formulas relating the physical geometry of the watershed to parameters of the unit hydrograph.

#### 3.2.2.7 Routing

The Muskingum method is used to route the Granary Creek watershed through the Mazon River to Highway 53. For discharge from the spillway of the cooling pond draining to the Mazon River, the Muskingum-Cunge 8-point method was utilized due to the near consistency of the cross-sectional geometry of the stream.

#### 3.2.2.8 Temporal Distribution

The temporal distributions evaluated are: front (hour 1), one-third (hour 24), center (hour 36), two-thirds (hour 48) and end (hour 72) peaking events.

#### 3.2.2.9 Hydrologic Modeling

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

#### 3.2.2.10 Hydraulic Modeling

Hydraulic modeling for the Mazon River and Granary Creek is performed using USACE HEC-RAS computer software. Cross sections are designated along the Unnamed Creek and the channel geometry is obtained using USACE HEC-GeoRAS computer software. The channel geometry is created using the ground surface topography developed from publicly available LiDAR (Illinois 2004). 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.

#### 3.2.2.11 Vertical Datum

The elevations at BGS are referenced to MSL vertical datum as stated in the UFSAR. The ground surface topography and the site-related features are referenced to North American Vertical Datum of 1988 (NAVD88). From the calculation BRW-13-FUK-0177 (Exelon 2014m), the vertical datum difference is equal to MSL – NAVD88 = 0.27 ft.

### 3.2.3 Results

The three PMF alternatives (all-season and cool-season) are analyzed. Alternative 1, the all-season PMF scenario, results in the highest peak flows in the Mazon River and is used for the hydraulic stream analysis to determine the peak water surface elevation in the Mazon River and in Granary Creek.

The maximum peak flow in the Kankakee River due to the PMF event is determined to be equal to 259,647.59 cfs. The maximum stillwater surface elevation in the Kankakee River due to the PMF event is determined to be equal to 569.74 ft NAVD88 or 570.01 ft MSL.

The maximum peak flow in the Mazon River due to the PMF event is determined to be equal to 178,475.46 cfs. The maximum peak flow in Granary Creek due to the PMF event is determined to be equal to 90,843.64 cfs. The maximum stillwater surface elevation in the Mazon River due to the PMF event is determined to be equal to 592.46 ft NAVD88 or 592.73 ft MSL. The maximum stillwater surface elevation in Granary Creek due to the PMF event is determined to be equal to 592.61 ft NAVD88 or 592.88 ft MSL.

### 3.2.4 Conclusions

The maximum water surface elevation in the Mazon River is 7.54 ft below the plant grade elevation and 8.54 ft below the grade floor elevations of safety-related buildings. Therefore, the PMF event in the Mazon River does not result in a flooding hazard at BGS. The maximum water surface elevation in Granary Creek is 7.12 ft below the plant grade elevation and 8.12 ft below the grade floor elevations of safety-related building.

The re-evaluated maximum water surface in the Kankakee River at the river screen house of 570.01 ft MSL is 8.71 ft higher than the design basis elevation of 561.3 ft MSL. The re-evaluated maximum water surface in the Mazon River of 592.73 ft MSL is 10.73 ft higher than the design basis elevation of 582.0 ft MSL. The re-evaluated maximum water surface in the Mazon River of 592.88 ft MSL is 16.88 ft higher than the design basis elevation of 576.0 ft MSL.

## 3.3 Probable Maximum Flood on Cooling Pond

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.

The PMF on the cooling pond is determined in Calculation BRW-13-FUK-0176 Cooling Pond PMF Analysis (Fukushima), (Exelon 2014l). All assumptions are listed in Calculation BRW-13-FUK-0176 (Exelon 2014l).

### 3.3.1 Inputs

The inputs for the analysis of both calculations are described below.

#### 3.3.1.1 Cooling Pond PMP

The cooling pond PMP estimates for BGS are derived from the generalized hydrometeorological report (HMR No. 52). The 72-hour PMP estimates are derived and adjusted based on the site location and site watershed area.

#### 3.3.1.2 500-Year Rainfall

The 500-year rainfall estimates are obtained from NOAA Precipitation Frequency Data Server (NOAA 2013l) for the BGS location.

#### 3.3.1.3 Pond Characteristics

The pond stage-area-volume relationship and the spillway discharge rating curve are utilized from the UFSAR as inputs to the HEC-HMS model.

#### 3.3.1.4 Mean Monthly Baseflow

The BGS watershed does not have streamflow gages that could be used to determine the base flows. Additionally, there is no available methodology for the state of Illinois to determine the baseflow arithmetically. Therefore, the baseflow is determined based on the records at the stream flow gages located in close proximity to BGS.

#### 3.3.1.5 Soil Data

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

#### 3.3.1.6 Land Use Land Cover Data

The USGS LULC Data is used (USGS 2013b).

#### 3.3.1.7 Surface Roughness Coefficients

Manning's roughness coefficients are obtained based on the topographical survey and available aerial imagery for BGS using the guidelines outlined in the Open-Channel Hydraulics (Chow 1959).

#### 3.3.1.8 Ground Surface Topography

The ground surface elevations are collected via publicly available LiDAR data performed by the Illinois Height Data Modernization program (Illinois 2004).

### 3.3.2 Methodology

The maximum water surface elevation in the cooling pond is determined using reservoir-storage routing method performed with USACE HEC-HMS computer software. The pond stage-area-volume curve and the spillway discharge rating curve from the UFSAR were used as inputs to the HEC-HMS model. The initial infiltration loss is zero, assuming that no losses occur at the onset of precipitation. The constant loss rate is determined based on the soils and land cover over the portion of the pond encompassing land terrain.

The 72-hour PMP falling on the cooling pond watershed is preceded by the lesser of 40% of the PMP or the 500-year storm followed by a 3-day dry period (NUREG). As a result, the 500-year storm was selected as the antecedent storm entered into the HEC-HMS model.

### 3.3.3 Results

The HEC-HMS model was run for five temporal distributions (front, one-third, center, two-thirds, end) where peak rainfall occurs in different location across the 72-hour time spectrum. The maximum stillwater surface elevation in the cooling pond due to the PMF event is determined to be equal to 599.36 ft MSL occurring with the end temporal distribution. Alternative 1, the all-season PMP scenario, results in the highest peak water surface elevation.

### 3.3.4 Conclusions

The maximum water surface elevation in the Cooling Pond 0f 599.36 ft. is 1.19 ft higher than the PMF elevation in the UFSAR. However, the top of the exterior pond dike is 600.0 ft or higher along the outer perimeter of the pond.

## 3.4 Probable Maximum Storm Surge and Seiche

The probable maximum storm surge (PMSS) and probable maximum seiche (PMS) are analyzed in Calculation BRW-13-FUK-0171 (Exelon 2014g).

The storm surge analysis is performed following the guidance outlined in NUREG/CR-7046, ANS-2.8-1992, JLD-ISG-2012-06 (NRC 2012b), and NUREG/CR-6966. NUREG/CR-7046, Appendix H.4 (NUREG/CR-7046) describes the combined events criteria for an enclosed body of water, which is appropriate for analyzing surge and seiche flooding at the BGS cooling pond. 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 cooling pond could be subjected to storm surge (wind setup and wave setup) due to severe wind storms. The specific combinations directed by NUREG/CR-7046 are stated in section 3.6 of this report. All assumptions are listed in Calculation BRW-13-FUK-0171 (Exelon 2014g).

### 3.4.1 Inputs

The inputs for the analysis are described below.

#### 3.4.1.1 LiDAR Surface Topography

The ground surface elevations are collected via LiDAR data performed by the Illinois Height Data Modernization program (Illinois 2004).

#### 3.4.1.2 Atmospheric Forcing (Wind)

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

#### 3.4.1.3 Hourly wind data

The wind data is obtained from Chicago/Midway International Airport (NOAA 2013f). The data is used to calculate natural oscillation period of external forcing events (wind storms).

### 3.4.2 Methodology

The probable maximum storm surge on the BGS cooling pond is simulated by applying the probable maximum wind storm caused by a 100-mph wind speed. The probable maximum seiche on the cooling pond is modeled by applying the forcing element, the wind speed, with amplitude close in magnitude to the amplitude of the cooling pond.

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

#### 3.4.2.1 Storm Surge

Storm surge is the rise in offshore water elevation caused 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 two processes: wind surge setup and pressure surge setup. The Probable Maximum Storm Surge (PMSS) is the surge that results from a combination of meteorological parameters of a probable maximum hurricane or probable maximum wind storm. 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).

The determination of storm surge height was calculated using equations derived in an article in the Journal of Research of the National Bureau of Standards, "Wind Tides in Small Closed Channels" by Garbis H. Keulegan (Keulegan). The two equations used to determine the PMSS, wind and pressure surge setup respectively, are a function of the wind speed, critical wind speed, depth of water and fetch length. The fetch is the longest straight line length in one direction at a certain elevation spanning the pond.

#### 3.4.2.2 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 pond period.

JLD-ISG-2012-06 (NRC 2012b) noted that "for water bodies with variable bathymetry and irregular shorelines seiche periods and water surface elevation should be determined by numerical modeling." However, due to the configuration of the Cooling Pond and the presence of land formations and berms located in the Cooling Pond, the storm surge and seiche evaluation was performed using simplified methods.

The HHA approach described in NUREG/CR-7046 was used to determine whether a seiche can result in significant flooding at BGS. 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 Cooling Pond,
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 Cooling Pond.

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

Hourly wind speed, wind direction, and pressure data are available from Midway International Airport which lies approximately 45 miles northeast of BGS. 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. Due to lack of detailed wind and pressure data that captures isolated non-convective weather systems such as squall lines, natural frequencies for these types of storm are not calculated.

#### 3.4.2.4 Natural Period of the BGS Cooling Pond

The natural period of a pond 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, calculated by spectral analyses or by using Merian's formula.

### 3.4.3 Results

As the result of the probable maximum storm surge and seiche analysis, the maximum water surface elevations in the pond, cause by these phenomena, are determined. These maximum water elevations include the wind setup and wave setup and are called still-water elevations. Additionally, the wave characteristics, such as wave length and wave period, are determined. The wave characteristics are used as input parameters into the wave runup computations performed in Calculation BRW-13-FUK-0173 Combined Effects Analysis (Fukushima), (Exelon 2014i).

#### 3.4.3.1 Probable Maximum Storm Surge Results

The critical fetch used to determine the PMSS is determined to be 1.36 miles. Storm surge resulting from a PMSS event would produce a maximum storm surge of 1.93 feet.

#### 3.4.3.2 Probable Maximum Seiche Results

A spectral analysis of wind speed data (1-hour recording time interval) for three historical non-convective historical events is performed by applying a FFT in Excel. The spectral analysis shows a dominating fundamental period of 85 hours. Wind speed data at a time interval of less than 1-hour would have been preferred for spectral analysis to capture potential oscillation periods of less than 1 hour in wind storms. However, data at less than a 1-hour recording time interval is not available from the nearby meteorological stations at this time.

Based on the spectral analysis performed for the pond, the fundamental water level mode (fundamental period) of the cooling pond is 14.76 minutes, which is significantly smaller than the fundamental period of the forcing mechanism of 85 hours. Since these values are far apart in time and are not in-phase, a conclusion is made that amplified seiche is not a plausible scenario for the BGS Cooling Pond.

## 3.5 Dam Assessment and Failure

The dam assessment and failure analysis is performed for the Kankakee River and the Mazon River watersheds in calculation Dam Failure and Assessment Analysis, BRW-13-FUK-0168 (Exelon 2014d). For dam assessment, the guidance for screening dams is from JLD-ISG-2013-01, "Guidance for Assessment of Flood Hazards Due to Dam Failure," (NRC 2013a) which divides the dams into four categories: inconsequential, noncritical, potentially critical and critical. Inconsequential dams are dams not owned by a Nuclear Power Plant (NPP) site and are identified by Federal or State agencies as having minimal or no adverse failure consequences beyond the property limits of the dam owner. Noncritical dams are shown by the dam assessment to have little impact on flooding at a NPP site. Potentially critical dams are shown by the dam assessment to potentially have an impact at a NPP site. A detailed analysis will show which of the potentially critical dams are critical. Critical dams are those dams whose failure, either alone or combined as part of a multiple dam failure scenario, would cause inundation of a NPP site. "Guidance for Assessment of Flooding Hazards Due to Dam Failure" provides four increasingly refined methods

to assist in classifying dams as either potentially critical or noncritical. All assumptions are listed in Calculation BRW-13-FUK-0171 (Exelon 2014g).

### 3.5.1 Inputs

The inputs for the analysis are described below.

#### 3.5.1.1 Watersheds

The Kankakee River watershed was delineated in Calculation DRE12-007, Dresden Kankakee River Watershed PMP Analysis HMR-52. The Mazon River watershed was delineated in Calculation BRW-13-FUK-0165, All-Season Probable Maximum Precipitation Analysis (Fukushima) (Exelon 2014a).

### 3.5.2 Methodology

The methodology used for the dam assessment and failure is determined in calculation Dam Failure and Assessment Analysis (Fukushima) BRW-13-FUK-0168 (Exelon 2014d).

#### 3.5.2.1 Dam Assessment

For this calculation, the fourth method: Hydrologic Model Method is used to assess the impact of the failure of all dams upstream of Braidwood.

The steps necessary to assess which dams are critical using Method 4: Hydrologic Model Method are:

1) Determine the location, height and storage of all dams upstream of Braidwood.
2) Group dams into a subset of hypothetical dams.
3) Determine stage-storage relationships for the hypothetical dams.
4) Enter hypothetical dams into a HEC-HMS model and simulate breaching the dams using HEC-HMS.
5) Create a rating curve to determine the relationship between flow and water surface elevations at Braidwood.
6) Compare the estimated stage to the flood protection level of SSCs or plant grade, if appropriate.
7) If the estimated stage due to breaching of the hypothetical dams is over the protection level of SSCs or plant grade, dams are removed iteratively from the model, to the point where the predicted water surface elevation is lower than the flood protection level of the SSCs (or plant grade).

#### 3.5.2.2 Dam Failure

Based on the dam assessment, there are no potentially critical dams upstream of Braidwood along the Kankakee River. The Braidwood Station Cooling Pond Dam is assumed to be potentially critical. The next step is to simulate the failure of all the hypothetical dams and determine the effects on PMF flow rate and water surface elevations.

Potentially critical dams are failed when water surface elevation within the dam reservoir is at peak. Hypothetical dams are treated similarly. Some subbasins within the Kankakee River watershed contain multiple hypothetical dams. Instead of subdividing the subbasins into multiple smaller subbasins, the time that maximum water surface elevation occurs

within each hypothetical dam is approximated as the time that each subbasin reaches maximum flow. All hypothetical dams within a subbasin are failed at the time that the subbasin reaches maximum flow. Hypothetical dams are conservatively assumed to be full at the time that they fail.

The "Guidance for Assessment of Flooding Hazards Due to Dam Failure" suggests that three dam failure scenarios be considered:

1) Hydrologic Dam Failure
2) Seismic Dam Failure
3) Sunny-Day Dam Failure

Hydrologic dam failure considers the failure of all dams due to overtopping due to high flow. The result of the dam assessment is that there are no critical dams in either the Kankakee or Mazon River watersheds. The hypothetical dams are added to the HEC-HMS models.

The risk of seismic dam failure is unknown at Braidwood. It is conservatively assumed that the risk of seismic failure is high enough that all dams in the watershed upstream of Braidwood fail. No analysis of the likelihood of seismic failure of the dams within the watershed was performed. Per the "Guidance for Assessment of Flooding Hazards Due to Dam Failure", if seismic failure is simply assumed without analysis, the seismic failure should be assumed to occur under 500-year flood conditions (or ½ PMF, whichever is less).

A sunny-day dam failure requires failing a single dam to determine the effects. Both the hydrologic dam failure and seismic dam failure calculation consider the failure of all dams within the watershed. Therefore, both the hydrologic and seismic dam failure scenarios will envelope the sunny-day dam failure scenario. Sunny-day dam failure is not examined for that reason.

### 3.5.3 Results

The results of the dam assessment, based on Method 4, are that none of the 38 dams within the Kankakee River watershed are potentially critical. The BGS Cooling Pond is assumed to be potentially critical as it is the only dam within the Mazon River watershed.

For dam failure, only hydrologic dam failure and seismic dam failure were considered. It was conservatively assumed that both scenarios led to every dam in the watersheds upstream of Braidwood being failed. The hydrologic dam failure increases the PMF water surface elevation by 1.06 ft to 571.07 ft MSL for the Kankakee River and by 1.52 ft up to 594.25 ft MSL for the Mazon River. The water surface elevation due to the PMF plus selected dam failure for both rivers is below the site grade elevation of 600.0 ft MSL. Since the PMF stillwater elevation in the cooling pond of 599.36 ft is below the lowest top of dike elevation of 600.0 ft., hydrologic dam failure was not considered.

### 3.5.4 Conclusions

The UFSAR quantitatively and qualitatively ruled out incidence of dam failure affecting the site for the Kankakee and Mazon Rivers. The results of the re-evaluation based on updated guidance show that safety-related facilities will not affected by dam failure.

## 3.6 Combined Events Flood

Combinations from NUREG/CR-7046 are analyzed for the cooling pond and the Mazon River. Appendix H.4 states that the following combinations of flood causing events provide an adequate design basis for locations pertinent to BGS:

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

Appendix H.1 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.

Appendix H.2 Flooding Caused by Seismic Dam Failures

Alternative 1 – Combination of:

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

Alternative 2 – Combination of:

- The lesser of one-half of the PMF or the 500-year flood
- A flood caused by dam failure resulting from an operating basis earthquake (SSE), and coincident with the peak of the flood in item 1 above
- Waves induced by 2-year wind speed applied along the critical direction.

The H.4.1 and H.1 combinations are analyzed in Calculation BRW-13-FUK-0173 Combined Effects Analysis (Fukushima), (Exelon 2014i). Associated effects to the site are also examined in this calculation. All assumptions are listed in Calculation BRW-13-FUK-0173 (Exelon 2014i).

### 3.6.1 Inputs

The inputs for the analysis are described below.

#### 3.6.1.1 Probable Maximum Surge and Seiche

The PMSS and PMS are determined in section 3.4 of this report and detailed in Calculation BRW-13-FUK-0171 Surge, Seiche and Tsunami Analysis (Fukushima) (Exelon 2014g).

#### 3.6.1.2 Mazon River Probable Maximum Flood

The Mazon River PMF water surface elevation is determined in section 3.2 of this report and detailed in Calculation BRW-13-FUK-0172 Riverine Hydraulics Analysis (Fukushima) (Exelon 2014h).

#### 3.6.1.3 LiDAR Topography

The ground surface elevations are collected via LiDAR data performed by the Illinois Height Data Modernization program (Illinois 2004).

#### 3.6.1.4 Hydrologic Model for the Cooling Pond

The hydrologic model for the cooling pond (HEC-HMS) is developed in Calculation BRW-13-FUK-0176 Cooling Pond PMF Analysis (Fukushima), (Exelon 2014l). This model is used to determine maximum water level in the pond due to the PMF. The same hydrologic model is used to determine the water level in the pond due to other precipitation events for the H.4.2 combinations described above.

#### 3.6.1.5 100-Year Rainfall

Appendix H.4.1 combination specifies that the lesser of the 100-year water surface level and the maximum controlled level in the pond should be used. The 100-year precipitations are determined from the NOAA Precipitation Frequency Data Server (NOAA 2013b) for the location of BGS.

#### 3.6.1.6 2-Year Wind Speed

The 2-year wind speed applicable to the location of BGS of 50 miles per hour is obtained from a generalized map presented in ANSI/ANS-2.8-1992.

#### 3.6.1.7 Fetch Length

The fetch length is measured using ArcGIS computer software using surface elevation data from the Illinois Height Modernization Program (Illinois 2004).

### 3.6.2 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.

#### 3.6.2.1 Appendix H.4.1 Combination – Cooling Pond

The combination is performed by determining the effective fetch up to the exterior dike of the cooling pond. The significant and maximum wave heights are calculated based on the drag coefficient, fetch length and wind friction velocity. The wave runup is calculated based on the maximum wave height. The 2% wave runup is calculated since the significant wave height is already adjusted to the maximum wave height as a conservative parameter. Only 2% of the possible waves can exceed this height. The wind setup is a function of the wind velocity, fetch distance and the average depth of the pond. The wave runup and setup and wind setup is added to the PMSS with the maximum controlled elevation as the initial water surface elevation.

#### 3.6.2.2 Appendix H.1 Combination – Cooling Pond

The combination is performed by determining the effective fetch up to the exterior dike of the cooling pond. The significant and maximum wave heights are calculated based on the drag coefficient, fetch length and wind friction velocity. The wave runup is calculated based on the maximum wave height. The 2% wave runup is calculated since the significant wave height is already adjusted to the maximum wave height as a conservative parameter. Only 2% of the possible waves can exceed this height. The wind setup is a function of the wind velocity, fetch distance and the average depth of the pond. The wave runup and setup and wind setup is added to the cooling pond PMF from the Combined Effects Analysis (Fukushima) calculation BRW-13-FUK-0173 (Exelon 2014i) used as the initial water surface elevation.

#### 3.6.2.3 Appendix H.1 Combination – Mazon River

The Appendix H.1 – Alternative 1 combination for the Mazon River is performed by determining the effective fetch at elevation 594.00 NAVD88, which is the closest approximated contour at the PMF elevation with dam failure of 594.25 MSL. The significant and maximum wave heights are calculated based on the drag coefficient, fetch length and wind friction velocity. The wave runup is calculated based on the maximum wave height. The 2% wave runup is calculated since the significant wave height is already adjusted to the maximum wave height as a conservative parameter. Only 2% of the possible waves can exceed this height. The wind setup is a function of the wind velocity, fetch distance and the average depth of the river along the fetch. The wave runup and setup and wind setup is added to the Mazon River PMF from the Riverine Hydraulics Analysis (Fukushima) calculation BRW-13-FUK-0172 (Exelon 2014h) used as the initial water surface elevation.

#### 3.6.2.4 Associated Effects

Hydrodynamic and hydrostatic loading was determined as well as wave impacts to the lake screen house, debris impacts, groundwater ingress, sediment deposition and erosion, flood duration and warning times.

### 3.6.3 Results

The critical combination of events for the cooling pond is the combination for the Appendix H.4.1, which results in the maximum water surface elevation combination of 602.15 ft MSL. The critical combination of events for the Mazon River is the combination for the Appendix H.1, which results

in the maximum water surface elevation combination of 596.51 ft MSL. All associated effects as described in section 3.6.2.4 were analyzed and not considered to have a significant impact on the plant site.

### 3.6.4 Conclusions

Even though the maximum water elevation, including runup, at the lake screen house is above the plant floor elevation (600.0 ft MSL), this does not result in a flooding hazard for the site because the pond water level is below the top of the exterior dike of 602.5 ft MSL (Exelon 1976). This dike separates the pond from the BGS site area. In the event that the dike structure fails during a stillwater PMF with a wind-wave event, any wave runup propagating north to the power block will be completely impeded by sets of concrete blocks and vehicle barriers that surround the site that will act to dissipate wave energy and prevent water from impacting any safety-related structures.

The analysis determines that the scenario in Appendix H.4.1 for the cooling pond is the governing scenario with an elevation of 602.15 ft MSL. This scenario bounds other combinations in Appendix H.1 for the Mazon River and the Cooling Pond and combination in Appendix H.2 for the Mazon River. For Mazon River, the H.2 scenario is completely bounded by H.1 scenario.

## 3.7 Ice-Induced Flooding

The ice-induced flooding is analyzed in the Calculation BRW-13-FUK-0169 Ice-Induced Flood and Channel Migration Analysis (Fukushima), (Exelon 2014e). There are no historical records available for the ice jams at the streams in the Mazon River watershed. However, ice jams have occurred in the same stream in the same region. Therefore, this phenomenon is plausible for the Mazon River watershed. It is determined that in the unlikely event of an ice jam in the Mazon River, the resulting water surface elevation will be lower that the all-season PMF water surface elevation. Therefore, the ice-induced flooding in the Mazon River is bounded by the PMF. All the other streams in close proximity to BGS are not a part of the BGS watershed and will not cause flooding hazard.

An ice jam on the Kankakee River is possible. There are records of ice jams along the river in proximity to the site. It is determined that in the unlikely event of an ice jam in the Kankakee River, the resulting water surface elevation will be lower that the all-season PMF water surface elevation. Therefore, the ice-induced flooding in the Kankakee River is bounded by the PMF. All assumptions are listed in Calculation BRW-13-FUK-0169 (Exelon 2014e).

## 3.8 Channel Migration

The channel migration possibility for the small streams in the vicinity of BGS is analyzed in the Calculation BRW-13-FUK-0169 Ice-Induced Flood and Channel Migration Analysis (Fukushima), (Exelon 2014e). As described in UFSAR, there is no historical or topographic evidence indicating that flow in the Kankakee and Mazon River can be diverted away from its present course. Currently, the conditions of both rivers are still the same, so channel migration of the river towards the site is still not likely. Based on comparison of historical topographic maps and present-day topographic map the streams have been at the same approximate location for the past 94 years. All assumptions are listed in Calculation BRW-13-FUK-0169 (Exelon 2014e).

## 3.9 Tsunami

Tsunami phenomenon in relationship to BGS is analyzed in Calculation BRW-13-FUK-0171 Storm Surge, Seiche and Tsunami Analysis (Fukushima) (Exelon 2014g). A tsunami is a series of water waves generated by a rapid, large-scale disturbance of a water body due to seismic, landslide, or

volcanic tsunami-genic sources. Therefore, only geophysical events that release a large amount of energy in a very short time into a water body generate tsunamis. The most frequent cause of tsunamis is an earthquake. Less frequently, tsunamis are generated by submarine and sub-aerial landslides.

The National Oceanic and Atmospheric Administration (NOAA) natural hazards tsunami database (NOAA 2013a) identifies no known tsunami causing earthquake events at the BGS. The U.S. Geological Survey Earthquake Hazards Program hazard fault database (USGS 2013a) contains no known earthquake greater than magnitude 6.5 in the Eastern or Central United States. Moreover, earthquake hazard level in the region of the BGS shows very small probability of ground shaking. As a result, the required level of seismic activity for development of a tsunami, i.e., an earthquake with a magnitude greater than 6.5, is absent from the region. Therefore, a tsunami wave is not an applicable flooding scenario at BGS. Moreover, as an inland site, BGS is not susceptible to oceanic tsunamis. All assumptions are listed in Calculation BRW-13-FUK-0171 (Exelon 2014g).

## 3.10 Error/Uncertainty

The analysis calculation evaluates the errors and uncertainties associated with the Effects of LIP and Cooling Pond PMF calculations. The flooding due to the LIP event and in the cooling pond at are the controlling flood hazard mechanisms because they result in the highest water surface elevations closest to the site.

The other flooding mechanisms do result in lower water surface elevations compared to the controlling flood hazard. Therefore, the error/uncertainty calculation for those analyses is not performed.

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

- Error/uncertainty associated with the selection of the Manning's roughness coefficients

The error and/or uncertainties accompanying the Cooling Pond PMF analysis are:

- Error/uncertainty associated with the selection of the constant loss rate

All assumptions are listed in Calculation Error/Uncertainty Analysis (Fukushima) BRW-13-FUK-0175 (Exelon 2014k). The errors/uncertainties are described in detail below.

### 3.10.1 Inputs

The inputs for the analysis are described below.

#### 3.10.1.1 Manning's Roughness Coefficients – Effects of LIP Analysis

The selected coefficients for the Manning's n-values for the vector grids are taken from the calculation Effects of LIP Analysis BRW-13-FUK-0177 (Exelon 2014m).

#### 3.10.1.2 Constant Loss Rate – Cooling Pond PMF Analysis

The selected constant loss rates for the hydrologic model are taken from the calculation Cooling Pond PMF Analysis BRW-13-FUK-0176 (Exelon 2014l).

### 3.10.2 Methodology

#### 3.10.2.1 Manning's Roughness Coefficients – Effects of LIP Analysis

The selected coefficients for the Manning's n-values are generally in the median range of acceptable n-values for overland flow. The 2-dimensional flow model was run with the lower and upper range of the Manning's n-values.

#### 3.10.2.2 Constant Loss Rate – Cooling Pond PMF Analysis

The selected constant loss for the cooling pond is the average value of saturated losses based on soil type and land cover. The hydrologic flow model was run with the lower and upper range of constant loss rates. The resulting highest PMF water surface elevation from both the lower and upper limit is then evaluated for the combination from Appendix H.4.1 in NUREG/CR-7046 as specified in the Combined Effects calculation BRW-13-FUK-0173 (Exelon 2014i).

### 3.10.3 Results

The highest water surface elevation resulting from the 2-dimensional model runs with the lower and upper limit of the Manning's n-values raises the peak elevation by 0.01 ft to 601.68 ft MSL. The highest water surface elevation resulting from the HEC-HMS hydrologic model runs with the lower and upper limit of the constant loss rate raises the peak elevation in combination H.4.1 in NUREG/CR-7046 by 0.02 ft to 601.44 ft MSL.

### 3.10.4 Conclusions

The error/uncertainty analysis of parameters related to calculations with the lowest margin to site SSCs indicates that the peak water surfaces will not negatively impact these structures.

## 4. FLOOD PARAMETERS AND COMPARISON WITH CURRENT DESIGN BASIS

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

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 2012a) 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 (NRC 2012a))
   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)

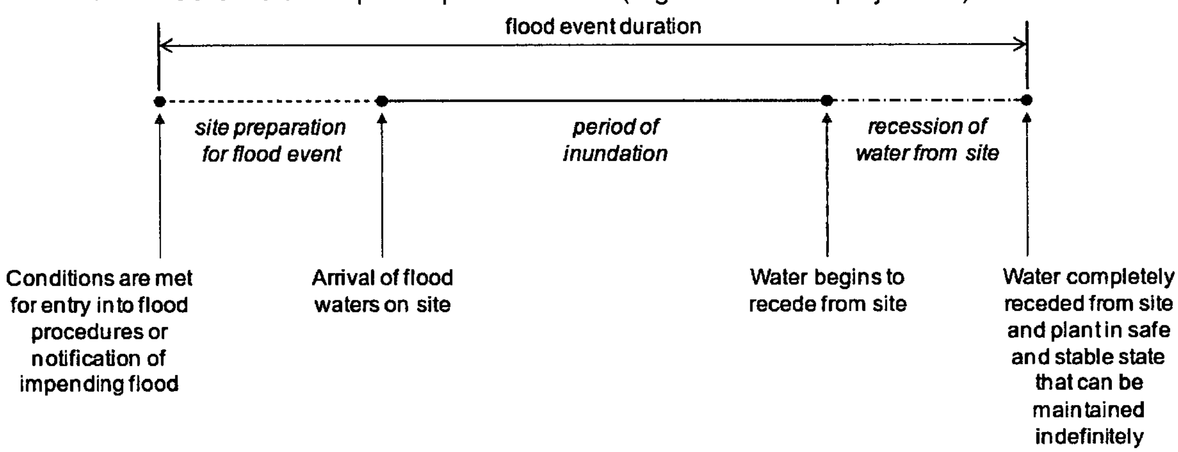


Figure 4.1- Illustration of Flood Event Duration (from Figure 6 of JLD-ISG-2012-05 (NRC 2012a))

Per Section 5.2 of JLD-ISG-2012-05 (NRC 2012a), 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 Braidwood, the following flood-causing mechanisms were either determined to be implausible or completely bounded by other mechanisms:

1. Kankakee River and Granary Creek (and associated upstream dam failure) flooding,
2. Tsunami;
3. Ice Induced Flooding;
4. Channel Migration or Diversion; and
5. Seiche,
6 Combined Effect Flood H.2 (from Appendix H of NUREG/CR-7046) for Mazon River, and
7 Combined Effect Flood H.2 (from Appendix H of NUREG/CR-7046) for Cooling Pond

Braidwood 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 (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, including hydrologic dam failure) for the Mazon River

3. Combinations in Section H.1 of NUREG/CR-7046 (Floods Caused by Precipitation Events) for the Cooling Pond

4. Combinations in Section H.4.1 of NUREG/CR-7046 (Floods along the Shores of Enclosed Bodies of Water, Shore Location) for the Cooling Pond.

Tables 4.1 through 4.5 summarize the parameters for each flood hazard and provide comparisons with the current design basis flood.

***Table 4.1 - 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, U.S. Bureau of Reclamation | HMR 51 and HMR 52 |
| Storm Duration | 48 hours | 72 hours |
| Cumulative PMP | 31.90 in | 27.71 in |
| Probable Maximum Flood on Cooling Pond | | |
| Hydrologic Model | Unknown | HEC-HMS |
| Total area | 5.3 sq.mi. | 5.59 sq. mi. |
| Lake Modeling Methodology | Stage-Storage | Stage-Storage |
| Discharge | Service and auxiliary spillways | Auxiliary spillway only |
| Probable Maximum Flood – Kankakee River | | |
| Hydrologic Model | Unknown | HEC-HMS |
| Total area | 5,150 sq.mi. | 5,146.82 sq. mi. |
| River Modeling Methodology | Step Backwater | HEC-RAS |
| Probable Maximum Flood – Mazon River | | |
| Hydrologic Model | Unknown | HEC-HMS |
| Total area | 220 sq.mi. | 216.07 sq. mi. |
| River Modeling Methodology | Step Backwater | HEC-RAS |
| Wind Wave Activity coincident with PMF on Lake | | |
| Methodology | Unknown | USACE Coastal Engineering Manual and ANSI/ANS-2.8-1992 |
| Wind speed | 40 mph | 100 mph |
| Wave parameters determination | Unknown | Hand Calculation |
| Wave setup and runup (ft)<br>Cooling Pond | 4.17 | 4.47 |
| Mazon River | Not analyzed | 2.15 |
| Local Intense Precipitation | | |
| Methodology | HMR 33, U.S. Bureau of Reclamation | HMR 51 and 52 |
| 1-Hour LIP (inches) | 17.80 | 17.84 |
| 5-min Peak Intensity (inches) | 5.98 | 6.00 |
| Effects Local Intense Precipitation | | |
| Methodology | Rational Method, Weir Flow, HEC-RAS Backwater Step Calculation | Hydrodynamic Modeling using FLO-2D Computer Software |
| Probable Maximum Surge and Seiche | | |
| Model | Not Evaluated | Hand Calculation |
| Combined Events (precipitation events on the lake combined with surge and seiche events) | | |
| Methodology | Not Evaluated | Hand Calculation |

***Table 4.2 - 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) | 601.85 | 601.67 | B |
| | 2. Max Wave Run-up Elevation (ft. MSL) | n/a | n/a | n/a |
| | 3. Max Hydrodynamic/Debris Loading (lb/ft) | n/a | 0.82 | B |
| | 4. Effects of Sediment Deposition/Erosion | n/a | n/a | B |
| | 5. Concurrent Site Conditions | n/a | n/a | n/a |
| | 6. Effects on Groundwater | n/a | n/a | n/a |
| Flood Event Duration | 7. Warning Time (hours) | n/a | n/a | n/a |
| | 8. Period of Site Preparation (hours) | n/a | n/a | n/a |
| | 9. Period of Inundation (hours) | n/a | n/a | n/a |
| | 10. Period of Recession (hours) | n/a | n/a | n/a |
| Other | 11. Plant Mode of Operations | Any | Any | B |
| | 12. Other Factors | n/a | n/a | n/a |

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

1. The re-evaluated elevation is bounded by the current design basis.
2. Consideration of wind-wave action for the site 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. 2-dimensional modeling indicates that the maximum hydrodynamic force per foot of water depth is 0.82. The maximum hydrostatic force is 1.08 lbs./ft. 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 factor combinations which could lead to a hazard due to debris load.
4. Because of generally low velocities, ranging between 0 and 2 fps around the power block area, sediment transport is not expected to be an effect of LIP flooding. The maximum velocity in the power block area (2 fps) is well below permissible velocities for paved surfaces so erosion and localized scour is also not expected to be an effect of LIP flooding.
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 UFSAR indicates that the LIP flood will not have an appreciable effect on groundwater. Since the site around the power block is impervious cover and LIP is a short duration event, groundwater changes are not expected to occur.
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 re-evaluated peak water surface is bounded by the design basis. Current plant operations and procedures by the site will still govern.
12. There are no other factors, including waterborne projectiles, applicable to the LIP flood.

***Table 4.3 - Combinations in Section H.1 of NUREG/CR-7046 – Mazon River (w/ hydrologic dam failure)***

| | Flood Scenario Parameter | CDB | Reevaluated | Bounded (B) or Not Bounded (NB) |
|---|---|---|---|---|
| Flood Level and Associated Effects | 1. Max Stillwater Elevation (ft. MSL) | 582.0 | 594.25 | NB |
| | 2. Max Wave Run-up Elevation (ft. MSL) | 584.0 | 596.51 | NB |
| | 3. Max Hydrodynamic/Debris Loading (lb/ft) | n/a | n/a | n/a |
| | 4. Effects of Sediment Deposition/Erosion | n/a | n/a | n/a |
| | 5. Concurrent Site Conditions | n/a | n/a | n/a |
| | 6. Effects on Groundwater | n/a | n/a | n/a |
| Flood Event Duration | 7. Warning Time (hours) | n/a | n/a | n/a |
| | 8. Period of Site Preparation (hours) | n/a | n/a | n/a |
| | 9. Period of Inundation (hours) | n/a | n/a | n/a |
| | 10. Period of Recession (hours) | n/a | n/a | n/a |
| Other | 11. Plant Mode of Operations | n/a | n/a | n/a |
| | 12. Other Factors | n/a | n/a | n/a |

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

1. The re-evaluated elevation is not bounded by the current design basis. However, the re-evaluated flood is well below the plant finish floor elevation of 601 ft MSL. Additionally, the openings to all buildings housing safety-related facilities are protected by 15 in (1.25 ft) steel barriers up to an elevation 602.25 ft MSL.
2. The re-evaluated elevation is not bounded by the current design basis. However, the re-evaluated flood is well below the plant finish floor elevation of 601 ft MSL. Additionally, the openings to all buildings housing safety-related facilities are protected by 15 in (1.25 ft) steel barriers up to an elevation 602.25 ft MSL.
3. Debris loading is not considered due to the distance from the Mazon River to the site.
4. Sediment deposition and erosion from fluid motion are not considered to have an effect on the PMF in the Mazon River due to the distance from the site.
5. High winds could be generated concurrent to a PMF. However, manual actions are not required to protect the plant from a river PMF so this concurrent condition is not applicable.
6. Groundwater fluctuation from a PMF flood on the Mazon River is not expected to impact the site due to the distance from the river.
7. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters do not apply to the Mazon River PMF.
8. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters do not apply to the Mazon River PMF.
9. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters do not apply to the Mazon River PMF.
10. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters do not apply to the Mazon River PMF.
11. Plant operations will not be affected as the re-evaluated flood due to the distance from the Mazon River and the available margin to the site plant grade.
12. There are no other factors, including waterborne projectiles, which apply to the Mazon River PMF.

***Table 4.4 - Combinations in Section H.1 of NUREG/CR-7046 – Cooling Pond***

| | Flood Scenario Parameter | CDB | Reevaluated | Bounded (B) or Not Bounded (NB) |
|---|---|---|---|---|
| Flood Level and Associated Effects | 1. Max Stillwater Elevation (ft. MSL) | 598.17 | 599.36 | NB |
| | 2. Max Wave Run-up Elevation (ft. MSL) | 602.34 | 601.42 | B |
| | 3. Max Hydrodynamic/Debris Loading (lb/ft) | n/a | n/a | n/a |
| | 4. Effects of Sediment Deposition/Erosion | n/a | n/a | B |
| | 5. Concurrent Site Conditions | n/a | n/a | n/a |
| | 6. Effects on Groundwater | n/a | n/a | 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 | n/a | n/a | n/a |
| | 12. Other Factors | n/a | n/a | n/a |

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

1. The re-evaluated stillwater elevation is not bounded by the current design basis flood. However, the re-evaluated flood is below the plant finish floor elevation of 601 ft MSL and top of dike elevation of 602.5 ft MSL. Additionally, the openings to all buildings housing safety-related facilities are protected by 15 in (1.25 ft) steel barriers up to an elevation 602.25 ft MSL.
2. The re-evaluated wind-wave runup elevation is bounded by the current design basis. flood.
3. Debris loading is not considered since the stillwater elevation is completely contained within the cooling pond exterior dike and the pond will have very low velocities that preclude significant debris loading effects. Also dynamic loads from wave impact were not evaluated because wind-wave is bounded by the CDB.
4. The pond exterior dike is protected from erosion with rock boulders to resist dike erosion from stillwater and/or wind-wave action.
5. High winds could be generated concurrent to a PMF. However, manual actions are not required to protect the plant from the Cooling Pond PMF so this concurrent condition is not applicable.
6. The UFSAR indicates that seepage from the Cooling Pond shall have minimal effect on groundwater levels at the site since the slurry trench along the north and west sides of the Pond act as a cutoff of potential seepage for the entire pond.
7. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters are not applicable to the Cooling Pond PMF.
8. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters are not applicable to the Cooling Pond PMF.
9. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters are not applicable to the Cooling Pond PMF.

10. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters are not applicable to the Cooling Pond PMF.
11. Plant operations will not be affected as the re-evaluated peak water surface with wind-wave effects is bounded by the current design basis.
12. There are no other factors, including waterborne projectiles, which apply to the Cooling Pond PMF.

***Table 4.5 - Combinations in Section H.4.1 of NUREG/CR-7046 – Cooling Pond***

| | Flood Scenario Parameter | CDB | Reevaluated | Bounded (B) or Not Bounded (NB) |
|---|---|---|---|---|
| Flood Level and Associated Effects | 1. Max Stillwater Elevation (ft. MSL) | 598.17 | 597.68 | B |
| | 2. Max Wave Run-up Elevation (ft. MSL) | 602.34 | 602.15 | B |
| | 3. Max Hydrodynamic/Debris Loading (lb/ft) | n/a | n/a | n/a |
| | 4. Effects of Sediment Deposition/Erosion | n/a | n/a | B |
| | 5. Concurrent Site Conditions | n/a | n/a | n/a |
| | 6. Effects on Groundwater | n/a | n/a | 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 | n/a | n/a | n/a |
| | 12. Other Factors | n/a | n/a | n/a |

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

1. Flooding due to combination listed in H.4.1 (NUREG/CR-7046) was not considered as part of the CDB. The re-evaluated flood from H.4.1 scenario is bounded by the CDB for Cooling Pond PMF scenario. The maximum stillwater elevation is the maximum controlled level in the pond with the effects of storm surge.
2. The re-evaluated wind-wave runup elevation is bounded by the by CDB for Cooling Pond PMF scenario.
3. Debris loading is not considered since the stillwater elevation is completely contained within the cooling pond exterior dike and the pond will have very low velocities that preclude significant debris loading effects. Also dynamic loads from wave impact were not evaluated because wind-wave is bounded by the CDB.
4. The pond exterior dike is protected from erosion with rock boulders to resist dike erosion from stillwater and/or wind-wave action.
5. High winds could be generated concurrent to a PMF. However, manual actions are not required to protect the plant from the Cooling Pond PMF so this concurrent condition is not applicable.
6. The UFSAR indicates that seepage from the Cooling Pond shall have minimal effect on groundwater levels at the site since the slurry trench along the north and west sides of the Pond act as a cutoff of potential seepage for the entire pond.
7. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters are not applicable to the Cooling Pond PMF.
8. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters are not applicable to the Cooling Pond PMF.
9. SSC's important to safety are currently protected by permanent/passive measures. Therefore, flood event duration parameters are not applicable to the Cooling Pond PMF.
10. SSC's important to safety are currently protected by permanent/passive measures.

Therefore, flood event duration parameters are not applicable to the Cooling Pond PMF.

11. Plant operations will not be affected as the re-evaluated peak water surface with wind-wave effects is bounded by the current design basis.
12. There are no other factors, including waterborne projectiles, which apply to the Cooling Pond Storm Surge flooding.

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**Enclosure 2**

Braidwood Generating Station, Units 1 and 2
Limited Integrated Assessment for External Flooding Report
Revision 0

(54 pages, including attachments)

# Braidwood Generating Station Units 1 &2

## Limited Integrated Assessment for External Flooding

**Response to Request for Information Pursuant to Title 10 of CFR 50.54(f) Regarding Recommendation 2.1 of the enclosure to SECY-11-0093, "Recommendations for Enhancing Reactor Safety in the 21st Century, the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident"**

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## Contents

## List of Tables



## List of Attachments

## Acronyms and Abbreviations

| | |
|---|---|
| ADAMS | Agencywide Documents Access & Management System |
| APM | Available Physical Margin |
| CFR | Code of Federal Regulation |
| CLB | Current Licensing Basis |
| COL | Combined Operating License |
| cfs | Cubic Feet per Second |
| EPP | Emergency Preparedness Procedure |
| Fig. | Figure |
| ft | Feet |
| FHRR | Flooding Hazard Reevaluation Report |
| FSAR | Final Safety Analysis Report |
| FWR | Flooding Walkdown Report |
| HUC | Hydrological Unit Code |
| ISG | Interim Staff Guidance |
| JLD | Japan Lessons-Learned Project Directorate |
| LIA | Limited Integrated Assessment |
| LIP | Local Intense Precipitation |
| MSL | Mean Sea Level |
| NAVD-88 | North American Vertical Datum of 1988 |
| NDE | Nominal Design Elevation |
| NRC | Nuclear Regulatory Commission |
| NSCW | Nuclear Service Cooling Water |
| NTTF | Near-Term Task Force |
| NWS | National Weather Service |
| PMF | Probable Maximum Flood |
| PMP | Probable Maximum Precipitation |
| psi | Pounds per Square Inch |
| SRM | Staff Requirements Memoranda |
| SSC | System, Structures, and Components |
| USGS | United States Geological Survey |
| VBS | Vehicle Barrier System |

# 1 Overview

Following the accident at the Fukushima Dai-ichi nuclear power plant resulting from the 2011 Great Tohoku Earthquake and tsunami, the Nuclear Regulatory Commission (NRC) established the Near-Term Task Force (NTTF) and tasked it with conducting a systematic and methodical review of NRC processes and regulations to determine whether improvements are necessary.

The resulting NTTF report concludes that continued U.S. nuclear plant operation does not pose an imminent risk to public health and safety and provides a set of recommendations to the NRC. The NRC directed the staff to determine which recommendations should be implemented without unnecessary delay (Staff Requirements Memorandum [SRM] on SECY-11-0093) (Reference 4).

Based on the NTTF Recommendations 2.1 and 2.3, the NRC issued its request for information pursuant to 10 CFR 50.54(f) on March 12, 2012 (Reference 1). Enclosure 2 of the NRC 10 CFR 50.54(f) letter addresses Recommendation 2.1 for the following purposes:

1. To gather information with respect to NTTF Recommendation 2.1, as amended by the SRM associated with SECY-11-0124 (Reference 6) and SECY-11-0137 (Reference 7), and the Consolidated Appropriations Act, for 2012 (Pub Law 112-74), Section 402 (Reference 8), to reevaluate seismic and flooding hazards at operating reactor sites and sites having a construction permit or 10 CFR 52 (Reference 9) combined license.
2. To collect information to facilitate NRC's determination if there is a need to update the design basis and systems, structures, and components (SSCs) important to safety to protect against the updated hazards at operating reactor sites.
3. To collect information to address Generic Issue 204 (Reference 10) flooding of nuclear power plant sites following upstream dam failures.

Recommendation 2.1 (Enclosure 2 of the NRC 10 CFR 50.54(f) letter) contains a "Requested Information" section detailing two items being requested from each licensed reactor site. The first requested item is the Flooding Hazard Reevaluation Report (FHRR) (Reference 2).

The second requested item of Recommendation 2.1 is an Integrated Assessment (IA) report, if required. Enclosure 2 of the NRC 10 CFR 50.54(f) letter addresses the situation in which an Integrated Assessment should be provided and the information the Integrated Assessment should contain. An Integrated Assessment is required for plants where the current design basis floods do not bound the reevaluated hazard for all flood causing mechanisms. The NRC Japan Lessons-Learned Project Directorate, JLD-ISG-2012-05, Guidance for Performing the Integrated Assessment for External Flooding, dated November 30, 2012 (ISG) (Reference 11).

On December 3, 2012, the NRC issued a letter regarding trigger conditions for performing an integrated assessment ('trigger letter' from David Skeen to NEI Executive Director) (Reference 5). The trigger letter

identifies four Integrated Assessment approach scenarios that are possible based on the results of the flood hazard reevaluation. The four possible scenarios are:

- Scenario 1 - Reevaluated Flood Hazard Bounded by Design Basis;
- Scenario 2 - Only Local Intense Precipitation;
- Scenario 3 - All Permanent and Passive Flood Protection; and
- Scenario 4 - Integrated Assessment Required.

An Integrated Assessment is not necessary for Scenario 1. A limited Integrated Assessment evaluation that only addresses specific sections of the ISG is required under Scenarios 2 and 3, in which case the limited assessment should be submitted with the FHRR. If Scenarios 1 through 3 do not apply, then, per Scenario 4, a full Integrated Assessment in accordance with the ISG is required.

The results of the flood hazard reevaluation for Braidwood Station, Units 1 & 2 (Braidwood), do not bound the current design basis flood for all applicable flood-causing mechanisms. However, since all flood protection features for Braidwood Units 1 & 2 are permanent and passive, Scenario 3 (above) is applicable and will be used to satisfy the Integrated Assessment requirements in Enclosure 2 of the 50.54(f) letter. Per the trigger letter, under Scenario 3, a licensee needs to show that the existing flood protection is reliable and has margin based on the reevaluated hazard. The trigger letter further states that the limited evaluation under Scenario 3 (referred to in this report as 'Limited Integrated Assessment' or 'LIA') should be performed using Section 6 of the Integrated Assessment ISG, including appropriate considerations described in Appendix A of the ISG and present-day codes and standards. The trigger letter also states that the results of this evaluation should be submitted with the hazard report. If the results of the evaluation do not show that the flood protection is reliable and has margin, a full integrated assessment is necessary, and should be submitted within 2 years of submitting the hazard report.

The Braidwood FHRR determined two non-bounded combined-effects flood scenarios: 1) Combinations in Section H.1 of NUREG/CR-7046 with Hydrologic Dam Failure for the Mazon River; and 2) Combinations in Section H.1 of NUREG/CR-7046 for the Cooling Pond. The LIA, discussed further below, was developed for these two non-bounding flood scenarios.

# 2 Limited Integrated Assessment Procedure

The content for this LIA report was developed to meet the requirements of NTTF Recommendation 2.1, per the guidance set forth by the NRC in the Japan Lessons-Learned Project Directorate (JLD) - Interim Staff Guidance (ISG)-2012-05, "Guidance for Performing the Integrated Assessment for External Flooding," Revision 0 and the trigger letter. Specifically, per the trigger letter, Section 6, "Evaluation of Effectiveness of Flood Protection," and Appendix A, "Evaluation of Flood Protection," of JLD-ISG-2012-05 were followed in performing this LIA. Figure 3 "Flood Protection Evaluation Process Flowchart" from the JLD-ISG-2012-05 was followed in conducting this evaluation (See Attachment 1 for ISG Figure 3)

# 3 Site Information Related to Flooding

The site is located about 4 miles southwest of the Kankakee River near the town of Custer Park in a strip-mined region presently characterized by many water-filled trenches and ponds. Cooling water for the plant is supplied by a cooling pond which covers one of these strip-mined areas. The pond has an average depth of approximately 8.21 feet at its normal pool elevation of 595 ft MSL, with a surface area of 2475 acres or 3.87 $mi^2$ and a storage volume of 22,300 acre-feet at normal pool elevation. The pond is contained by dikes having a top elevation of 600 ft MSL, except for that portion of the dike just south of the plant (northern-most portion of the dike), which has a top elevation of 602.5 ft MSL. The dike system is not a Seismic Category I structure.

The essential service cooling pond (ESCP) is located in the northwestern corner of the cooling pond in an area excavated below the surrounding pond bottom, to an elevation of 584 feet. The ESCP has a surface area of 99 acres and a depth of 6.0 feet at a pool elevation of 590.0 feet.

Makeup water for the pond is pumped from the river screen house on the Kankakee River via pipeline to the northeast corner of the cooling pond. Blowdown water is discharged from the plant by pipeline to the blowdown outfall structure to the discharge flume or multi-port diffuser spillway for release to the Kankakee River.

The Kankakee River is joined by Horse Creek at Custer Park. Horse Creek lies about 2.5 miles east of the site at its nearest point. The Mazon River flows northwest to the Illinois River. At its closest point, the Mazon River is joined by Granary Creek, 1 mile southwest of the site and about 4 miles south of the safety-related facilities. Crane Creek, a tributary of Granary Creek, flows north to meet Granary Creek about 1.5 miles south of the site. The flow in both creeks is intermittent. Floods on these small local streams and the Kankakee River would not affect safety-related portions of the plant.

The nearest highways to the site, Illinois State Routes 53 and 129, are adjacent to the northwest boundary of the site. Interstate 55 is less than 2 miles west-northwest of the site (centerline of the reactors), and State Route 113 is approximately 2 miles north of the site. Access to the plant is via State Route 53. Onsite roads in the immediate plant area vary in elevation from 598.0 ft MSL to 601.0 ft MSL. The Illinois Central Gulf Railroad runs parallel to and between State Routes 53 and 129 provides spur track access to the site.

The terrain around the plant site is relatively flat, with ground surface elevations varying from 595 ft to 605 ft MSL. Per the FSAR, the plant grade is at a nominal elevation of 600.0 ft MSL and all grade floors of safety-related buildings are at elevation 601.0 ft MSL.

The plant grade elevation (600.0 ft MSL) is 5 ft above the normal pool elevation in the cooling pond (595.0 ft MSL (UFSAR)). The site plant grade elevation is 21 ft above the mean water level in the Lake Michigan, equal to approximately 579 ft MSL (USACE 2014).

# 4 Flood Hazard Definition – Non-bounded Combined-Effect Flood Scenarios

As discussed above, the results of the flood hazard reevaluation for Braidwood Units 1 & 2 are not bounded by the current design basis floods. The FHRR determined two non-bounded combined-effect flood scenarios: 1) Combinations in Section H.1 of NUREG/CR-7046 with Hydrologic Dam Failure for the Mazon River; 2) Combinations in Section H.1 of NUREG/CR-7046 for the Cooling Pond.

Section 4.0 of the FHRR provides tables that define and describe the flood scenario parameters for the two non-bounded combined-effect floods that will be used in this LIA. The relevant flood scenario parameters are summarized below. (See FHRR Section 4.0 for detailed Flood Scenario Parameter tables and associated notes.)

**Table 4.1 – Combinations in Section H.1 of NUREG/CR-7046 (with Hydrologic Dam Failure) for Mazon River**

| Flood Scenario Parameter | Current Design Basis | Reevaluated | Bounded (B) or Not Bounded (NB) |
|---|---|---|---|
| **Max Stillwater Elevation (ft MSL)** | 582.00 | 594.25 | NB |
| **Max Wave Run-up Elevation (ft MSL)** | 584.00 | 596.51 | NB |

Table 4.2 – Combinations in Section H.1 of NUREG/CR-7046 for the Cooling Pond

| Flood Scenario Parameter | Current Design Basis | Reevaluated | Bounded (B) or Not Bounded (NB) |
|---|---|---|---|
| **Max Stillwater Elevation (ft MSL)** | 598.17 | 599.36 | NB |
| **Max Wave Run-up Elevation (ft MSL)** | 602.34 | 601.42 | B |
| **Effects of Sediment Deposition/Erosion** | n/a | n/a | B |
| **Effects of Groundwater** | n/a | n/a | B |

Several of the flood scenario parameters were determined to be not applicable. For example, all of the flood event duration parameters for both combined-effect floods were not applicable because safety-related SSC's are protected by permanent/passive features that do not require manual actions. (See FHRR Section 4.0 for detailed Flood Scenario Parameter tables and associated notes.)

The following sections provide a summary of the non-bounded combined-effect flood scenarios from the FHRR that were evaluated as part of this LIA. In general, information in Sections 4.1 and 4.2, below, was excerpted from the FHRR.

## 4.1 Combinations in Sec. H.1 of NUREG/CR-7046 (w/ Hydrologic Dam Failure) – Mazon River

This section provides a summary of the assessment of the combined-effect flood for the Mazon River, as described in the FHRR. The PMF 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 the PMP and other hydrologic factors favorable for maximum flood runoff such as sequential storms and snowmelt. For Braidwood, the Kankakee River and Mazon River with Granary Creek are the pertinent river systems evaluated in the UFSAR and reevaluated in the FHRR.

As outlined in the guidance provided in ANSI/ANS-2.8-1992 and in NUREG/CR-7046, Appendix H, Section H.1, three alternative combinations are examined:

- 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.

Alternative 1 (all-season PMP) resulted in the highest peak flows in the Mazon River Watershed and was used for the hydraulic analysis to determine the peak water surface elevation along the Mazon River. Peak flood levels in the Mazon River exceeded those in Granary Creek and the Kankakee River so only the Mazon River flooding was used to establish the flood scenario parameters. The PMF peak flow in the Mazon River was calculated to be 178,475 cfs. The maximum PMF stillwater elevation in the Mazon River was calculated to be 594.25 ft MSL. Applying the 2-year wind speed, the peak wind-wave runup elevation was calculated to be 596.51 ft MSL.

## 4.2 Combinations in Sec. H.1 of NUREG/CR-7046 – Cooling Pond

This section provides a summary of the assessment of the combined-effect flood for the cooling pond, as described in the FHRR. The PMF is the hypothetical flood (peak discharge, volume and hydrograph shape) that is considered to be the most severe reasonably possible, based on comprehensive

hydrometeorological application of the PMP 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, Section H.1, three alternative combinations are examined:

- 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.

Alternative 1 (all-season PMP) resulted in the highest peak stillwater levels in the cooling pond. The maximum PMF stillwater elevation in the Cooling Pond was calculated to be 599.36 ft MSL. Applying the 2-year wind speed, the peak wind-wave runup elevation was calculated to be 601.42 ft MSL.

# 5 Evaluation of Flood Protection Features

This section describes the evaluation of flood protection features credited with providing protection against the non-bounded combined-effect flood hazards. The section also provides an overview of each of the non-bounded flood scenario parameters, describing and summarizing key data associated with the determination that the hazard was non-bounded by the current design basis. The overview is followed by a description of the flood protection feature(s) and associated failure modes for each flood hazard, including three key subsections: 1) performance criteria; 2) flood protection evaluation; and 3) flood protection performance justification.

Section 5 of this report is organized as follows:

**Section H.1 Combined-Effect Flood for the Mazon River with Hydrologic Dam Failure (Section 5.1)**:

- Overview of flood scenario parameters for this hazard

- Flood Protection Feature 1 - Site topography and grading
  - Identification of failure modes
    - Performance criteria, flood protection evaluation and flood protection performance justification subsections.

**Section H.1 Combined-Effect Flood for the Cooling Pond (Section 5.2):**

- Overview of flood scenario parameters for this hazard
- Flood Protection Feature 1 - Site topography, grading, and cutoff/slurry wall
  - Identification of failure modes
    - Performance criteria, flood protection evaluation and flood protection performance justification subsections.
- Flood Protection Feature 2 – Northern Dike System
  - Identification of failure modes
    - Performance criteria, flood protection evaluation and flood protection performance justification subsections.
- Flood Protection Feature 3 - Protection against ingress through essential service water discharge and circulating water discharge pipe pathways.
  - Identification of failure modes
    - Performance criteria, flood protection evaluation and flood protection performance justification subsections.

To assess reliability and margin, each of the flood protection features identified above were evaluated based on the following three key items:

- Performance Criteria: This subsection describes both qualitative and/or quantitative criteria of the feature to protect against water ingress into a safety-related SSCs. Per the ISG, aspects such as the load-bearing ratings, material and size of the feature, and the feature's condition from inspection are considered in this subsection. These aspects are not considered when the flood protection feature is not challenged by the flood water.

- Flood Protection Evaluation: This subsection describes the feature's ability to protect against the bounding flood parameters at the site. Overall, the soundness of the flood protection features are demonstrated by confirming the features are in satisfactory condition, higher than the reevaluated flood height, and structurally adequate based on quantitative engineering evaluations and existing data. Other aspects of JLD-ISG 2012-05, Section 6.2 and Appendix A are considered in this subsection.

- Flood Protection Performance Justification: This subsection describes the reasons why the credited flood protection features are capable of withstanding the flood height and associated effects for the bounding set of flood scenario parameters. Additionally, the section identifies the limiting margin associated with the individual flood protection features.

In accordance with ISG Figure 3 "Flood Protection Evaluation Process Flowchart," two sets of flood scenario parameters (those associated with Combinations in Section H.1 of NUREG/CR-7046 with Hydrologic Dam Failure for the Mazon River; and those associated with Combinations in Section H.1 of NUREG/CR-7046 for the Cooling Pond) were evaluated to determine whether all applicable flood protection systems have adequate reliability and margin. The following provides the detailed evaluation of each set of flood scenario parameters.

## 5.1 Section H.1 Combinations with Hydrologic Dam Failure for Mazon River

See Table 4.1, above, for a summary of the relevant flood scenario parameters for this combined effects flood.

### 5.1.1 Overview of Flood Scenario Parameters

As summarized in the Table 4.1, above, the maximum design basis Stillwater Elevation is 582.0-ft MSL and the maximum reevaluated Stillwater Elevation is 594.25-ft MSL. The maximum design basis Wave Run-up Elevation is 584.0-ft MSL and the maximum reevaluated Wave Run-up Elevation is 596.51-ft MSL. Therefore, this combined-effect flood scenario is not bounded by the current design basis for both stillwater and wind-wave runup elevations.

The terrain around the Braidwood Units 1 & 2 site is relatively flat, with ground surface elevations varying from 595-ft MSL to 605-ft MSL. The plant grade and floor elevations for all buildings/structures that house safety-related equipment are 600.0-ft MSL and 601.0-ft MSL, respectively. Additionally, although the finished floors for the safety-related structures are at elevation 601-ft MSL, the openings to all buildings housing safety-related facilities are protected by 15 in (1.25 ft) steel barriers up to elevation 602.25-ft MSL.

The feature protecting the site from the reevaluated flood hazard along the Mazon River, defined by the Section H.1 Combined-Effect Flood (with upstream dam failure), is 'site topography and grading'.

### 5.1.2 Flood Protection Feature 1 - Site Topography and Grading

The site is protected from flooding along the Mazon River by being elevated above the reevaluated flood hazard. The nominal site grade elevation is 600-ft MSL and the peak flood elevation for the Section H.1 Combined-Effect Flood (with upstream dam failure) is 596.51-ft MSL, which provides 3.49 feet of nominal margin. Additional flood protection margin is provided by the higher elevation of the plant finished floor (601.0 ft MSL) and the 15 inch high steel barriers installed at each door of buildings housing safety-related equipment (602.25-ft MSL).

Table 5.1 provides critical plant elevations and margin for buildings housing safety-related SSCs.

**Table 5.1 – Critical Plant Design Elevations (for Mazon River Flooding)**

| Units 1 & 2 Plant Level | Elevation per FSAR (feet MSL) | Wind Wave Runup Margin (feet) |
|---|---|---|
| Plant Finished Grade | 600.00 | 3.49 |
| Plant Safety-Related SSCs Finished Floor | 601.00 | 4.49 |
| Plant Buildings Entry Ways | 602.25 | 5.74 |

The following failure modes were identified for site topography and grading:

- Groundwater ingress; and;
- Settlement.

#### *5.1.2.1 Performance Criteria*

The performance criteria listed in Section 6.2 of JLD-ISG-2012-05, along with related methodology of Appendix A, are not applicable for a site topography evaluation as the natural elements composing site elevation protect safety-related SSCs at the site grade elevation when margin exists above the controlling site flood elevation. However, performance criteria was developed for the two failure modes listed above (groundwater ingress and settlement). The performance criteria established for this flood protection feature are:

- Demonstrate, through review of the soil information and flood duration, that Mazon River flooding is not expected to affect groundwater levels at the plant.
- Demonstrate that settlement at the site has stabilized and is not expected to have any significant effect on margin for the life of the plant.

#### *5.1.2.2 Flood Protection Evaluation*

Groundwater Ingress:

FSAR Section 2.5.4.6 "Groundwater Conditions" states that for the design of safety-related plant structures, the groundwater level was assumed to be at plant grade, elevation 600-ft MSL. All subsurface and foundations are designed to withstand full hydrostatic loads.

FSAR Section 2.4.13.5 "Design Bases for Subsurface Hydrostatic Loading" also states that the design groundwater level for hydrostatic loading is elevation 600-ft MSL. The FSAR states that foundations of the main plant buildings are below the groundwater levels measured in the sand aquifer during site investigations and construction (Subsections 2.4.13.2.2.2 and 2.5.4.6 of the FSAR). Seepage into the main plant excavation was controlled during construction by a slurry wall cutoff installed through the sand aquifer and 2 feet into the underlying till. Precipitation and seepage into the excavation were removed by the use of sump pumps. No dewatering systems are used to permanently lower groundwater levels under safety- or non-safety-related buildings.

Section 2.4.13.2.2.1 "Permeability" of the FSAR states that permeability values for the various hydrogeologic units at the site were determined from laboratory tests on soil samples, field permeability tests conducted in the ESCP area, and water pressure tests in the bedrock.

Laboratory permeability test results, reported in Table 2.5-35 of the FSAR, show the permeability of the sand deposits to range from 3.66 X $10^{-4}$ cm/sec to 7.32 X $10^{-2}$ cm/sec. For the evaluation of seepage from the ESCP, an average value of 6 X $10^{-3}$ cm/sec was used. The average permeability of the till was found to be 2.6 X $10^{-6}$ cm/sec. For discontinuous, well-graded gravel and silts within the glacial drift at a depth of 35.5 to 40.5 feet in Borings H-1 and H-3, the permeability was found to average 8.4 X $10^{-4}$ cm/sec.

Water pressure tests were performed in the Pennsylvanian-age Carbondale and Spoon Formations and in the underlying Brainard Shale and Fort Atkinson Limestone of the Ordovician-age Maquoketa Shale Group. No water losses were recorded in 20% and 50% of the tested intervals in the Carbondale and Spoon Formations, respectively, or in 40% of the tested intervals in the Maquoketa Shale Group. In those intervals in which water losses were recorded, permeabilities ranged from 1.93 X $10^{-6}$ to 4.92 X $10^{-4}$ cm/sec in the Carbondale Formation, 1.76 X $10^{-6}$ to 6.20 X $10^{-4}$ cm/sec in the Spoon Formation, and 2.33 X $10^{-6}$ to 4.58 X $10^{-5}$ cm/sec in the Maquoketa Shale Group. The FSAR states that these permeability values probably reflect secondary permeability along infrequent joints and fractures within these formations rather than intergranular, primary permeability of the rock mass. In addition, the upper tested intervals of the boreholes generally had higher permeabilities than those at greater depths, probably reflecting the effects of weathering on the strata.

Per the FSAR Section 2.4.13.3 "Accident Effects," a cement-bentonite slurry trench was installed around the perimeter of the main plant excavation through the Parkland Sand and the Equality Formation into the silty clay glacial till of the Wedron Formation. The FSAR states that this trench would restrict any seepage into or out of the auxiliary building and, presumably, would restrict any potential seepage into the main plant area as well. Note that this slurry trench was not tested to determine its ability to limit groundwater ingress; this is not credited with providing primary protection, but, instead, is considered to provide defense-in-depth protection.

Based on the reevaluated flood hazard, the maximum Stillwater Elevation for this hazard is 594.24 ft MSL. Per the FSAR, the groundwater level was assumed to be at plant grade (elevation 600 ft MSL) for the design of safety-related plant structures and all subsurface and foundations are designed to withstand full hydrostatic loads. Additionally, the natural soil strata between the Mazon River and the plant and a slurry trench (installed around the perimeter of the main plant during construction) would restrict potential seepage into the main plant area from flooding along the Mazon River. Therefore, flooding along the Mazon River is not expected to affect groundwater ingress at the plant.

Settlement:

Section 2.5.4.10.1.1 of the FSAR addresses the predicted settlement of the Braidwood plant. Per the FSAR, because the plant is founded on over-consolidated till, bedrock, or granular fill, no significant settlement will be caused by dynamic loads. Coal mining was performed in the vicinity of the plant,

however, No. 2 Coal (Colchester Member) was encountered in all the borings drilled to sufficient depth to penetrate it (57 borings), indicating the absence of any mining activity in the plant area. As a result, the FSAR concludes that there is no possibility of collapse or subsidence due to mines.

A system of construction settlement monuments was established for the foundations of Category I structures during 1977 and 1979 as shown in Figure 2.5-263 of the FSAR. These monuments were installed and monitored by the contractor for the purpose of construction control and settlement monitoring. Seven of these monuments have been monitored continuously from the beginning of construction in 1977 to August 1980. Many of the other original monuments were discontinued because of construction interferences, and some were replaced in February 1979 by new monuments at similar locations within the same building. Other new monuments were also added at this time. In August 1980, monitoring was halted because settlement was complete under approximately 95% of the plant static load with measurements within the accuracy of the surveying equipment and methods used.

In September 1981, a new set of operational settlement monuments was established throughout the plant. The intent of monitoring these monuments was to provide additional data to show that plant settlement under full static load is complete. The new monuments were installed on two floor levels to reduce errors introduced into survey circuits by eliminating excessive traveling between different building levels.

The operational phase monuments, installed in September 1981, show that their maximum settlement measured through April 1988 was generally less than or equal to -0.01 feet (-0.012 feet maximum) except for Unit 2 containment monuments. The Unit 2 containment monuments numbers 41, 18, 17, R4, Z1 and Z show an average settlement of approximately -0.017 feet. This settlement is believed to be a result of small increases in dead load over the monitoring period and construction activities. Attachment 2 provides Tables 2.5-41 (Tabulated Differential Settlement for Survey Monuments) and 2.5-42 (Projected Maximum Total and Differential Settlements) from the FSAR.

The differential settlements given in Table 2.5-42 of the FSAR are all less than or equal to -0.03 feet. Per the FSAR, this is significantly less than 1/2-inch or more which was assumed in the design of the auxiliary building and fuel handling building.

The FSAR states that all Category I structures have been designed to account for the maximum total and differential settlement.

The lake screen house is founded within a very stiff to hard glacial till of the Wedron Formation. The till is over-consolidated and has an ultimate bearing capacity of approximately 45,000 psf (Subsection 2.5.4.10.1.2). The approximate static bearing pressure for the screen house is 3,000 psf resulting in a factor of safety of 15. The estimated settlement of the screen house is less than 1/4 inch total and 1/8 to 1/4 inch differential (per Subsection 2.5.4.10.2.2 of the FSAR).

Construction phase settlement monitoring was not performed for the lake screen house but has been included in the operational phase settlement monitoring.

Six operational phase settlement monuments have been installed in the lake screen house to provide data to show that settlement is complete. The results given in Table 2.5-41 of the FSAR show maximum settlement values less than or equal to -0.01 feet (+0.007 feet maximum). This movement is considered negligible and indicates that settlement has stabilized. Therefore, settlement is not expected to reduce margin above the reevaluated flood levels in the Mazon River.

#### *5.1.2.3 Flood Protection Performance Justification*

As demonstrated above, Braidwood Units 1 & 2 site topography, with a nominal grade elevation of 600-ft MSL, is adequately reliable in protecting the plant from flooding along the Mazon River. The available margin (3.49 feet) was determined to be adequate, with the following justifications:

- The hydrologic analysis for the Mazon River included the following conservatisms in developing PMF flow rates:
    - Fully saturated soils for constant loss rates;
    - Zero initial losses; and
    - Generalized non-linear response adjustments to the unit hydrograph.
- The available margin exceeds established criteria for uncertainties in the hydraulic model used to estimate flood levels. For example, Title 44 of the Code of Federal Regulations, Section 65.10 (44CFR65.10), specifies requirements for the accreditation of levee systems in the National Flood Insurance Program (NFIP). Paragraph (b)(1)(i) of 44CFR65.10 requires a minimum freeboard (i.e. margin) to the protection level (i.e. top of a levee, plant grade, etc.) of 3 feet to account for uncertainties (including statistical uncertainties in peak flow rates) in the stillwater profile estimation for a flood having a 1% chance of being equaled or exceeded in any given year (i.e. 1% chance or 100-year flood). Similar hydraulic modeling methods (i.e. HEC-RAS) were used for the Braidwood flood hazard reevaluation to calculate flood levels along the Mazon River as are typically used for NFIP flood insurance studies. The available margin exceeds this criteria and it's acceptability is compounded by two factors:
    - The reevaluated flood along the Mazon River was developed using deterministic methods. Therefore, statistical uncertainties are not relevant.
    - The NFIP criteria in 44CFR65.10 is associated with stillwater, whereas the available margin includes wind-wave runup.

## 5.2 Section H.1 Combinations for Cooling Pond

See Table 4.2, above, for a summary of the relevant flood scenario parameters for this combined effects flood.

### 5.2.1 Overview of Flood Scenario Parameters

As summarized in Table 4.2, above, the maximum design basis Stillwater Elevation is 598.17-ft MSL and the maximum reevaluated Stillwater Elevation is 599.36-ft MSL. The maximum Wave Run-up Elevation from the design basis is 602.34-ft MSL and the maximum reevaluated Wave Run-up Elevation is 601.42-ft MSL.

As stated above, the terrain around Braidwood Units 1 & 2 site is relatively flat, with ground surface elevations varying from 595-ft MSL to 605-ft MSL. The nominal plant grade and floor elevations are 600.0-ft MSL and 601.0-ft MSL, respectively (UFSAR). All safety-related SSCs at the site are housed within buildings/structures at grade elevation of 600.0-ft MSL (nominal) and finished floor elevation of 601.0-ft MSL. Additionally, although the floors for the safety-related structures are at elevation 601-ft MSL, the openings of all buildings housing safety-related facilities are protected by 15 in (1.25 ft) steel barriers up to elevation 602.25-ft MSL.

The cooling pond is impounded by an exterior dike with the top of the dike elevation varying from 600.0-ft MSL to 602.5-ft MSL (Exelon 1976). The top of northern dike, which separates the plant from the cooling pond, is at elevation 602.5 ft MSL. The pond-side slope of the northern dike is 3:1 with the pond floor at elevation 590.0 ft MSL. The plant-side grade of the northern dike is at elevation 600 ft MSL. (See FSAR Figures 2.4-47 and 2.4-48 provided as Attachment 3.)

As shown in Table 4.2, above, the maximum reevaluated Stillwater Elevation is not bounded by the current design basis flood. However, the maximum reevaluated Stillwater flood elevation of 599.3 ft MSL is below plant grade elevation 600.0 ft MSL, plant floor elevation 601.0 ft MSL, and top of dike elevation 602.5 ft MSL. Additionally, the openings to all buildings housing safety-related equipment are protected up to an elevation 602.25 ft MSL.

The features protecting the plant from the reevaluated Cooling Pond flood hazard, defined by the Section H.1 Combined-Effect Flood are:

- Site topography, grading, and slurry trench;
- Northern dike system; and
- Protection against ingress through Essential Service Water Discharge and Circulating Water Discharge pipe pathways.

### 5.2.2 Flood Protection Feature 1 - Site Topography, Grading, and Slurry Trench

The site is protected from Cooling Pond flooding by being elevated above the reevaluated stillwater level associated with this hazard. The nominal site grade elevation is 600-ft MSL and the peak flood elevation for the Section H.1 Combined-Effect Flood is 599.36-ft MSL, which provides 0.64 foot of nominal margin. Additional flood protection margin is provided by the higher elevation of the plant finished floor (601.0 ft MSL) and the 15 inch high steel barriers installed at each door of buildings housing safety-related equipment (602.25 ft MSL). Note that an earthen dike provides additional protection from Cooling Pond flooding, however, for the purposes of evaluating site topography/grading protection against stillwater levels, the dike will be ignored. A slurry trench was constructed along the Cooling Pond dike as a seepage

barrier. Table 5.2 provides critical plant elevations and margin associated with the Section H.1 Combination for the Cooling Pond for buildings housing safety-related SSCs.

**Table 5.2 – Critical Plant Design Elevations (for Cooling Pond Flooding)**

| Units 1 & 2 Plant Level | Elevation per FSAR (feet MSL) | Stillwater Margin (feet) |
|---|---|---|
| Plant Finished Grade | 600.00 | 0.64 |
| Plant Safety-Related SSCs Finished Floor | 601.00 | 1.64 |
| Plant Buildings Entry Ways | 602.25 | 2.89 |

The following failure modes are associated with site topography and grading:

- Groundwater ingress; and;
- Settlement.

#### 5.2.2.1 *Performance Criteria*

The performance criteria listed in Section 6.2 of JLD-ISG-2012-05, along with related methodology of Appendix A, are not applicable for a site topography evaluation as the natural elements composing site elevation protect safety-related SSCs when margin exists above the controlling stillwater elevation. However, performance criteria was developed for the two failure modes listed above (groundwater ingress and settlement). The performance criterion established for this flood protection feature are:

- Demonstrate, through review of the soil information and design properties of the slurry trench, that increased stillwater levels in the Cooling Pond is not expected to affect groundwater ingress at the plant.
- Demonstrate that settlement at the site has stabilized and is not expected to have any significant effect on margin for the life of the plant.

#### 5.2.2.2 *Flood Protection Evaluation*

Groundwater Ingress:

Section 2.4.13.2.3 "Effects of Seepage from Cooling Pond" of the FSAR states that seepage from the cooling pond should have minimal effect on groundwater levels around the site. Seepage to the Cambrian-Ordovician Aquifer is limited by the relatively impermeable Pennsylvanian-age shales of the Carbondale and Spoon Formations and by the Ordovician-age shales of the Maquoketa Shale Group. Seepage to the sand aquifer will be limited by a slurry trench cutoff, constructed through the cooling pond dike and generally extending 2 feet into the till or onto Pennsylvanian-age bedrock (See FSAR Figures 2.4-47 and 2.4-48 provided as Attachment 3 and Figure 2.4-35 provided as Attachment 4). Per the FSAR, the cooling lake perimeter dike slurry trench is continuous around the perimeter of the

cooling lake and, therefore, continuous along the north and west sides of the essential service cooling pond. The slurry trench along the ESCP is a soil-bentonite backfilled slurry trench extending from elevation 597 feet to the top of the till and, in most cases, is keyed into the till. The slurry trench will be a continuous seepage cutoff around the entire perimeter of the pond. However, the design of the ESCP does not rely on the slurry trench as a seepage barrier. The ESCP seepage has been conservatively determined (see Subsection 2.5.6 of the FSAR) assuming the trench does not exist.

As part of the original design evaluation for the slurry trench cutoff, a prototype slurry trench test section was constructed in the cooling pond area. The test consisted of several pumping tests to determine the average permeability of the in-place soil-bentonite and cement-bentonite backfill materials. The results of these tests were used in the design of the slurry trench cutoff and the cooling pond dike (Subsection 2.5.6). The maximum permeability values determined for the in-place slurry trench test section were as follows:

- Soil-bentonite (using natural onsite soil) $6.0 \times 10^{-7}$cm/sec
- Cement-bentonite $4.4 \times 10^{-6}$cm/sec

Based upon these permeability values, it was determined that the amount of seepage through the entire length of the cooling pond dike is estimated to be less than 5 cfs (Subsection 2.5.6 of the FSAR). Therefore, when considering the approximately 10-mile perimeter of the cooling pond, the effect on local groundwater levels in the sand aquifer should be very small and restricted to the immediate perimeter of the cooling pond. Therefore, a small increase in the reevaluated stillwater level is not expect to affect groundwater levels at the plant. Furthermore, the design basis assumes a groundwater level at plant grade (elevation 600 ft MSL) for the design of safety-related plant structures and all subsurface and foundations are designed to withstand full hydrostatic loads.

<u>Settlement</u>:

As discussed above, Section 2.5.4.10.1.1 of the FSAR addresses the predicted settlement of the Braidwood plant. Based on the FSAR, because the plant is founded on overconsolidated till, bedrock, or granular fill, no significant settlement will be caused by dynamic loads. Coal mining was performed in the vicinity of the plant, however, No. 2 Coal (Colchester Member) was encountered in all the borings drilled to sufficient depth to penetrate it (57 borings), indicating the absence of any mining activity in the plant area. The FSAR concludes that there is therefore no possibility of collapse or subsidence due to mines.

A system of construction settlement monuments was established for the foundations of Category I structures during 1977 and 1979 as shown in Figure 2.5-263 of the FSAR. These monuments were installed and monitored by the contractor for the purpose of construction control and settlement monitoring. Seven of these monuments have been monitored continuously from the beginning of construction in 1977 to August 1980. Many of the other original monuments were discontinued because of construction interferences, and some were replaced in February 1979 by new monuments at similar locations within the same building. Other new monuments were also added at this time. In August 1980, monitoring was halted because settlement was complete under approximately 95% of

the plant static load with measurements within the accuracy of the surveying equipment and methods used.

In September 1981, a new set of operational settlement monuments was established throughout the plant. The intent of monitoring these monuments was to provide additional data to show that plant settlement under full static load is complete. The new monuments were installed on two floor levels which will reduce errors introduced into survey circuits by eliminating excessive traveling between different building levels.

The operational phase monuments, installed in September 1981, show that their maximum settlement measured through April 1988 was generally less than or equal to -0.01 feet (-0.012 feet maximum) except for Unit 2 containment monuments. The Unit 2 containment monuments numbers 41, 18, 17, R4, Z1 and Z show an average settlement of approximately -0.017 feet. This settlement is believed to be a result of small increases in dead load over the monitoring period and construction activities. Attachment 2 provides Tables 2.5-41 (Tabulated Differential Settlement for Survey Monuments) and 2.5-42 (Projected Maximum Total and Differential Settlements) from the FSAR.

The differential settlements given in Table 2.5-42 of the FSAR are all less than or equal to -0.03 feet. Per the FSAR, this is significantly less than 1/2-inch or more which was assumed in the design of the auxiliary building and fuel handling building (FSAR).

The FSAR states that all Category I structures have been designed to account for the maximum total and differential settlement.

The lake screen house is founded within a very stiff to hard glacial till of the Wedron Formation. The till is over-consolidated and has an ultimate bearing capacity of approximately 45,000 psf (Subsection 2.5.4.10.1.2). The approximate static bearing pressure for the screen house is 3,000 psf resulting in a factor of safety of 15. The estimated settlement of the screen house is less than 1/4 inch total and 1/8 to 1/4 inch differential (per Subsection 2.5.4.10.2.2 of the FSAR).

Construction phase settlement monitoring was not performed for the lake screen house but has been included in the operational phase settlement monitoring.

Six operational phase settlement monuments have been installed in the lake screen house to provide data to show that settlement is complete. The results given in Table 2.5-41 of the FSAR show maximum settlement values less than or equal to -0.01 feet (+0.007 feet maximum). This movement is considered negligible and indicates that settlement has stabilized. Therefore, settlement is not expected to reduce margin above the reevaluated flood levels in the Cooling Pond.

#### *5.2.2.3 Flood Protection Performance Justification*

As demonstrated above, Braidwood Units 1 & 2 site topography, with a nominal grade elevation of 600-ft MSL, is adequately reliable in protecting the plant from flooding at the Cooling Pond. Furthermore, the reevaluated maximum stillwater elevation of 599.36-ft MSL is below the plant grade elevation 600-ft MSL. The design basis assumes a groundwater level at plant grade for the design of

safety-related plant structures and all subsurface and foundations are designed to withstand full hydrostatic loads.

The available margin (0.64 foot) was determined to be adequate, with the following justification. During the flood hazard reevaluation, a sensitivity analysis (Reference 16) was conducted for the PMF stillwater calculation for the cooling pond. Since the watershed consists largely of the cooling pond itself, routing and runoff transformation parameters are not relevant to the PMF stillwater calculation. The primary input parameter for the sensitivity assessment is the constant loss rate. As stated in Section 3.10.2.2 of the FHRR, the selected constant loss for the cooling pond is the average value of saturated losses based on soil type and land cover. The hydrologic model was run with the lower and upper range of constant loss rates. The resulting sensitivity calculations show that the stillwater level is accurate to +/- 0.02 foot, well below the available margin of 0.64 foot.

### 5.2.3 Flood Protection Feature 2 - Northern Dike System

The northern dike system (located on the south side of the plant, between the plant and the cooling pond) protects the plant site from cooling pond flooding; specifically from wind-generate waves. With a top elevation of 602.5-ft MSL and a maximum reevaluated wind-wave runup elevation of 601.42-ft MSL, the existing dike provides 1.08 feet of margin for the Section H.1 (NUREG/CR-7046) Combined-Effect flood. It should be noted that the Section H.4.1 (NUREG/CR-7046) Combined-Effect flood for the Cooling Pond, while completely bounded by the design basis flood, exceeded the Section H.1 Combined-Effect flood wind-wave runup elevation. Therefore, the wind-wave runup elevation for the Section H.4.1 Combined-Effect flood (602.15-ft MSL), which is below the design basis wind-wave runup elevation (602.34-ft MSL) by 0.19 foot, was considered in the evaluation of the northern dike system.

The following failure modes were considered applicable to the dike system:

- Sloughing/Slope Stability; and
- Erosion from wind-generated waves.

Other failure modes, such as seepage, internal erosion, and piping were not considered credible or applicable since the reevaluated stillwater level is below the nominal grade elevation of the plant side of the dike. Slope failure, due to erosion or sloughing, could cause head-cutting and breach the crest, allowing wind-generated waves to spill onto the site and, therefore, were evaluated.

#### *5.2.3.1 Performance Criteria*

Based on guidance in Section 6.2 and related methodology of Appendix A, Section A.1.1.1, of the ISG the criteria listed below were considered applicable in evaluating the performance of this feature. The reevaluated stillwater level being lower than nominal plant grade provided the basis for judging the applicability of the performance criteria in Section 6.2 of the ISG. That is, even if the interior dike slope failed due, for example, to erosion or sloughing, high stillwater in the cooling pond would not flood the plant since grade on plant side of dike is higher than the maximum reevaluated stillwater level.

- Demonstrate the stability of the interior slope (3:1) using appropriate, present-day design codes and standards.
- Demonstrate that erosion control against wave action were appropriately considered in the dike design.
- Demonstration that the maintenance and inspection regime of the dike was evaluated to assess whether:
  - The dike is inspected at regular intervals;
  - Written procedures are in place for proper maintenance; and
  - Personnel responsible for inspecting the dike have been trained in inspection techniques, implementing preventative and compensatory measures, and correcting or repairing deterioration.

*5.2.3.2 Flood Protection Evaluation*

Sloughing/Slope Stability:

To evaluate the potential for sloughing/slope stability of the northern dike, the National Resources Conservation Services (NRCS) *Conservation Practice Standard – Pond* Code 378 manual (Reference 12 and Attachment 5) was used. The section titled "Design Criteria for Excavated Embankments" identifies that side slopes of excavated ponds shall not be steeper than one horizontal to one vertical. With a much milder gradient, the cooling pond-side slope of 3:1 satisfies this standard for stability.

Erosion due to Wave-Action:

Per Section 2.4.8.2.6 "Coincident Wind Wave Activity" of the FSAR, the integrity and stability of the exterior dike A to the north of the essential cooling pond were analyzed by considering the maximum wave due to 40-mph wind on PMF pool with an antecedent SPF condition. The plant design assumed that the interior dike south of the essential cooling pond would not exist. The largest fetch resulting from this assumption is at location A (northern dike). Per the FSAR, the wave run-ups (including setups) were calculated based on a shallow water condition and are 3.10 feet and 4.17 feet for the significant and maximum waves, respectively. Superimposing the wave runup values on the PMF level of 598.17 feet resulted in a wave runup elevation of 601.27 feet for significant waves and elevation 602.34 feet for maximum waves at location A (i.e. the northern dike). In order to provide protection for this extreme event, the exterior dike at location A (northern dike) was built 2.5 feet higher than at the other locations. The top elevation of this dike is 602.5 feet.

Per the FSAR, the protection of the pond-side slope of the dikes against wind wave action is based on the local wind wave characteristics. The FSAR states that the basis used to determine the required riprap sizes and thickness of the riprap layer is extreme wind (60 mph) over normal pool or 25-mph wind (40 mph at the northern dike) over PMF pool, whichever produces the maximum wave height. Per the FSAR, based on these design conditions, the riprap sizes and thicknesses were determined using the procedures defined in U.S. Army Corps of Engineers, Shore Protection Manual, Volume I-III, U.S. Army Coastal Engineering Research Center, 1977 Reference 14). For instance, the significant wave

height generated by the 40-mph wind over PMF pool (598.17 feet) governs the design criteria for the portion of dike at location A (Figure 2.4-34 of FSAR). The 40-mph wind velocity will produce a significant wave height of 2.35 feet (Table 2.4-11 of FSAR). An 18-inch thick riprap with at least an average stone weight of 68 pounds (maximum weight of 250 pounds and minimum weight of 5 pounds) laid on a 12-inch thick gravel bedding is provided. Details of the riprap are shown in Figure 2.4-35 of the FSAR. See FSAR Figure 2.4-48 (Attachment 3) and FSAR Figure 2.4-35 (Attachment 4) for slope armoring detail.

In addition, in the event that the dike structure were to fail during a stillwater PMF with a wind-wave event, any wave runup propagating north to the power block will be impeded by sets of concrete blocks and vehicle barriers that surround the site that will act to dissipate wave energy and prevent water from impacting any safety-related structures. The vehicle barriers provide defense-in-depth for this potential failure mode.

The above design basis for the riprap protection is considered reliable for the reevaluated hazard primarily because the maximum reevaluated wind-wave runup elevation (602.15-ft MSL) is bounded by the design basis wind-wave runup elevation (602.34-ft MSL). The dike provides 0.35 foot of margin above the maximum reevaluated wind-wave runup elevation. Furthermore, defense-in-depth wind-wave protection is provided by the concrete blocks and vehicle barriers located between the plant and cooling pond.

<u>Maintenance and Inspection Program</u>:

Ongoing monitoring of the dike consists of a multi-level program, including quarterly (minor) and annual (major) inspection monitoring. The quarterly dike/lake monitoring is performed in accordance with procedure BwVS 1000-2 "Minor Inspection Procedure – Braidwood Cooling Lake." The purpose of this procedure is to inspect the cooling pond for potential problems which could ultimately affect the structural integrity of the dikes. The annual dike/lake inspection monitoring is performed in accordance with procedure BwVS 1000-1 "Braidwood Cooling Lake Major Inspection." The purpose of this procedure is to set the standards and criteria to which the Braidwood cooling pond shall be inspected annually (includes inspection of the dike). Procedures BwVS 1000-1 and BwVX 1000-2 are provided as Attachment 6.

The quarterly monitoring/inspection procedure is performed by trained plant personnel. The annual major inspection is performed by a consultant having, per the procedure prerequisites, the following qualifications:

- Be a licensed professional engineer;
- Have at least 10 years of experience and expertise in dam design and construction and in the investigation of the safety of existing dams; and
- Not be, or not have been, an employee of Exelon or its affiliates or an agent acting on behalf of Exelon or its affiliates up to two years before being retained by Exelon to perform a major inspection.

Both procedure BwVS 1000-1 and BwVS 1000-2 require that corrective action be initiated if any deficiencies are identified during performance of the procedures. Based on plant personnel knowledge, no significant slope stability issues have ever been observed or experienced along the dike.

This ongoing inspection and monitoring program provides assurance that the dike will remain in good condition throughout the life of the plant. If deficiencies are identified during any of the monitoring and inspection activities, they are properly reported and addressed in timely manner.

#### *5.2.3.3 Flood Protection Performance Justification*

As discussed above, the northern dike system provides adequate reliability in protecting the plant from Cooling Pond flooding, specifically sloughing and wind-wave activity. The maximum reevaluated wind-wave runup elevation is bounded by the design basis wind-wave runup elevation. To prevent sloughing, the interior dike slope was excavated/constructed at a slope of 3:1. This is much more mild than the 1:1 excavated slope design criteria identified in NRCS Conservation Practice Standard – Pond Code 378 manual (Reference 12). Also, the pond-side slope is armored with 18 inches of riprap, over a 12-inch thick gravel bed, to provide protection against wind-wave erosion. Finally, an ongoing maintenance and inspection program, which includes inspection and monitoring of the dike quarterly (minor inspection) and annually (major inspection), ensures that the system remains in good repair.

With a top elevation of 602.5-ft MSL, the dike provides 3.14 feet and 0.35 foot of margin above the stillwater and wind-wave runup elevations, respectively. This margin is considered adequate given the following justifications:

- The available margin exceeds effects of uncertainty in pond depth (Reference 16).
- Conservatisms in the wind-wave runup calculation, including fetch lengths, roughness factor, and the reduction factor due to the berm (Reference 15).

### 5.2.4 Flood Protection Feature 3 - Protection against Ingress through Essential Service Water Discharge and Circulating Water Discharge Pipe Pathways

There are three pipes that penetrate, or pass below, the northern dike, including two 48 inch diameter Essential Service Water Discharge pipes (one for each unit) and the River Makeup Water pipe. There are also two Circulating Water Discharge pipes (one for each Unit) that enter the cooling pond through the Circulating Water Discharge Structure (i.e. they do not penetrate the dike directly). These pipes were assessed as part of this LIA for creating a potential pathway for floodwaters to reach the plant.

The two Essential Service Water Discharge pipes and two Circulating Water Discharge pipes create a potential floodwater backflow pathway from the cooling pond to the plant. These pipes are submerged at the design basis stillwater elevation of 599.36, however during the reevaluated Section H.1 Combination for the Cooling Pond, an additional 1.19 feet of water head (equal to 0.52 psi) will be applied to these piping systems. The plant is protected from backflow through the Essential Service Water Discharge and Circulating Water Discharge pipes by the piping, pumps and associated seals for each of these systems.

Additionally, the Essential Service Water Discharge and River Makeup Water Discharge pipes create a potential seepage pathway via the pipe penetrations through, or below, the northern dike. The plant is protected from this potential seepage by site topography and grading.

The failure modes determined for this flood protection feature are:

- Backflow of floodwater due to the increased pressure created by the higher stillwater elevation;
- Potential for seepage around pipe penetrations through or below the northern dike.

#### *5.2.4.1 Performance Criteria*

Backflow Prevention:

The backflow prevention performance criteria requires demonstrating that the additional head pressure (1.19 feet of water or 0.52 psi) exerted by the increased reevaluated stillwater elevation does not cause water to backflow into the plant and impact safety-related SSCs.

Seepage around Penetration:

The potential for groundwater seepage to pass around any of the pipes penetrating through or below the dike was also considered. However, since all buildings housing all safety-related SSCs are built above elevation 600 ft MSL and have foundations designed to withstand hydrostatic pressure to grade (i.e. to a nominal elevation of 600 ft MSL), site topography and grading provide adequate reliability and margin for this potential failure mode. Site topography and grading as a flood protection feature is described in detail in Section 5.2.2 and, therefore will not be repeated in this section. Based on the evaluation described in Section 5.2.2 and reevaluated stillwater elevation of 599.36 ft MSL, site grading and topography provide adequate reliability and margin against potential seepage around any pipes penetrating or passing below the northern dike.

#### *5.2.4.2 Flood Protection Evaluation*

The Essential Service Water Discharge pipe is 48 inches in diameter and the Circulating Water Discharge pipe is 192 inches in diameter. These are both very large and robust piping and pumping systems. Given the substantial size of these systems, the additional 1.19 feet of water pressure (0.52 psi) is considered insignificant and would not be expected to have an impact on safety-related SSCs.

#### *5.2.4.3 Flood Protection Performance Justification*

As demonstrated above, Braidwood Units 1 & 2 site topography, with a nominal grade elevation of 600-ft MSL, is adequately reliable in protecting the plant from potential seepage around any pipe penetrating through or below the northern dike given the reevaluated stillwater elevation of 599.36 ft MSL. The 0.64 feet of margin is considered to be adequate given that this available margin significantly exceeds uncertainty estimates in flood levels (+/- 0.02 foot) (Reference 16).

Furthermore, the size and robustness of the Essential Service Water Discharge and Circulating Water Discharge pipe and pumping systems provide adequate reliability in protecting the plant from potential backflow due to the increased pressure from the higher reevaluated stillwater elevation.

The reevaluated stillwater flood elevation of 599.36-ft MSL creates an additional 1.19 feet (0.52 psi) of pressure on these systems. The results of the sensitivity analysis (Reference 16) show that the stillwater level is accurate to +/- 0.02 foot, supporting the adequate margin determination for this protection feature.

# 6 Evaluation Results

Two reevaluated combined-effect flood hazards were determined to be not bounded by the current design basis. The reevaluated Section H.1 Combination for the Mazon River (with hydrologic dam failure) has a maximum stillwater elevation that is 12.25 feet higher than the design basis maximum stillwater elevation and a maximum wind-wave runup elevation that is 12.51 feet higher than the design basis maximum wind-wave runup elevation. The reevaluated Section H.1 Combination for the Cooling Pond has a maximum stillwater elevation that is 1.19 feet higher than the design basis maximum stillwater elevation. Since these flood-causing mechanisms were not bounded by the current design basis, an Integrated Assessment was developed as required in NTTF Recommendation 2.1 (Enclosure 2 to the March 12, 2012 10 CFR 50.54(f) letter). Since all flood protection features at Braidwood are permanent and passive, a limited and focused Integrated Assessment was deemed to be appropriate, per the guidance provided in the trigger letter.

Flood scenario parameters are identified in the FHRR for the Section H.1 Combination for the Mazon River (with Hydrologic Dam Failure) and the Section H.1 Combination for the Cooling Pond.

For the Section H.1 Combination for the Mazon River (with Hydrologic Dam Failure), both the maximum stillwater elevation and maximum wind-wave runup elevation are not bounded. Site topography is the only flood protection feature (system) associated with this hazard. Site topography provides a natural barrier that protects safety-related SSCs below elevation 600-ft MSL, which yields 3.49 feet of reliable, available margin above the bounding water elevation of 596.51-ft MSL for this hazard. The two potential failure modes (groundwater ingress and settlement) were assessed and found to be not credible. The adequacy of the margin is supported by the following two key points:

- The hydrologic analysis for the Mazon River had several conservatisms included in developing the PMF flow rates (fully saturated soils for constant loss rate; zero initial losses; and generalized non-linear response adjustments to the unit hydrograph); and
- The available margin exceeds established criteria for uncertainties in the hydraulic model used to estimate flood level.

Given the reliability of site topography as a flood protection feature, the conservatisms in the PMF hydrology, and the exceedance of established uncertainty criteria in hydraulic modeling, this flood protection system is considered to have adequate reliability and margin.

For the Section H.1 Combination for the Cooling Pond, the maximum stillwater elevation was determined to be not bounded by the current design basis flood. The flood protection features (systems) associated

with this hazard include: site topography, grading and slurry trench; northern dike system; and protection against ingress through the Essential Service Water Discharge and Circulating Water Discharge pipe and pumping systems.

Site topography provides a natural barrier that protects safety-related SSCs below elevation 600-ft MSL, which corresponds to 0.64 foot of reliable and available margin above the reevaluated maximum stillwater elevation of 599.36 ft MSL. Additionally, a slurry trench having very low permeability was designed, tested and installed around the perimeter of the cooling pond to limit any seepage from the cooling pond, thus greatly reducing the potential for groundwater ingress due to slight increase in cooling pond elevation. The small increase in the reevaluated stillwater level is not expected to have an effect on groundwater levels at the plant. Additionally, the design basis assumes a groundwater level at plant grade (elevation 600-ft MSL) for the design of safety-related plant structures. All subsurface and foundations are designed to withstand full hydrostatic loads. Finally, during the flood hazard reevaluation, a sensitivity analysis was conducted for the PMF stillwater calculation for the cooling pond, which determined that the reevaluated stillwater elevation calculation is accurate to +/-0.02 feet. Based on this evaluation, the features (site topography, grading and cutoff/slurry wall) provide flood protection with adequate reliability and margin and the two potential failure modes (groundwater ingress and settlement) were found to be not credible.

With a top elevation of 602.5-ft MSL, the northern dike system provides 3.14 feet and 0.35 foot of margin above the stillwater and wind-wave runup elevations, respectively. This margin is considered adequate given the following justifications:

- The available margin exceeds effects of uncertainty in pond depth (Reference 16).
- Conservatisms in the wind-wave runup calculation, including fetch lengths, roughness factor, and the reduction factor due to the berm (Reference 15).

Potential failure modes, including dike sloughing and erosion from wind waves, are addressed through appropriate slope design/construction, slope armoring, and a quarterly and annual monitoring and inspection program. In addition, in the event that the dike structure were to fail due to wind-wave activity, wave runup propagating north to the power block will be impeded by sets of concrete blocks and vehicle barriers that surround the site. These blocks will dissipate wave energy and prevent water from impacting safety-related structures. Based on this evaluation, this flood protection system is considered to be reliable and to provide adequate margin.

Finally, site topography and grading provide reliable protection and adequate margin from potential flooding due to seepage through pipe penetrations in the northern dike. The robustness of the Essential Service Water Discharge and Circulating Water Discharge pipe and pumping systems provide adequate flood protection reliability and margin against potential flood water infiltration through these pipe and pumping systems given the insignificant increase in pressure created by the reevaluated stillwater elevation..

# 7 References


1. U.S. Nuclear Regulatory Commission, Request for Information Pursuant to Title 10 of the Code of Federal Regulations (CFR) 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, March 12, 2012.
2. Braidwood Generating Station Flood Hazard Reevaluation Report (FHRR), March 2014.
4. U.S. Nuclear Regulatory Commission. Staff Requirements – SECY-11-0093 – Near-Term Report and Recommendations for Agency Actions Following the Events in Japan, August 19, 2011.
5. Letter from David L. Skeen, U.S. Nuclear Regulatory Commission, to Joseph E. Pollock, Nuclear Energy Institute – "Trigger Conditions for Performing an Integrated Assessment and Due Date for Response", dated December 3, 2012.
6. U.S. Nuclear Regulatory Commission. Staff Requirements – SECY-11-0124 – Recommended Actions to be Taken Without Delay From the Near-Term Task Force Report, October 18, 2011.
7. U.S. Nuclear Regulatory Commission. "Prioritization of Recommended Actions to be Taken in Response to Fukushima Lessons Learned," SECY-11-0137, October 3, 2011.
8. Consolidated Appropriation Act, for 2012 (PUB LAW 112-74), Section 402.
9. 10 CFR 52 – Licenses, Certifications, and Approvals for Nuclear Power Plants.
10. U.S. Nuclear Regulatory Commission. Generic Issue (GI) 204, "Flooding of Nuclear Power Plant Sites Following Upstream Dam Failure."
11. U.S. NRC Japan Lessons-Learned Project Directorate, JLD-ISG-2012-05, Guidance for Performing the Integrated Assessment for External Flooding, dated November 30, 2012 (ISG).
12. National Resources Conservation Services (NRCS) *Conservation Practice Standard – Pond* Code 378 manual.
13. Braidwood Generating Station Final Safety Analysis Report (FSAR) Section 2.4 "Hydrologic Engineering" and Section 2.5 "Geology, Seismology, ad Geotechnical Engineering."
14. U.S. Army Corps of Engineers, Shore Protection Manual, Volume I-III, U.S. Army Coastal Engineering Research Center, 1977.
15. Enercon, BRW-13-FUK-0173, Combined-Effects Analysis, Revision 0, 2014.
16. Enercon, BRW-13-FUK-0175, Error/Uncertainty Analysis, Revision 0, 2014.
17. Flooding Walkdown Report in Response to the 50.54(f) Information Request Regarding Near-Term Task Force Recommendation 2.3: Flooding for the Braidwood Nuclear Power Station, November 9, 2013, Rev 1.

# ATTACHMENT 1

ISG Figure 3, Flood Protection Evaluation Flowchart

## Attachement 1 - ISG Figure 3, Flood Protection Evaluation Flowchart

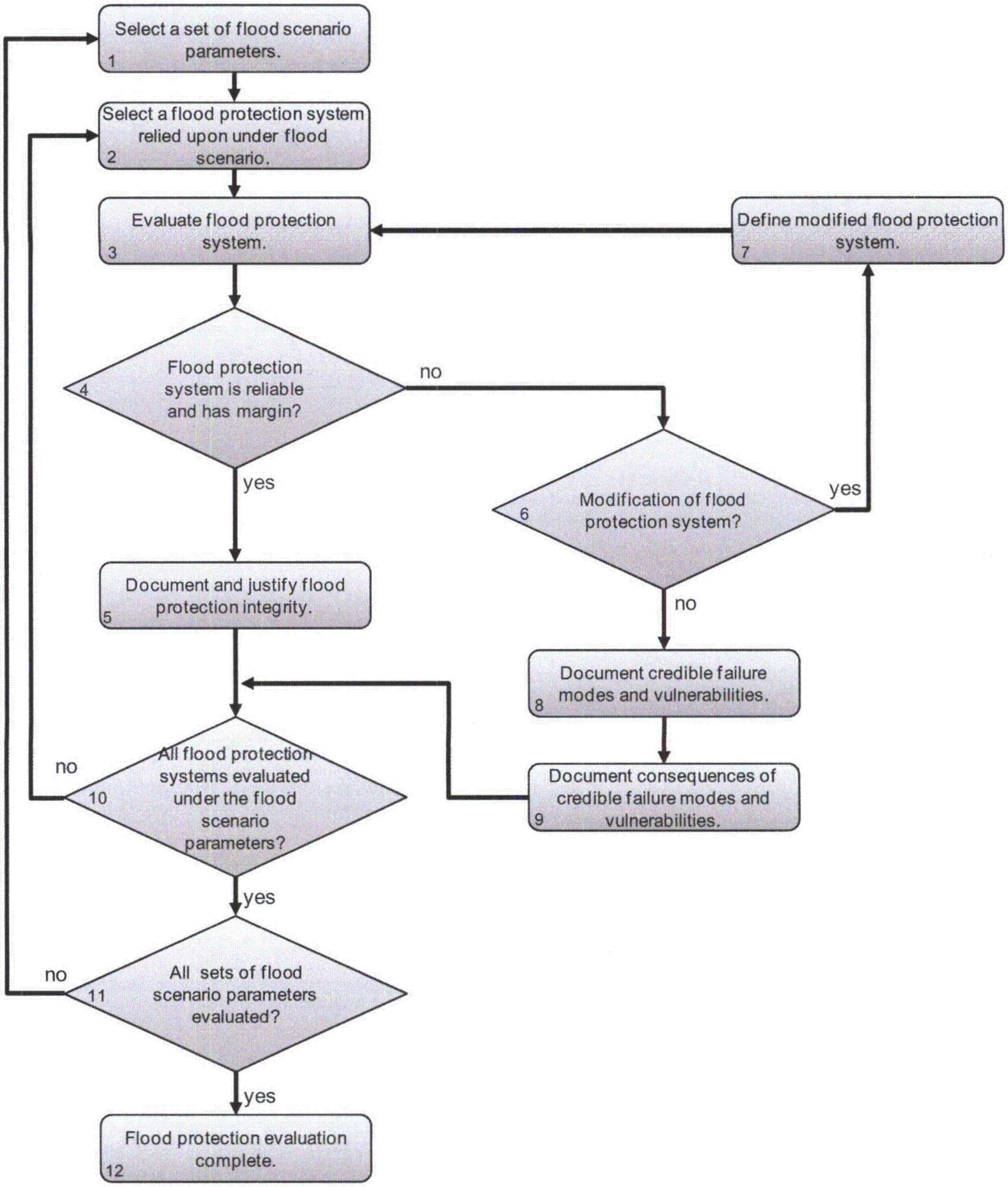



**Figure 3: Flood protection evaluation process flowchart**

# ATTACHMENT 2

FSAR Tables 2.5-41 & 2.5-42

TABLE 2.5-41

TABULATED DIFFERENTIAL SETTLEMENTS FOR SURVEY MONUMENTS

| BUILDING | MONUMENT NUMBER | PERIOD OF MEASUREMENT | MAXIMUM MEASURED DIFFERENTIAL MOVEMENT (feet)* | DIFFERENTIAL MOVEMENT BASED ON STABILIZED ELEVATION (ft) |
|---|---|---|---|---|
| Fuel | 9 | 2/79 to 12/81 | +0.002 | -0.015 |
| | 10 | 2/79 to 8/80 | -0.012 | |
| | New 10 | 9/81 to 4/88 | -0.006 | |
| | New 9 | 9/81 to 12/85 | -0.008 | |
| | 51 | 9/81 to 4/88 | -0.004 | |
| | 52 | 9/81 to 6/83 | +0.001 | |
| | 52 A | 6/83 to 6/86 | -0.011 | |
| Refueling Water Storage Tanks | 40 | 2/79 to 8/80 | -0.025 | -0.010 |
| | New 40 | 9/81 to 4/88 | -0.011 | |
| | 55 | 9/81 to 4/88 | -0.006 | |
| Auxiliary Building | KK | 2/77 to 8/80 | -0.059 | -0.039 |
| | LL | 2/77 to 8/77 | -0.013 | |
| | JJ | 2/77 to 5/77 | -0.010 | |
| | 21 | 2/79 to 8/80 | -0.020 | -0.010 |
| | 22 | 2/79 to 8/80 | -0.013 | -0.010 |
| | 23 | 2/79 to 8/80 | -0.015 | -0.005 |
| | 24 | 2/79 to 8/80 | -0.020 | -0.015 |
| | 26 | 2/79 to 8/80 | -0.021 | -0.020 |
| | 27 | 2/79 to 8/80 | -0.027 | -0.020 |
| | 28 | 2/79 to 8/80 | -0.025 | |
| | New 21 | 9/81 to 1/87 | -0.004 | |
| | New 26 | 9/81 to 6/87 | -0.004 | |
| | New 27 | 9/81 to 3/87 | -0.011 | |
| | New 29 | 9/81 to 6/87 | +0.002 | |
| | 53 | 9/81 to 3/87 | +0.008 | |
| | 54 | 9/81 to 10/87 | -0.002 | |

TABLE 2.5-41 (Cont'd)

| BUILDING | MONUMENT NUMBER | PERIOD OF MEASUREMENT | MAXIMUM MEASURED DIFFERENTIAL MOVEMENT (feet)* | DIFFERENTIAL MOVEMENT BASED ON STABILIZED ELEVATION (ft) |
|---|---|---|---|---|
| Unit 1 Containment | U | 2/77 to 8/80 | -0.061 | -0.070 |
| | V | 2/77 to 8/80 | -0.052 | -0.063 |
| | N | 2/77 to 8/80 | -0.080 | -0.067 |
| | N2 | 3/77 to 6/77 | -0.014 | |
| | N4 | 3/77 to 6/77 | -0.014 | |
| | P | 2/77 to 8/77 | -0.004 | |
| | 13 | 2/79 to 2/80 | -0.012 | -0.008 |
| | 14 | 2/79 to 8/80 | -0.005 | -0.007 |
| | 15 | 2/79 to 8/80 | -0.010 | -0.012 |
| | 36 | 2/79 to 8/80 | -0.003 | -0.012 |
| | 39 | 2/79 to 8/80 | -0.018 | -0.012 |
| | New U | 9/81 to 3/86 | -0.006 | |
| | New V | 9/18 to 10/82 | +0.018 (Damaged) | |
| | New N | 9/81 to 4/88 | -0.010 | |
| | New 3 | 9/81 to 3/87 | -0.008 | |
| | New 37 | 9/81 to 4/88 | -0.008 | |
| | New 39 | 9/81 to 4/88 | -0.012 | |
| Unit 1 Safety Valve Room | 1 (Northeast Room) | 2/79 to 8/80 | -0.011 | -0.015 |
| | 3 (Northwest Room) | 2/79 to 8/80 | -0.027 | -0.025 |
| Unit 2 Safety Valve Room | 42 | 2/79 to 8/80 | -0.024 | -0.015 |

TABLE 2.5-41 (Cont'd)

| BUILDING | MONUMENT NUMBER | PERIOD OF MEASUREMENT | MAXIMUM MEASURED DIFFERENTIAL MOVEMENT (feet)* | DIFFERENTIAL MOVEMENT BASED ON STABILIZED ELEVATION (ft) |
|---|---|---|---|---|
| Unit 2 Containment | | | | |
| | AA | 2/77 to 6/77 | +0.005 | |
| | BB | 2/77 to 6/77 | +0.006 | |
| | R | 2/77 to 8/77 | -0.001 | |
| | R1 | 2/77 to 8/77 | -0.014 | |
| | R2 | 2/77 to 5/77 | -0.020 | |
| | R3 | 2/77 to 8/77 | -0.013 | |
| | R4 | 2/77 to 8/80 | -0.078 | -0.074 |
| | Z | 2/77 to 8/80 | +0.064 | -0.065 |
| | 18 | 2/79 to 8/80 | -0.020 | -0.015 |
| | 19 | 2/79 to 8/80 | -0.024 | -0.018 |
| | 20 | 2/79 to 8/80 | -0.020 | -0.012 |
| | 43 | 2/79 to 8/80 | -0.017 | -0.008 |
| | 44 | 2/79 to 5/80 | -0.007 | -0.010 |
| | Z1 | 9/81 to 10/86 | -0.020 (Damaged) | |
| | New R4 | 9/81 to 6/87 | -0.021 | |
| | New 17 | 9/81 to 5/84 | -0.001 | |
| | New 18 | 9/81 to 4/88 | -0.022 | |
| | New 41 | 9/81 to 4/88 | -0.023 | |
| | New Z | 9/81 to 6/87 | -0.014 | |
| Units 1 & 2 Turbine Room | | | | |
| | CC | 2/77 to 5/77 | -0.001 | |
| | HH | 2/77 to 8/77 | -0.033 | |
| | T | 2/77 to 8/77 | -0.002 | |
| | W | 3/77 to 8/77 | -0.013 | |
| | X | 2/77 to 8/77 | +0.001 | |
| | 4 | 2/79 to 8/80 | -0.010 | -0.015 |

TABLE 2.5-41 (Cont'd)

| BUILDING | MONUMENT NUMBER | PERIOD OF MEASUREMENT | MAXIMUM MEASURED DIFFERENTIAL MOVEMENT (feet)* | DIFFERENTIAL MOVEMENT BASED ON STABILIZED ELEVATION (ft) |
|---|---|---|---|---|
| | 5 | 2/79 to 8/80 | -0.001 | -0.005 |
| | 6 | 2/79 to 8/82 | +0.003 | 0 |
| | 33 | 2/79 to 8/82 | -0.005 | 0 |
| | New 4 | 9/81 to 1/88 | +0.001 | |
| | New 33 | 9/81 to 4/88 | -0.012 | |
| | New 34 | 9/81 to 4/88 | +0.002 | |
| | 56 | 9/81 to 9/85 | -0.012 | |
| | 58 | 9/81 to 4/88 | +0.006 | |
| | 59 | 9/81 to 4/88 | -0.011 | |
| Heater Bay | 57 | 9/81 to 6/87 | -0.018 | |
| Radwaste/Service Building | DD | 2/77 to 8/77 | -0.003 | |
| | XX | 2/77 to 8/80 | -0.013 | -0.023 |
| | 34 | 2/79 to 8/80 | -0.008 | 0 |
| Lake Screen House | 60 | 1/84 to 1/88 | +0.005 | |
| | 61 | 1/84 to 1/88 | +0.007 | |
| | 62 | 1/84 to 4/88 | +0 | |
| | 63 | 1/84 to 1/88 | +0.006 | |
| | 64 | 1/84 to 4/88 | +0.001 | |
| | 65 | 1/84 to 4/88 | +0.003 | |

Key: - indicates downward movement for period of measurement given.
+ indicates upward movement for period of measurement given.

TABLE 2.5-42

PROJECTED MAXIMUM TOTAL AND DIFFERENTIAL

SETTLEMENTS

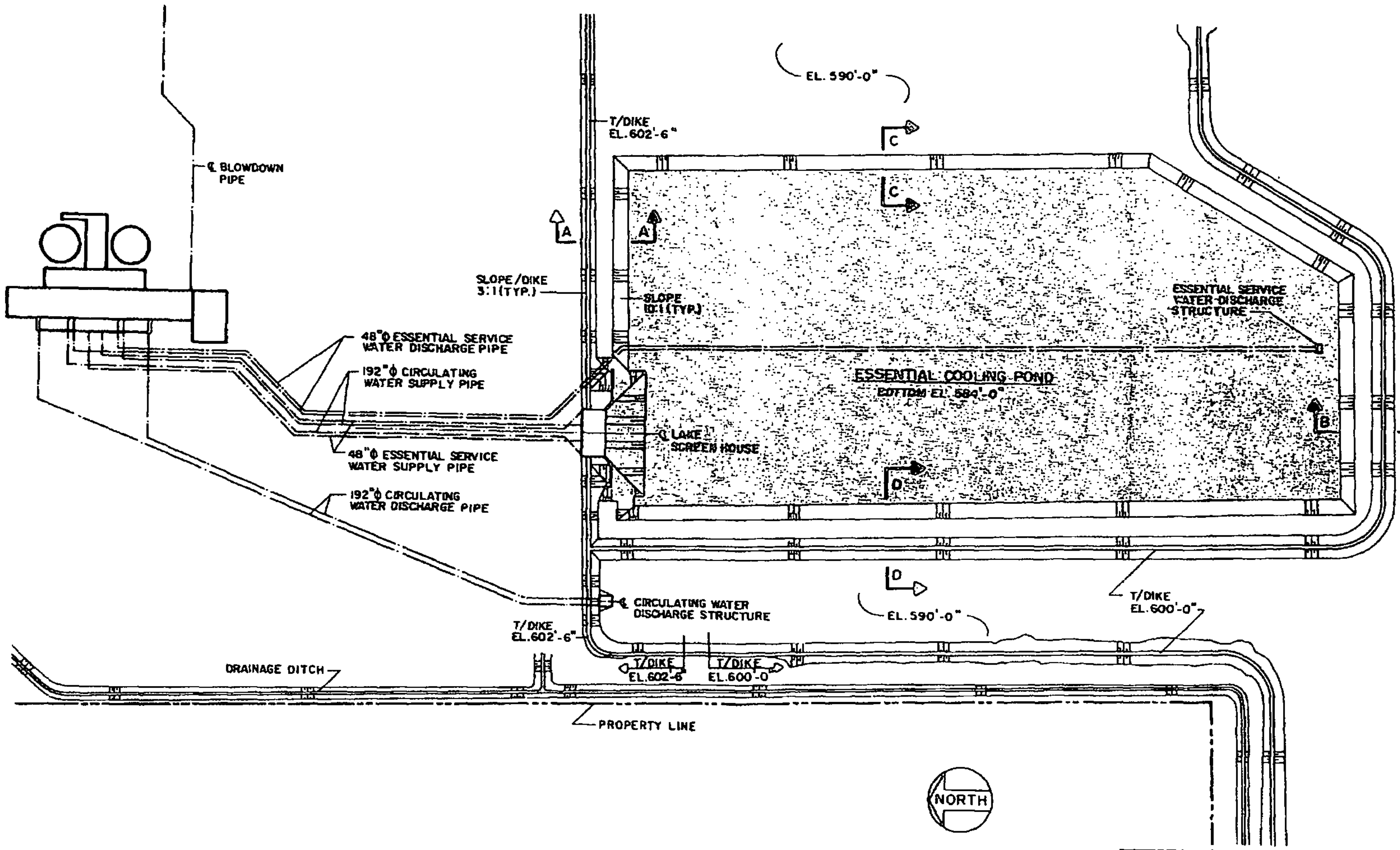
| CATEGORY I STRUCTURE | PROJECTED MAXIMUM* TOTAL SETTLEMENT (feet) | MAXIMUM DIFFERENTIAL SETTLEMENT (feet) |
|---|---|---|
| Unit 1 Containment | -0.074 | -0.01 |
| Unit 2 Containment | -0.078 | -0.01 |
| Auxiliary Building | -0.041 | -0.03 |
| Fuel Building | -0.04** | -0.02** |
| Refueling Water Tanks | -0.04** | -0.02** |

* Projected maximum total settlement determined by increasing by 5% the difference between stabilized monument elevations and the monument initial elevations. Monuments, U, V, Z, N, $R_4$, and KK were monitored from the beginning of construction to August 1980. These monuments were used to compute total settlement for the containments and auxiliary building areas.

** Settlement values given here are estimated conservatively because a significant amount of construction occurred before monuments were installed. Actual measurements indicate less than or equal to -0.025 feet total settlement.

# ATTACHMENT 3

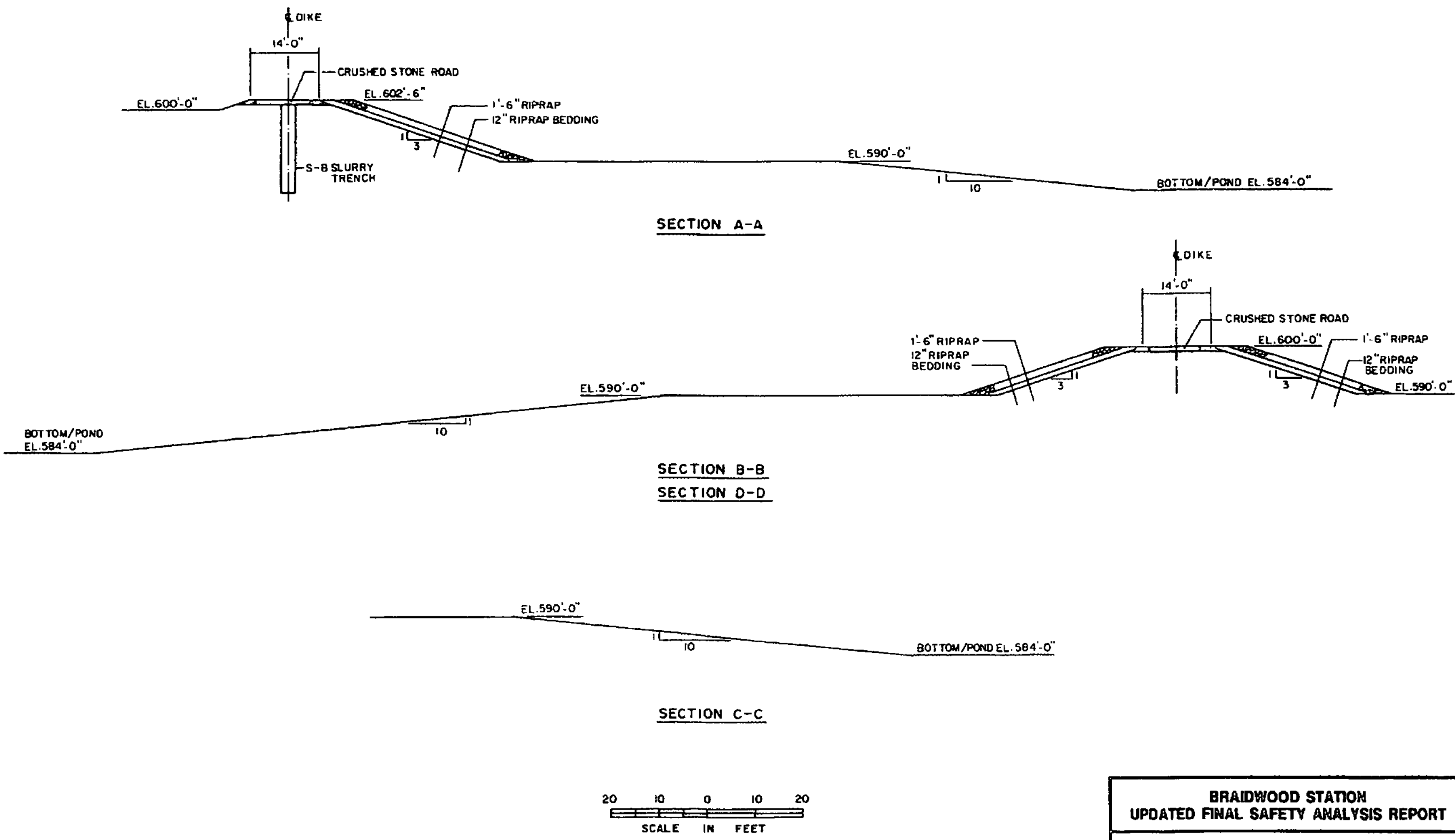
FSAR Figures 2.4-47 & 2.4-48




**BRAIDWOOD STATION**
**UPDATED FINAL SAFETY ANALYSIS REPORT**

FIGURE 2.4-47

ESSENTIAL COOLING POND




**BRAIDWOOD STATION**
**UPDATED FINAL SAFETY ANALYSIS REPORT**

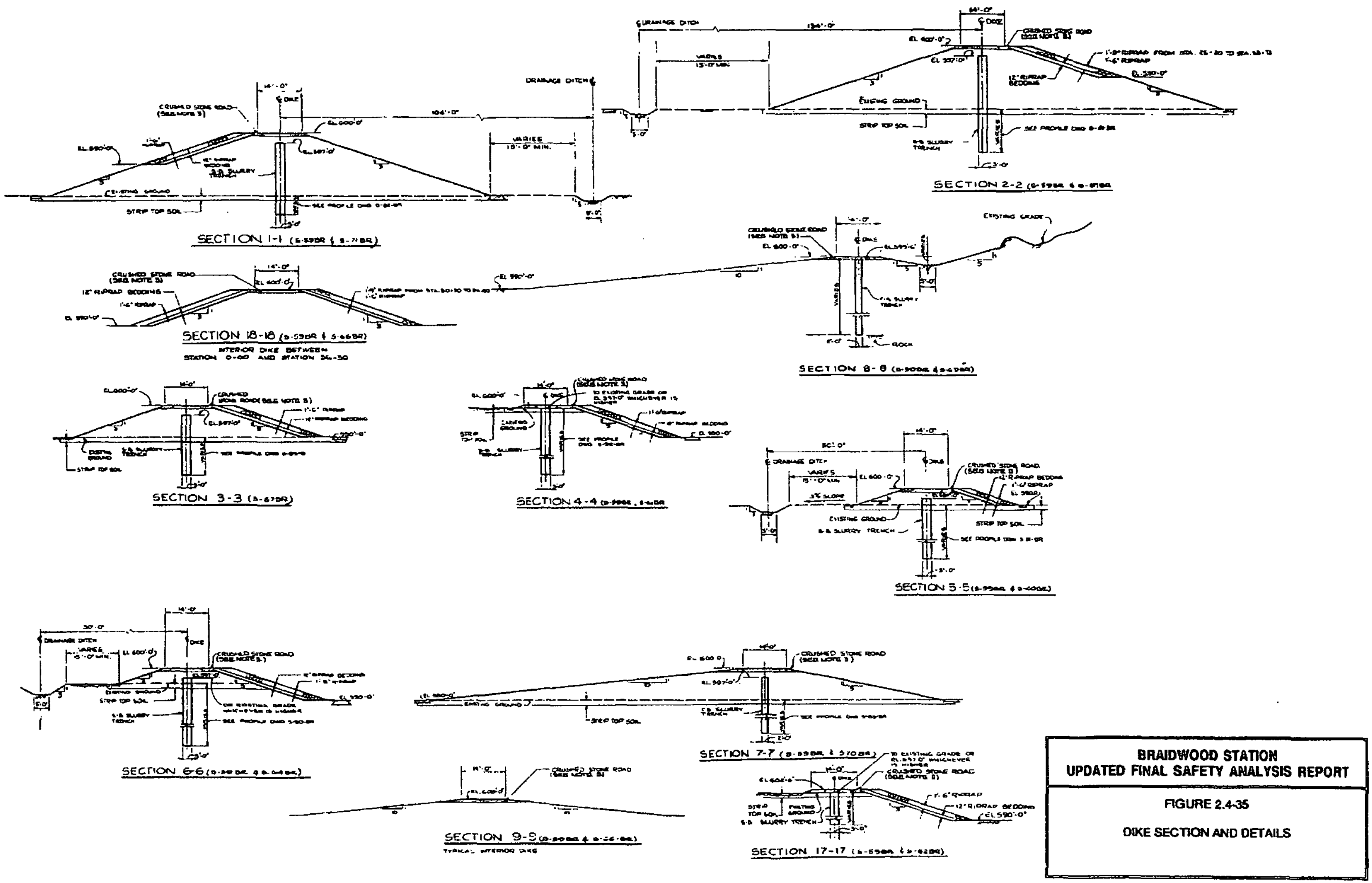
**FIGURE 2.4-48**

**ESSENTIAL COOLING POND SECTIONS**

# ATTACHMENT 4

FSAR Figure 2.4-35

**Attachement 4 - FSAR Figure 2.4-35**



# ATTACHMENT 5

NRCS Conservation Practice Standard Pond Manual 378

# NATURAL RESOURCES CONSERVATION SERVICE
# CONSERVATION PRACTICE STANDARD

# POND

**(No.)**

**CODE 378**

## DEFINITION

A water impoundment made by constructing an embankment or by excavating a pit or dugout.

In this standard, ponds constructed by the first method are referred to as embankment ponds, and those constructed by the second method are referred to as excavated ponds. Ponds constructed by both the excavation and the embankment methods are classified as embankment ponds if the depth of water impounded against the embankment at the auxiliary spillway elevation is 3 feet or more.

## PURPOSE

To provide water for livestock, fish and wildlife, recreation, fire control, develop renewable energy systems, and other related uses, and to maintain or improve water quality.

## CONDITIONS WHERE PRACTICE APPLIES

This standard establishes the minimum acceptable quality for the design and construction of low-hazard ponds where:

Failure of the dam will not result in loss of life; damage to homes, commercial or industrial buildings, main highways, or railroads; or in interruption of the use or service of public utilities.

The product of the storage times the effective height of the dam is less than 3,000. Storage is the volume, in acre-feet, in the reservoir below the elevation of the crest of the auxiliary spillway. The effective height of the dam is the difference in elevation, in feet, between the auxiliary spillway crest and the lowest point in the cross section taken along the centerline of the dam. If there is no auxiliary spillway, the top of the dam is the upper limit.

The effective height of the dam is 35 feet or less.

### General Criteria Applicable to All Ponds

All federal, State and local requirements shall be addressed in the design.

A protective cover of vegetation shall be established on all exposed areas of embankments, spillways and borrow areas as climatic conditions allow, according to the guidelines in conservation practice standard 342, Critical Area Planting.

**Site conditions.** Site conditions shall be such that runoff from the design storm can be safely passed through (1) a natural or constructed auxiliary spillway, (2) a combination of a principal spillway and an auxiliary spillway, or (3) a principal spillway.

**Drainage area.** The drainage area above the pond must be protected against erosion to the extent that expected sedimentation will not shorten the planned effective life of the structure. The drainage area shall be large enough so that surface runoff and groundwater will provide an adequate supply of water for the intended purpose unless an alternate water source exists to serve this purpose. The quality shall be suitable for the water's intended use.

**Reservoir area.** The topography and geology of the site shall permit storage of water at a depth and volume that will ensure a dependable supply, considering beneficial use, sedimentation, season of use, and evaporation and seepage losses. If surface runoff is the primary source of water for a pond, the soils shall be impervious enough to prevent excessive seepage losses or shall be of a type that sealing is practicable.

### Design Criteria for Embankment Ponds

**Geological Investigations.** Pits, trenches, borings, review of existing data or other suitable means of investigation shall be

conducted to characterize materials within the embankment foundation, auxiliary spillway and borrow areas. Soil materials shall be classified using the Unified Soil Classification System.

**Foundation cutoff.** A cutoff of relatively impervious material shall be provided under the dam if necessary to reduce seepage through the foundation. The cutoff shall be located at or upstream from the centerline of the dam. It shall extend up the abutments as required and be deep enough to extend into a relatively impervious layer or provide for a stable dam when combined with seepage control. The cutoff trench shall have a bottom width adequate to accommodate the equipment used for excavation, backfill, and compaction operations. Side slopes shall not be steeper than one horizontal to one vertical.

**Seepage control.** Seepage control is to be included if (1) pervious layers are not intercepted by the cutoff, (2) seepage could create swamping downstream, (3) such control is needed to insure a stable embankment, or (4) special problems require drainage for a stable dam. Seepage may be controlled by (1) foundation, abutment, or embankment filters and drains; (2) reservoir blanketing; or (3) a combination of these measures.

**Embankment.** The minimum top width for a dam is shown in table 1. If the embankment top is to be used as a public road, the minimum width shall be 16 feet for one-way traffic and 26 feet for two-way traffic. Guardrails or other safety measures shall be used where necessary and shall meet the requirements of the responsible road authority. For dams less than 20 feet in height, maintenance considerations or construction equipment limitations may require increased top widths from the minimum shown in Table 1.

**Table 1. Minimum top width for dams**

| Total height of embankment | Top width |
|---|---|
| *feet* | *feet* |
| Less than 10 | 6 |
| 10 – 14.9 | 8 |
| 15 – 19.9 | 10 |
| 20 – 24.9 | 12 |
| 25 – 34.9 | 14 |
| 35 or more | 15 |

**Side Slopes.** The combined upstream and downstream side slopes of the settled embankments shall not be less than five horizontal to one vertical, and neither slope shall be steeper than two horizontal to one vertical. All slopes must be designed to be stable, even if flatter side slopes are required. Downstream or upstream berms can be used to help achieve stable embankment sections

**Slope Protection.** If needed to protect the slopes of the dam from erosion, special measures, such as berms, rock riprap, sand-gravel, soil cement, or special vegetation, shall be provided (Technical Releases 56, "A guide for Design and Layout of Vegetative Wave Protection for Earth Dam Embankments" and 69, "Riprap for Slope Protection Against Wave Action" contain design guidance).

**Freeboard.** The minimum elevation of the top of the settled embankment shall be 1 foot above the water surface in the reservoir with the auxiliary spillway flowing at design depth. The minimum difference in elevation between the crest of the auxiliary spillway and the settled top of the dam shall be 2 feet for all dams having more than a 20-acre drainage area or more than 20 feet in effective height.

**Settlement.** The design height of the dam shall be increased by the amount needed to insure that after settlement the height of the dam equals or exceeds the design height. This increase shall not be less than 5 percent of the height of the dam, except where detailed soil testing and laboratory analyses or experience in the area show that a lesser amount is adequate.

**Principal spillway.** A pipe conduit, with needed appurtenances, shall be placed under or through the dam, except where rock, concrete, or other types of lined spillways are used, or where the rate and duration of flow

can be safely handled by a vegetated or earth spillway.

For dams with a drainage area of 20 acres or less, the principal spillway crest elevation shall not be less than 0.5 feet below the auxiliary spillway crest elevation. For dams with a drainage area over 20 acres, this difference shall not be less than 1.0 feet.

When design discharge of the principal spillway is considered in calculating peak outflow through the auxiliary spillway, the crest elevation of the inlet shall be such that the design discharge will be generated in the conduit before there is discharge through the auxiliary spillway.

Pipe conduits designed for pressure flow must have adequate anti-vortex devices. The inlets and outlets shall be designed to function satisfactorily for the full range of flow and hydraulic head anticipated.

The capacity of the pipe conduit shall be adequate to discharge long-duration, continuous, or frequent flows without flow through the auxiliary spillways. The diameter of the principal spillway pipe shall not be less than 4 inches. Pipe conduits used solely as a supply pipe through the dam for watering troughs and other appurtenances shall not be less than 1-1/4 inches in diameter.

If the pipe conduit diameter is 10 inches or greater, its design discharge may be considered when calculating the peak outflow rate through the auxiliary spillway.

Pipe conduits shall be ductile iron, welded steel, corrugated steel, corrugated aluminum, reinforced concrete (pre-cast or site-cast), or plastic. Pipe conduits through dams of less than 20 feet total height may also be cast iron or unreinforced concrete.

Pipe conduits shall be designed and installed to withstand all external and internal loads without yielding, buckling, or cracking. Rigid pipe shall be designed for a positive projecting condition. Flexible pipe shall be designed for a maximum deflection of 5 percent. The modulus of elasticity for PVC pipe shall be assumed as one-third of the amount designated by the compound cell classification to account for long-term reduction in modulus of elasticity. Different reductions in modulus may be appropriate for other plastic pipe materials.

The minimum thickness of flexible pipe shall be SDR 26, Schedule 40, Class 100, or 16 gage as appropriate for the particular pipe material. Connections of flexible pipe to rigid pipe or other structures shall be designed to accommodate differential movements and stress concentrations.

All pipe conduits shall be designed and installed to be water tight by means of couplings, gaskets, caulking, waterstops, or welding. Joints shall be designed to remain watertight under all internal and external loading including pipe elongation due to foundation settlement.

Pipe conduits shall have a concrete cradle or bedding if needed to provide improved support for the pipe to reduce or limit structural loading on pipe to allowable levels.

Cantilever outlet sections, if used, shall be designed to withstand the cantilever load. Pipe supports shall be provided when needed. Other suitable devices such as a Saint Anthony Falls stilling basin or an impact basin may be used to provide a safe outlet.

All steel pipe and couplings shall have protective coatings in areas that have traditionally experienced pipe corrosion, or in embankments with saturated soil resistivity less than 4000 ohms-cm or soil pH less than 5. Protective coatings shall be asphalt, polymer over galvanizing, aluminized coating or coal tar enamel as appropriate for the pipe type. Plastic pipe that will be exposed to direct sunlight shall be ultraviolet-resistant and protected with a coating or shielding, or provisions provided for replacement as necessary

**Renewable Energy**. For detailed criteria where the purpose is to develop renewable energy systems refer to interim conservation practice standard Renewable Energy Production (716).

**Cathodic Protection.** Cathodic protection is to be provided for coated welded steel and galvanized corrugated metal pipe where soil and resistivity studies indicate that the pipe needs a protective coating, and where the need and importance of the structure warrant additional protection and longevity. If cathodic protection is not provided for in the original design and installation, electrical continuity in the form of joint-bridging straps should be

considered on pipes that have protective coatings. *Cathodic protection should be added* later if monitoring indicates the need.

**Seepage Control.** Seepage control along a pipe conduit spillway shall be provided if any of the following conditions exist:

- The effective height of dam is greater than 15 feet.
- The conduit is of smooth pipe larger than 8 inches in diameter.
- The conduit is of corrugated pipe larger than 12 inches in diameter.

Seepage along pipes extending through the embankment shall be controlled by use of a drainage diaphragm, unless it is determined that anti-seep collars will adequately serve the purpose.

**Drainage Diaphragm.** The drainage diaphragm shall function both as a filter for adjacent base soils and a drain for seepage that it intercepts. The drainage diaphragm shall consist of sand meeting the requirements of ASTM C-33, for fine aggregate. If unusual soil conditions exist such that this material may not meet the required filter or capacity requirements, a special design analysis shall be made.

The drainage diaphragm shall be a minimum of 2 feet thick and extend vertically upward and horizontally at least three times the outside pipe diameter, and vertically downward at least 18 inches beneath the conduit invert. The drainage diaphragm shall be located immediately downstream of the cutoff trench, but downstream of the centerline of the dam if the cutoff is upstream of the centerline.

The drainage diaphragm shall be outletted at the embankment downstream toe using a drain backfill envelope continuously along the pipe to where it exits the embankment. Drain fill shall be protected from surface erosion.

**Anti-seep Collars.** When anti-seep collars are used in lieu of a drainage diaphragm, they shall have a watertight connection to the pipe. Maximum spacing shall be approximately 14 times the minimum projection of the collar measured perpendicular to the pipe but not more than 25 feet. The minimum spacing shall be 10 feet. Collar material shall be compatible with pipe materials. The anti-seep collar(s) shall increase by at least 15 percent the *seepage path along the pipe.*

**Trash Guard.** To prevent clogging of the conduit, an appropriate trash guard shall be installed at the inlet or riser unless the watershed does not contain trash or debris that could clog the conduit.

**Other Outlets.** A pipe with a suitable valve shall be provided to drain the pool area if needed for proper pond management or if required by State law. The principal spillway conduit may be used as a pond drain if it is located where it can perform this function.

**Auxiliary spillways.** Auxiliary spillways convey large flood flows safely past earth embankments and have historically been *referred to as "Emergency Spillways".*

An auxiliary spillway must be provided for each dam, unless the principal spillway is large enough to pass the peak discharge from the routed design hydrograph and the trash that comes to it without overtopping the dam. The following are minimum criteria for acceptable use of a closed conduit principal spillway without an auxiliary spillway: a conduit with a cross-sectional area of 3 $ft^2$ or more, an inlet that will not clog, and an elbow designed to facilitate the passage of trash.

The minimum capacity of a natural or constructed auxiliary spillway shall be that required to pass the peak flow expected from a design storm of the frequency and duration shown in Table 2, less any reduction creditable to conduit discharge and detention storage.

The auxiliary spillway shall safely pass the peak flow, or the storm runoff shall be routed through the reservoir. The routing shall start either with the water surface at the elevation of the crest of the principal spillway or at the water surface after 10 days' drawdown, whichever is higher. The 10-day drawdown shall be computed from the crest of the auxiliary spillway or from the elevation that would be attained if the entire design storm were impounded, whichever is lower. Auxiliary spillways shall provide for passing the design flow at a safe velocity to a point downstream where the dam will not be endangered.

Constructed auxiliary spillways are open channels that usually consist of an inlet channel, a control section, and an exit channel. They shall be trapezoidal and shall be located in undisturbed or compacted earth or in-situ

rock. The side slopes shall be stable for the material in which the spillway is to be constructed. For dams having an effective height exceeding 20 feet, the auxiliary spillway shall have a bottom width of not less than 10 feet.

Upstream from the control section, the inlet channel shall be level for the distance needed to protect and maintain the crest elevation of the spillway. The inlet channel may be curved to fit existing topography. The grade of the exit channel of a constructed auxiliary spillway shall fall within the range established by discharge requirements and permissible velocities.

**Structural auxiliary spillways.** If chutes or drops are used for principal spillways or auxiliary spillways, they shall be designed according to the principles set forth in the Part 650, Engineering Field Handbook and the National Engineering Handbook, Section 5, Hydraulics; Section 11, Drop Spillways; and Section 14, Chute Spillways. The minimum capacity of a structural spillway shall be that required to pass the peak flow expected from a design storm of the frequency and duration shown in table 2, less any reduction creditable to conduit discharge and detention storage.

Table 2. Minimum auxiliary spillway capacity

| | | | Minimum design storm[2] | |
|---|---|---|---|---|
| Drainage area (Ac.) | Effective height of dam[1] (Ft.) | Storage (Ac-Ft) | Frequency (Years) | Minimum duration (Hours) |
| 20 or less | 20 or less | < than 50 | 10 | 24 |
| 20 or less | > than 20 | < than 50 | 25 | 24 |
| > than 20 | | < than 50 | 25 | 24 |
| All others | | | 50 | 24 |

1. As defined under "Conditions where Practice Applies".

2. Select rain distribution based on climatological region.

**Criteria for Excavated Ponds**

**Runoff.** Provisions shall be made for a pipe and auxiliary spillway, if needed, that will meet the capacity requirements of Table 2. Runoff flow patterns shall be considered when locating the excavated pond and placing the spoil.

**Side slopes.** Side slopes of excavated ponds shall be stable and shall not be steeper than one horizontal to one vertical. If livestock will water directly from the pond, a watering ramp of ample width shall be provided. The ramp shall extend to the anticipated low water elevation at a slope no steeper than three horizontal to one vertical.

**Inlet protection.** If surface water enters the pond in a natural or excavated channel, the side slope of the pond shall be protected against erosion.

**Excavated material.** The material excavated from the pond shall be placed so that its weight will not endanger the stability of the pond side slopes and it will not be washed back into the pond by rainfall. It shall be disposed of in one of the following ways:

Uniformly spread to a height that does not exceed 3 feet, with the top graded to a continuous slope away from the pond.

Uniformly placed or shaped reasonably well, with side slopes assuming a natural angle of repose. The excavated material will be placed at a distance equal to the depth of the pond but not less than 12 feet from the edge of the pond.

Shaped to a designed form that blends visually with the landscape.

Used for low embankment construction and leveling of surrounding landscape.

Hauled away.

**CONSIDERATIONS**

**Visual resource design.** The visual design of ponds should be carefully considered in areas of high public visibility and those associated with recreation. The underlying criterion for all visual design is appropriateness. The shape and form of ponds, excavated material, and plantings are to relate visually to their surroundings and to their function.

The embankment may be shaped to blend with the natural topography. The edge of the pond may be shaped so that it is generally curvilinear rather than rectangular. Excavated material can be shaped so that the final form is smooth, flowing, and fitting to the adjacent landscape rather than angular geometric mounds. If feasible, islands may be added for visual interest and to attract wildlife.

**Cultural Resources**. Consider existence of cultural resources in the project area and any project impacts on such resources. Consider conservation and stabilization of archeological, historic, structural, and traditional cultural properties when appropriate.

**Fish and Wildlife**. Project location and construction should minimize the impacts to existing fish and wildlife habitat.

When feasible, structure should be retained, such as trees in the upper reaches of the pond and stumps in the pool area. Upper reaches of the pond can be shaped to provide shallow areas and wetland habitat.

If fish are to be stocked, consider criteria and guidance in conservation practice standard 399, Fishpond Management.

**Vegetation**. Stockpiling topsoil for placement on disturbed areas can facilitate revegetation.

Consider placement and selection of vegetation to improve fish and wildlife habitat and species diversity.

**Water Quantity**. Consider effects upon components of the water budget, especially:

- Effects on volumes and rates of runoff, infiltration, evaporation, transpiration, deep percolation, and ground water recharge.
- Variability of effects caused by seasonal or climatic changes.
- Effects on downstream flows and impacts to environment such as wetlands, aquifers, and; social and economic impacts to downstream uses or users.
- Potential for multiple purposes.

**Water Quality**

- Consider effects on erosion and the movement of sediment, pathogens, and soluble and sediment-attached substances that are carried by runoff.
- Effects on the visual quality of onsite and downstream water resources.
- Short-term and construction-related effects of this practice on the quality of downstream water courses.
- Effects of water level control on the temperatures of downstream water to prevent undesired effects on aquatic and wildlife communities.
- Effects on wetlands and water-related wildlife habitats.
- Effects of water levels on soil nutrient processes such as plant nitrogen use or denitrification.
- Effects of soil water level control on the salinity of soils, soil water, or downstream water.
- Potential for earth moving to uncover or redistribute toxic materials such as saline soils.

## PLANS AND SPECIFICATIONS

Plans and specifications for installing ponds shall be in keeping with this standard and shall describe the requirements for applying the practice to achieve its intended purpose.

## OPERATION AND MAINTENANCE

An operation and maintenance plan shall be developed and reviewed with the landowner or individual responsible for operation and maintenance.

# ATTACHMENT 6

Braidwood Procedures BwVS 1000-1 & BwVS 1000-2

BwVS 1000-1
Revision 1
Reference Use

# BRAIDWOOD COOLING LAKE MAJOR INSPECTION

## A. STATEMENT OF APPLICABILITY

The purpose of this procedure is to set the standards and criteria to which the Braidwood Cooling Lake shall be inspected annually.

## B. REFERENCES

1. Commonwealth Edison Company Instruction No. 11-0, Surveillance Program for Earth Dikes and Dams, dated 5/1/92.
2. Illinois Department of Natural Resources (IDNR) Guidlelines and Forms for Inspection of Illinois Dams.
3. Braidwood Lake Monitoring Program – Braidwood Station – Sargent & Lundy Report GD-9 revised January 27, 1987.
4. U.S. Nuclear Regulatory Commission Regulatory Guide 1.127.
5. IDNR Permit No. NE2000125 for Braidwood Nuclear Station Cooling Pond Dam.

## C. PREREQUISITES

1. The Station shall secure the services of a qualified consultant to perform the annual inspection of the Cooling Lake. A qualified consultant shall:
   a. Be a licensed professional engineer.
   b. Have at least 10 years experience and expertise in dam design and construction and in the investigation of the safety of existing dams.
   c. Not be, or not have been, an employee of Exelon or its affiliates or an agent acting on behalf of Exelon or its affiliates up to two years before being retained by Exelon to perform a major inspection.
2. The plant could be in any operating mode.
3. Notify the proper departments as needed before the start of the inspection, such as: Security, Work Execution Center ...etc.

D. PRECAUTIONS - None

E. LIMITATIONS AND ACTIONS

1. The Major Lake Inspection shall be performed annually.

2. In the event that an immediate concern arises from this inspection regarding the health of the cooling lake dike, notify the Shift Manager and follow approved plant processes to address the issue.

F. MAIN BODY

1.0 Major Lake Inspection

1.1 Secure the services of a qualified consultant to perform a major inspection of the Braidwood Cooling Lake Dike.

1.2 Inspection shall be performed in accordance with the IDNR Guidelines and Forms for Inspection of Illinois Dams and Regulatory Guide 1.127.

1.3 Obtain complete report and forms from the consultant, and review for completeness.

1.4 Ensure complete report is received by Chemistry / Environmental Department and Design Engineering Department for review before submittal by Chemistry / Environmental to IDNR.

1.5 Initiate corrective action necessary to alleviate problem areas identified in the Major Lake Inspection Report.

G. ACCEPTANCE CRITERIA

Completion of the Annual Lake Inspection per the IDNR Guidelines and Forms for Inspection of Illinois Dams and Regulatory Guide 1.127 shall constitute Acceptance Criteria for Section 1.0.

# MINOR INSPECTION PROCEDURE
# BRAIDWOOD COOLING LAKE

## A. STATEMENT OF APPLICABILITY

The purpose of this procedure is to inspect the cooling lake for potential problems which could ultimately affect the structural integrity of the dikes.

## B. REFERENCES

1. Commonwealth Edison Company Instruction No. 11-0, Surveillance Program for Earth Dikes and Dams dated 5/1/92.

2. Illinois Department of Natural Resources (IDNR) Guidelines and Forms for Inspection of Illinois Dams.

3. Braidwood Lake Monitoring Program - Braidwood Station - Sargent & Lundy Report GD-9 revised January 27, 1987

4. Report from the consultant performing the most recent major inspection.

## C. PREREQUISITES

1. Notify the proper departments as needed before the start of the inspection, such as: Security, Work Execution Center...etc.

2. The plant may be in any operating MODE.

3. Access to a four wheel drive vehicle or equivalent.

4. Binoculars (optional)

## D. PRECAUTIONS

None.

## E. LIMITATIONS AND ACTIONS

1. At a minimum, the cooling lake SHALL be inspected on a quarterly basis (spring, summer, fall and winter) or more frequently if necessary

2. In the event that an immediate concern arises from this inspection regarding the health of the cooling lake dike, notify the Shift Manager and follow approved plant processes to address the issue.

F. <u>MAIN BODY</u>

1.0 Minor Inspection Procedure

1.1 Before inspecting the lake, read or be familiar with the information listed in the Reference section of this procedure.

> **NOTE**
> The following steps may be performed simultaneously or in any order.

1.2 Drive slowly around the entire lake and both branches of the interior dike roadway.

1.3. Frequent stops should be made to walk down suspected problem areas, especially areas where maintenance has been performed to correct previously identified problems.

1.4. Visually inspect for the following:

a. *Cracking, slumping or falling of the top or face of the dikes*

b. Erosion and the condition of vegetation on the face of the dikes

c. Wet areas on the face or at the base of the dikes

d. Condition of the rip-rap material and any exposed bedding of the dikes

e. Animal burrows or tree growth on the dikes

f. Obstructions in the peripheral drainage ditches and culverts such as debris, dams, silt or other impediments that may restrict flow

g. Condition of the Spillway and Freshwater Holding Pond

h. Pits, holes, cracks and other defects on the roadway

i. Damage to the exterior fence

j. Damage or deterioration of the observation well casings and slope measurement casings.

F. 1.4 k. Any floating debris or ice which could cause a problem especially at the Spillway.

l. Excess aquatic weed growth in shallow plats, especially near the Essential Service Cooling Pond Area.

m. Any other problems identified in the Major Inspection Report.

1.5 After the inspection, write a brief report to the System Manager Supervisor summarizing the condition of the lake. The period covered by the inspection (Quarter and Year) shall be indicated on the report.

1.6 Write Action Requests to address problems identified during the inspection.

G. <u>ACCEPTANCE CRITERIA</u>

Action requests written to address problems.

**Enclosure 3**

**CD-R labeled:**

Braidwood Generating Station
Pertinent Site Data