**Enclosure 1**

Limerick Generating Station
Flood Hazard Reevaluation Report
Revision 0

(56 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

## Limerick Generating Station

**3146 Sanatoga Road**
**Pottstown, Pennsylvania 19464**

---

Exelon

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

Prepared by:

ENERCON
Excellence—Every project. Every day

Enercon Services, Inc.
1601 Northwest Expressway, Suite 1000
Oklahoma City, OK 73118

Revision 0
Submitted Date: November 13, 2014

| | Printed Name | Affiliation | Signature | Date |
|---|---|---|---|---|
| Preparer: | Kurt Roblyer | ENERCON | | 11/13/2014 |
| Verifier: | Anubhav Gaur | ENERCON | | 11/13/2014 |
| Approver: | Pat Brunette | ENERCON | | 11/13/2014 |
| Lead Responsible Engineer: | Brian Welman | Exelon | | 1/30/2015 |
| Branch Manager | Gregory A Williams | Exelon | | 2/18/2015 |
| Senior Manager Design Engineering | Wayne Lewis | Exelon | | 2/18/2015 |
| Exelon Corporate | Joseph V. Bellini | Exelon | | 2/15/2015 |

## Contents

1. PURPOSE ..... 7
1.1 Background ..... 7
1.2 Requested Actions ..... 7
1.3 Requested Information ..... 8
2. SITE INFORMATION ..... 9
2.1 Detailed Site Information ..... 9
2.2 Current Design Basis ..... 12
2.2.1 Effects of Local Intense Precipitation ..... 12
2.2.2 Flooding in Streams and Rivers ..... 13
2.2.3 Dam Breaches and Failures ..... 13
2.2.4 Storm Surge & Seiche ..... 14
2.2.5 Tsunami ..... 14
2.2.6 Ice Induced Flooding ..... 14
2.2.7 Channel Migration or Diversion ..... 14
2.2.8 Low Water Considerations ..... 14
2.2.9 Combined Effect Flood ..... 15
2.3 Flood Related Changes to the License Basis ..... 15
2.4 Changes to the Watershed and Local Area since License Issuance ..... 15
2.5 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features ..... 15
3. SUMMARY OF FLOOD HAZARD REEVALUATION ..... 17
3.1 Effects of Local Intense Precipitation ..... 17
3.1.1 Inputs ..... 18
3.1.2 Methodology ..... 18
3.1.3 Results ..... 19
3.1.4 Conclusions ..... 20
3.2 Probable Maximum Flood in Streams and Rivers ..... 20
3.2.1 Inputs ..... 21
3.2.2 Methodology ..... 22
3.2.3 Results ..... 27
3.2.4 Conclusions ..... 28
3.3 Storm Surge, Seiche, and Tsunami Screening ..... 28
3.3.1 Inputs ..... 28
3.3.2 Methodology ..... 29
3.3.3 Results ..... 30
3.3.4 Conclusions ..... 31

3.4 Dam Failure ....31
3.4.1 Inputs ....31
3.4.2 Methodology ....31
3.4.3 Results ....33
3.4.4 Conclusions ....33
3.5 Combined Events Flood ....33
3.5.1 Inputs ....34
3.5.2 Methodology ....35
3.5.3 Results ....36
3.5.4 Conclusions ....37
3.6 Ice-Induced Flooding ....37
3.6.1 Inputs ....37
3.6.2 Methodology ....38
3.6.3 Results ....39
3.6.4 Conclusions ....39
3.7 Channel Migration ....39
3.7.1 Inputs ....40
3.7.2 Methodology ....40
3.7.3 Results ....40
3.7.4 Conclusions ....40
3.8 Error/Uncertainty ....40
3.8.1 Inputs ....41
3.8.2 Methodology ....41
3.8.3 Results ....42
3.8.4 Conclusions ....42
3.9 Associated Effects ....42
3.9.1 Inputs ....42
3.9.2 Methodology ....42
3.9.3 Results ....45
3.9.4 Conclusions ....45
4. FLOOD PARAMETERS AND COMPARISON WITH CURRENT DESIGN BASIS ....46
5. REFERENCES ....53

## List of Tables

Table 4.0.1 - Summary of Licensing Basis and External Flooding Study Parameters....49
Table 4.0.2 – Local Intense Precipitation....51
Table 4.0.3 – Combinations in Section H.1 of NUREG/CR-7046 for Possum Hollow Run Including Dam Failure....52

## List of Figures

Figure 2.1.1 – Present-Day General Site Map....10
Figure 2.1.2 – Present-Day Detailed Site Layout....11
Figure 4.0.1 - Illustration of Flood Event Duration (NRC JLD-ISG-2012-05, Figure 6)....47

## Acronyms and Abbreviations

| | |
|---|---|
| ANS | American Nuclear Society |
| ANSI | American National Standards Institute |
| CEM | Coastal Engineering Manual |
| cfs | cubic feet per second |
| CLB | Current Licensing Basis |
| CN | Curve Number |
| DEM | Digital Elevation Model |
| DTM | Digital Terrain Model |
| EM | Engineer Manual |
| ESP | Early Site Permit |
| ESRI | Environmental Systems Research Institute |
| FEMA | Federal Emergency Management Agency |
| FIS | Flood Insurance Study |
| fps | foot, feet per second |
| ft | foot, feet |
| GHCND | Global Historical Climatology Network-Daily |
| 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) |
| HSG | Hydrologic Soil Group |
| in | inch (inches) |
| LiDAR | Light Detection and Ranging |
| LIP | Local Intense Precipitation |
| LGS | Limerick Generating Station |
| mi | mile(s) |
| min | minute(s) |
| MSL | mean sea level |
| NAVD | North American Vertical Datum |
| NCDC | National Climatology Data Center |
| NGDC | National Geophysical Data Center |
| NGVD | National Geodetic Vertical Datum |
| NID | National Inventory of Dams |
| 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 |
| PASDA | Pennsylvania Spatial Data Access |
| PCTBF | Postulated Cooling Tower Basin Failure |
| PennDOT | Pennsylvania Department of Transportation |
| PMF | probable maximum flood |
| PMP | probable maximum precipitation |
| PMS | probable maximum seiche |

| | |
|---|---|
| PMSS | probable maximum storm surge |
| PMT | probable maximum tsunami |
| sq.mi. | square mile(s) |
| SPF | standard project flood |
| SSCs | structures, systems, and components |
| SSURGO | Soil Survey Geographic |
| UFSAR | Updated Final Safety Analysis Report |
| UH | Unit Hydrograph |
| UHS | Ultimate Heat Sink |
| USACE | Unites States Army Corps of Engineers |
| USGS | Unites States Geological Survey |
| VBS | Vehicle Barrier System |
| yr | year(s) |

# 1. PURPOSE

## 1.1 Background

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

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

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

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

On behalf of Exelon Generation Co. (Exelon) for the Limerick Generating Station (LGS), this Flood Hazard Reevaluation Report (Report) provides the information requested in the March 12, 50.54(f) letter; specifically, the information listed under the 'Requested Information' section of Enclosure 2, paragraph 1 ('a' through 'e'). The 'Requested Information' section of Enclosure 2, paragraph 2 ('a' through 'd'), Integrated Assessment Report, will be addressed separately if the current design basis floods do not bound the reevaluated hazard for all flood causing mechanisms.

## 1.2 Requested Actions

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

For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard with the hazard evaluation. Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them. The scope of the integrated assessment report will include full power operations and other plant configurations that could be susceptible due to the status of the flood protection features. The scope also includes those

features of the ultimate heat sinks (UHS) that could be adversely affected by the flood conditions and lead to degradation of the flood protection (the loss of UHS from non-flood associated causes are not included). It is also requested that the integrated assessment address the entire duration of the flood conditions.

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

### 1.3 Requested Information

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

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

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

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

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

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

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

# 2. SITE INFORMATION

## 2.1 Detailed Site Information

The Limerick Generating Station is located in southeastern Pennsylvania on the Schuylkill River, about 1.7 miles southeast of the limits of the Borough of Pottstown and about 20.7 miles northwest of the Philadelphia city limits. The Schuylkill River passes through the site and separates the western portion, which is located in East Coventry Township, Chester County, from the eastern portion, which is partly in Limerick Township and partly in Lower Pottsgrove Township, both in Montgomery County, Pennsylvania. All of the major plant structures are located in the Limerick Township.

The natural ground elevations vary from 110 feet mean sea level datum (MSL) at the Schuylkill River to 280 feet MSL at the highest elevation. The present-day general location of the site is presented in Figure 2.1.1. The detailed site layout is presented in Figure 2.1.2.

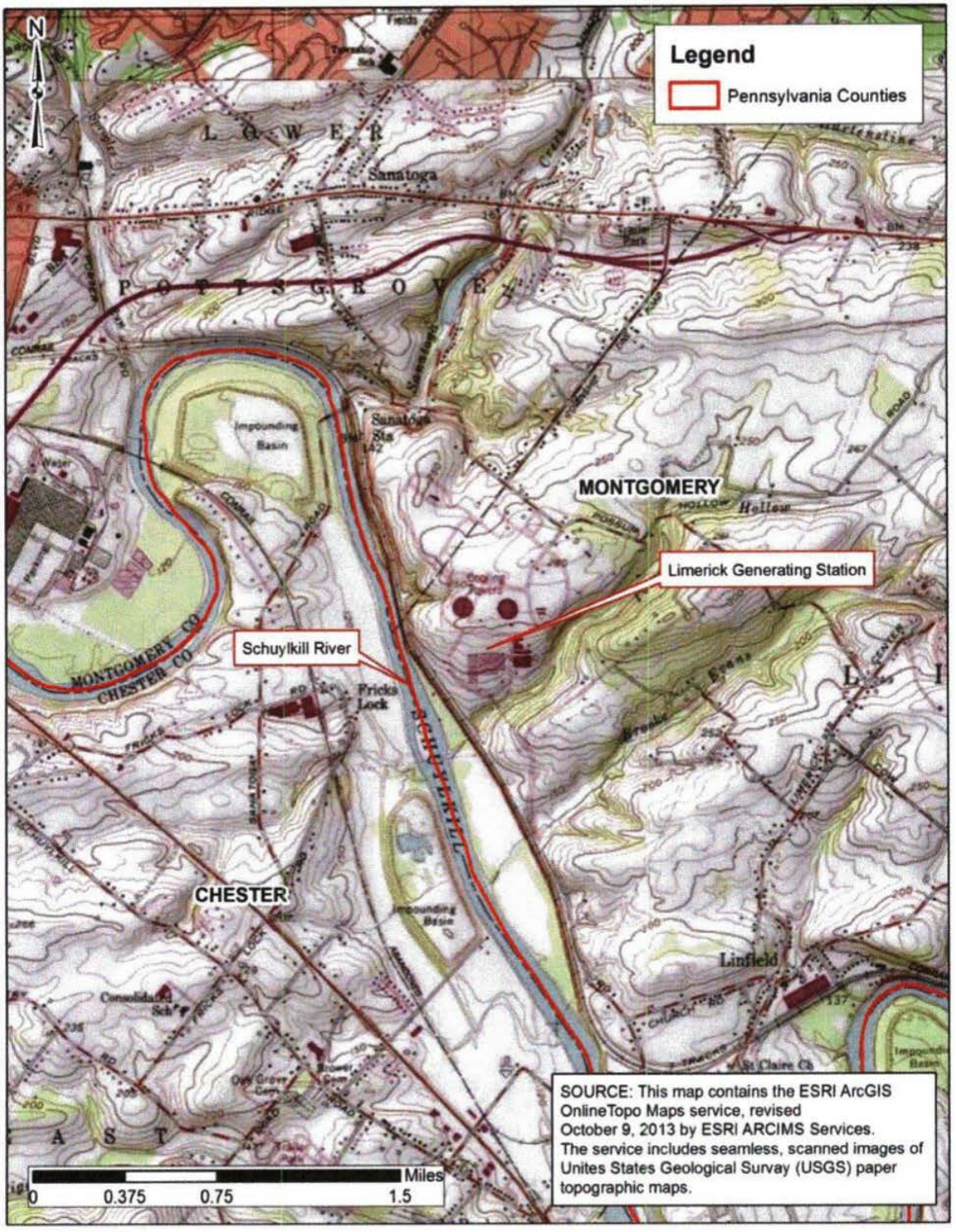


**Figure 2.1.1 – Present-Day General Site Map**

**Figure 2.1.2 – Present-Day Detailed Site Layout**

Safety-related elevations are as follows (UFSAR):

a. reactor enclosure
   1. personnel door to service water pipe tunnel - 201 ft
   2. personel door to outside - 217 ft
   3. personel door to radwaste enclosure or turbine enclosure - 217 ft
   4. equipment airlock - 217 ft
   5. railroad car airlock - 217 ft
   6. personnel door to turbine enclosure - 269 ft

b. control structure
   1. double doors to turbine enclosure - 200 ft
   2. double doors to turbine enclosure - 217 ft
   3. personel doors to turbine enclosure - 239 ft
   4. personel door to turbine enclosure - 254 ft
   5. personel doors to turbine enclosure - 269 ft
   6. personel door to turbine enclosure - 290 ft
   7. personel doors to turbine enclosure - 304 ft
   8. double doors to turbine enclosure - 304 ft

c. diesel generator enclosure
   1. personel doors to outside cell - 217 ft
d. Spray pond pump structure
   1. personel doors to outside - 268 ft
   2. roll-up doors to outside - 268 ft

The elevations at LGS are referenced to an MSL datum (UFSAR). It is determined that LGS MSL vertical datum is the same as National Geodetic Vertical Datum of 1929 (NGVD29) (Exelon 2014a).

## 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 LGS current licensing basis (CLB).

### 2.2.1 Effects of Local Intense Precipitation

The CLB uses a 72-hour probable maximum precipitation (PMP) for the site area equal to 39.7 inches as the LIP. The 72 hour PMP was divided into 6 hour increments, which were subsequently divided into 1 hour increments. The 1 hour PMP was further divided into 5, 10, and 15 minute increments. The maximum 6 hour increment is 26.8 inches. The maximum 1 hour increment is 10.2 inches.

The rational method is used to convert rainfall to runoff. The runoff coefficient was assumed to be 1. Kirpich's formula was used to estimate the times of concentration. All flow is assumed to be surface flow, over land or over roadway. Rating curves for flow over embankments or roadways were developed using an equation for critical flow or the broad crested weir equation.

The site is divided into three main functional areas: the turbine-reactor complex area, the cooling tower area, and the spray pond area. Runoff from the three main functional areas drains toward several low points, which in turn drain away from the site.

The spray pond spillway drains to a natural channel, across a road embankment, and then northward to Sanatoga Creek. The spray pond spillway is designed to pass a routed PMF (48 hour storm) preceded by an SPF (standard project flood) assumed equal to one-half the PMF.

The runoff from the north portion of the cooling tower area collects at a road junction, located at the northern central part of the cooling tower area. The runoff then drains westward along the roadway and the adjacent ditch. The entire runoff for the area passes through a low point in the roadway and enters a ravine that drains to the Schuylkill River. All the runoff from the drainage area is assumed to collect at the road junction. A discharge rating curve was used to determine water surface elevations at the road junction and along the roadway embankment between the two cooling towers to demonstrate runoff does not overtop the roadway fill and enter the turbine-reactor complex area.

Runoff drains into the turbine-reactor complex area from the southwest and eastern parts of the cooling tower area. The finished floor elevation of relevant safety-related structures at the power plant complex area (power block) is elevation 217 feet. Drainage is generally away from the structures. Jersey barriers have been strategically placed and are assumed to block the drainage flow, thus forming boundaries for the drainage areas. Backwater calculations performed for the

flow path (using the U.S. Army Corps of Engineers (USACE) HEC-RAS model) yielded a maximum water surface elevation of 218.6 feet along the northern edge of the Turbine Building (UFSAR).

If all roof drains and scuppers are blocked, water could pond on the roofs to a depth controlled by the height of the roof parapets. The highest parapet on any safety-related structure is less than the maximum 24 hour PMP of 34.4 inches. Assuming that some accumulation and overflowing occur, the maximum water depth could equal the height of the parapet plus a small amount of head providing flow over the parapet. The design roof load due to PMP for all the safety-related structures is equivalent to this maximum water depth (UFSAR).

Sanatoga Creek drains an area of less than 10 square miles, including most of the spray pond. Sanatoga Creek is isolated from the turbine-reactor area by the ridge containing the cooling towers. The ridge forms the drainage boundary between Sanatoga Creek and Possum Hollow Run. The SPF elevation of the Schuylkill River near its confluence with Sanatoga Creek is estimated to be 155 feet. The lowest elevation in the vicinity of the spray pond is elevation 240 feet, and the crest of the spray pond spillway is at elevation 251 feet. It is inconceivable that the water surface elevation in Sanatoga Creek, with backwater that is due to a concurrent SPF in the Schuylkill River, would rise higher than elevation 240 feet. It is considered unlikely that a PMF on the Schuylkill River would coincide with a PMF on the Sanatoga Creek. Therefore, it was concluded that a PMF in Sanatoga Creek would not endanger safety-related structures, and further detailed analysis was not considered necessary (UFSAR).

Possum Hollow Run has a drainage area of 1.3 square miles. It rises approximately 2.5 miles northeast of the site and flows southwesterly, entering the Schuylkill River through a gorge along the south side of LGS. The PMF is assumed to occur in Possum Hollow Run at the same time that an SPF is occurring in the Schuylkill River. The Schuylkill River SPF is assumed to be 50% of the PMF, or 250,000 cfs, which results in a Schuylkill River stage of elevation 152 feet. Using the slope-area method, a rating curve was developed for Possum Hollow Run at the point where the elevation 152 feet contour crosses the stream. A PMF hydrograph was developed for Possum Hollow Run using the procedure outlined by the U.S. Bureau of Reclamation in the Design of Small Dams. The resulting PMF hydrograph peak is 3,840 cubic feet per second (cfs). The water surface elevation corresponding to the peak PMF discharge is estimated to be 159 feet (UFSAR).

### 2.2.2 Flooding in Streams and Rivers

From Appendix B of Regulatory Guide 1.59, the PMF for the Schuylkill River at the LGS site, corresponding to a drainage area of 1,170 square miles, is 500,000 cfs. A discharge rating curve for the Schuylkill River near LGS was developed using observed flood levels, computer backwater studies, and slope-area methods. The PMF flood stage at the LGS site is elevation 174 feet.

### 2.2.3 Dam Breaches and Failures

Minor dams (less than 100 feet high and less than 4,000 acre-feet in volume) are either too small or too remote to cause significant flooding at LGS in the event of their seismic failure. Three major dams (Ontelaunee Dam, Maiden Creek Dam, and Blue Marsh Dam) are included in dam failure permutations. Total and instantaneous failure is considered simultaneously with the LGS SPF. The multiple failure analysis considered the peaks of all three structures to arrive at LGS simultaneously.

A hypothetical dam representing the volume of the three dams is placed downstream of the three dams. Using a slope-area method pre-existing SPF conditions in the river are estimated. The generalized dam failure approach developed by Sakkas was used to determine the maximum flood depth at LGS. Conveyance relations are used to relate the computed depth to the actual cross section. The peak flow rate that is due to simultaneous failure of the three dams is conservatively estimated at 762,000 cfs. When superimposed on the SPF flow of 250,000 cfs, the combined event results in a total flow rate of 1,012,000 cfs. This results in a water surface elevation of 201 feet at LGS (UFSAR).

One dam failure permutation is the postulated failure of Blue Marsh Dam superimposed on the SPF in the Schuylkill River. This results in a water surface elevation of [redacted] feet at LGS. The flood wave caused by the failure of Ontelaunee Dam is superimposed on the above condition, resulting in a water surface elevation of [redacted] feet at LGS. Thus, the rise in water surface elevation that is due to the failure of Ontelaunee Dam is [redacted] feet.

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

The assumption is made that a failure of Ontelaunee Dam also produces an incremental increase in stage of [redacted] feet at LGS during the PMF. Thus, the water surface elevation for the PMF plus a failure of Ontelaunee, is estimated as elevation [redacted] feet in the Schuylkill River. The maximum stage due to a PMF in Possum Hollow Run was not computed, but was found to be less than elevation 186 feet at a point nearly due east of the turbine-reactor area complex (UFSAR).

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

(b)(3):16 U.S.C. § 824o-1(d),(b)(4),(b)(7)(F)

The dam break waves are transients and do not contain enough volume to cover the entire Schuylkill Valley above LGS to the computed maximum depth at LGS. However, for an approximate wind-wave analysis, the following conditions are assumed; the water surface at LGS is elevation 201 feet, and wind velocity is 40 mph. Using curves from the USACE Shore Protection Manual, the significant and maximum wave runups were calculated to be 4.9 feet and 6.4 feet, respectively. Thus, the highest water surface elevation at LGS that is due to the most severe dam break permutation coincident with wave activity would be 207 feet (UFSAR).

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

### 2.2.4 Storm Surge & Seiche

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

### 2.2.5 Tsunami

The LGS CLB considers flooding due to tsunami as not applicable for the site (UFSAR).

### 2.2.6 Ice Induced Flooding

The LGS CLB considers flooding due to ice effects as not applicable for the site (UFSAR).

### 2.2.7 Channel Migration or Diversion

The LGS CLB considers flooding due to channel migration or diversion as not applicable for the site (UFSAR).

### 2.2.8 Low Water Considerations

Extreme low flow in streams does not affect the ability of any safety-related facilities to perform adequately, including the UHS (spray pond) (UFSAR).

### 2.2.9 Combined Effect Flood

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

## 2.3 Flood Related Changes to the License Basis

LGS has been evaluated to adequately withstand the effects of flooding in the current licensing basis. During upgrades to the site that may create an impediment of outflow of precipitation onsite, the effects of local intense precipitation is re-evaluated in accordance with approved calculation methodology and appropriately incorporated into the licensing basis. However, there have been no flood related changes to the license basis.

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

The LGS watershed has a total area of approximately 1,167 square miles. Based on aerial images of the watershed, the changes to the watershed include land development within the watershed area, which is a very small percentage of the overall watershed area. The changes to the local area sub-watershed for LGS include buildings, parking lots, and security barrier upgrades that have been added to the site since license issuance.

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

The water surface (elevation 201 feet) resulting from the dam break analysis is transient. It is estimated that the time required for the dam break flood wave at the river cross-section at LGS to rise above and subside back to the SPF elevation (152 feet) is approximately 7.5 hours. The length of time for which the flood wave will be above elevation 177 feet and elevation 195 feet would be about 4.5 and 2 hours, respectively (UFSAR Section 2.4.2.2).

The shortest distance from the elevation 201 feet (dam break flood elevation without wave run-up) contour to the nearest safety-related structure inland (diesel oil storage tanks) is about 126 feet. The foundation of these structures is at elevation 194 feet. For flood waters to reach these structures, percolation through the embankment would have to occur. An analysis of the percolation was made assuming Darcy's law is valid and that flow is one-dimensional (UFSAR Section 2.4.2.2).

The base of the porous medium (base of embankment material) was assumed to rise linearly from elevation 170 feet at the toe of the embankment to elevation 194 feet at the tanks. The results of the analysis indicate that, if a conservative permeability of $5 \times 10^4$ ft/yr is assumed and if the flood crest remains at elevation 202 feet (UFSAR Section 2.4.3 shows maximum flood elevation of 201 feet) for a duration of two hours, groundwater would not be above elevation 194 feet anywhere within 80 feet of the tanks. Therefore the dam break flood wave would not affect hydrostatic pressures on the foundations of safety-related structures (UFSAR Section 2.4.2.2).

An engineering evaluation of the site drainage conditions determined that the only potentially adverse effect is that flood water due to the local intense precipitation (PMP) event or the postulated cooling tower basin failure (PCTBF) could enter the turbine enclosure for a limited period of time. At certain site locations, the PCTBF event results in higher flood levels than the PMP event, but the PMP event is the bounding event due to the longer duration of the event. Consequently, measures are taken to limit the possible flow of flood water from the turbine enclosure into the control enclosure where the safety-related chilled water system is located at elevation 200 feet. These measures include controlling openings in the lower portion of the turbine enclosure walls, relying on flood control

features within the turbine building, and by limiting the flow of water through the turbine enclosure doors by providing additional curbs and barriers inside the turbine enclosure and by administratively controlling the opening of certain doors. As a result of these measures, flooding into the control enclosure due to the present site drainage condition results in control enclosure flood level below the elevation of the safety-related chilled water system (UFSAR Section 2.4.2.3.3).

Furthermore, administrative controls have been implemented to ensure that future changes in site conditions, affecting flood water run-off, will receive engineering evaluation and approval prior to implementation (UFSAR Section 2.4.2.3.3).

# 3. SUMMARY OF FLOOD HAZARD REEVALUATION

Flooding hazards from various flood-causing mechanisms are evaluated for LGS 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* (NRC NUREG/CR-7046), the Hierarchical Hazard Assessment (HHA) approach is utilized in the reevaluation study. The HHA approach is a progressively refined, stepwise estimation of site-specific hazards that evaluates the safety of SSCs with the most conservative plausible assumptions consistent with available data. Consistent with the HHA approach, flooding mechanisms that are determined to be not applicable for the site are screened out using qualitative and quantitative assessments with conservative, simplified assumptions and/or physical reasoning based on physical, hydrological and geological characteristics of the site. For the flooding mechanisms that can potentially affect the design basis, detailed analyses are performed based on present-day methodologies, standards and available data.

This section describes in detail the reevaluation analysis performed for each plausible flooding mechanism: flooding due to the local intense precipitation, flooding in streams and rivers, dam failure, ice induced flooding, channel diversion, and combined effects. Bases for screening-out other flood-causing mechanisms are also provided.

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

- NRC Standard Review Plan, NUREG-0800, revised March 2007 (NRC NUREG-0800);
- NRC Office of Standards Development, Regulatory Guides, RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976 (NRC RG 1.102);
- RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977 (NRC RG 1.59);
- NUREG/CR-7046 "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," dated November 2011 (NRC NUREG/CR-7046);
- NUREG/CR-6966 "Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America" dated March 2009 (NRC NUREG/CR-6966);
- American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS-2.8-1992), dated July 28, 1992;
- NRC JLD-ISG-2012-06, "Guidance for Performing a Tsunami, Surge and Seiche Flooding Safety Analysis", Japan Lessons-Learned Project Directorate Interim Staff Guidance, Revision 0 (NRC JLD-ISG-2012-06).

The following provides the flood causing mechanisms and their associated water surface elevations that are analyzed in the LGS 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 and the effects of the LIP, which are the resulting impacts from the flood water surface elevations and flow depths, are determined in Calculation LM-0699, Local Intense Precipitation (LIP) – Fukushima Flood Hazard Assessment (Exelon 2014e). The effects of the LIP are computed for the safety-related structures at LGS. The assumptions associated with LIP are listed in Calculation LM-0699.

### 3.1.1 Inputs

The inputs for the analysis are described below.

#### 3.1.1.1 Local Intense Precipitation

The LIP at LGS is performed in Calculation LM-0698, Probable Maximum Precipitation (PMP) – Fukushima Flood Hazard Assessment (Exelon 2014d). The LIP estimates are derived using the generalized hydrometeorological study (HMR No. 51 and HMR No. 52). Point (1-sq.mi.) PMP depths for the LGS are equal to 17.9 inches and 26.9 inches for 1-hour and 6-hour, respectively. The rainfall hyetograph distribution used is based on Figure B-5 of NUREG/CR-7046 (NRC NUREG/CR-7046), which suggests a front loaded rainfall distribution.

While HMR 52 does not specifically state the time intervals be arranged in this particular order, with the typical west-east flow across North America, the type of storm set-up that would provide a LIP would likely be a mesoscale convective system (such as a squall line for example). Using the conceptual model of this type of system, the initial precipitation is associated with the mature cells and a zone of convergence and as such will be very intense. The storm motion and nature of the system would then see a decrease in the precipitation after the initial burst as the rear trailing stratiform region with the cold pool moves over the area. This type of meteorological system fits with the front loaded distribution (Exelon 2014j).

#### 3.1.1.2 Ground Surface Topography

Site topography is based on the site topographic survey data (Exelon 2014l) for the secure area and power block. Adjacent areas outside the secure area are based on Light Detection and Ranging (LiDAR) elevation points produced by the PAMAP Program for Pennsylvania spatial data (Exelon 2014l). A single digital terrain model (DTM) is developed in ArcGIS 10.1 from the topographic survey data and the LiDAR data.

#### 3.1.1.3 Manning's Roughness Coefficients

Manning's Roughness Coefficients are assigned based on the land cover types selected from visual assessment of aerial imagery (ESRI 2014) and field observations.

### 3.1.2 Methodology

The Effects of LIP analysis uses a two-dimensional (2D) hydrodynamic model, the FLO-2D model Build 13-11-06 (FLO-2D). FLO-2D is a physical process model that routes flood hydrographs and rainfall-runoff over unconfined flow surfaces using the dynamic wave approximation to the momentum equation. FLO-2D moves flood volume on a series of tiles (grids) for overland flow. Overland flood routing in two-dimensions is accomplished through a numerical integration of the equations of motion and the conservation of fluid volume. Channel flow routing is not included.

A two-dimensional model is appropriate and a better suitable model compared to a one-dimensional model to simulate the overland flow conditions at the site, which are sheet flow and shallow open channel flow. The two-dimensional model determines the flow direction based on

the ground topography when in the one-dimensional model the flow direction has to be assigned. The two-dimensional model uses a grid to represent the ground surface. Each grid element is treated as a computational cell and the hydraulic relationships are determined for each cell depending on the hydrologic and hydraulic properties of the cell itself and the surrounding cells. The ground is closely represented in the two-dimensional model because each grid element is assigned a corresponding ground surface elevation and roughness coefficient. Buildings are modeled as elevated grid elements to conservatively ensure rainfall runs off the building rooftops to surrounding areas. The vehicle barrier system (VBS) and other walls are modeled as levee structures. A sensitivity analysis is performed to examine the consequences of the presence or absence of the VBS.

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

- Delineate FLO-2D Model computational boundary.
- Generate FLO-2D Model components (e.g., ground elevations, buildings, levees, etc.).
- Calculate Manning's n roughness coefficients.
- Assign rainfall hyetographs.
- Review supercritical flow locations.
- Perform LIP-induced flood simulation.

Following the guidance outlined in NUREG/CR-7046 the runoff losses are ignored. Roof drains and parapet walls are not considered. The roof rainfall is assumed to be contributing to the overland runoff. The site drainage network of any culverts and storm sewers is assumed to be non-functional at the time of the LIP event.

The 1-hour, 1-sq.mi. LIP is equal to 17.9 inches, and the 6-hour 10-square mile PMP is equal to 26.9 inches for LGS (Exelon 2014d).

The model computational boundary is based on the site topography, anticipated location of model boundary conditions, and the local drainage area contributing to LGS. The local drainage area for LGS is delineated based on LGS topographic information (Exelon 2014l).

The computational domain of the FLO-2D model extends to the steeper slopes beyond the VBS to the south and west. The north boundary is located beyond the cooling towers and spray pond. The east boundary is located beyond the plant access road adjacent to Possum Hollow Run.

Outflow grid elements allow discharge off the FLO-2D grid system without affecting the water surface elevation at grid elements of interest by placing them far enough from the area of interest. Grid elements along the computational boundary are assigned as outflow grid elements to prevent ponding of water.

The elevations at LGS are referenced to MSL datum in the UFSAR. Based on available information it is assumed the MSL datum referenced in the UFSAR is equivalent to NGVD29. The elevations in the topographic survey and the LiDAR data are referenced to North American Vertical Datum of 1988 (NAVD88). All data points are shifted from NAVD88 to NGVD29 using the web-based program VERTCON.

### 3.1.3 Results

Maximum LIP flood elevations at critical doors of the power block range from 217.1 feet (NGVD29) at a diesel generator door on the south side of the main building to 218.4 feet (NGVD29) at the north exterior door to the turbine building. Maximum velocities at critical doors

range from 0.3 feet per second (fps) to 3.0 fps. Maximum flow depths at critical doors range from 0.4 to 2.3 feet, with the highest depths occurring at the north exterior doors to the turbine building. The sensitivity analysis of the presence or absence of the VBS indicates the presence of the VBS results in the most conservative LIP depths in the secure areas.

### 3.1.4 Conclusions

1. The maximum water surface elevations at all locations on the site due to the LIP at LGS result from a PMP depth of 17.9 inches within an hour and 26.9 inches within 6 hours.
2. The maximum water surface elevation at critical doors of the power block is 218.4 feet NGVD29 at the north exterior door to the turbine building.

The maximum reevaluated water surface elevation due to the LIP at LGS equal to 218.4 feet NGVD29 does not exceed the CLB maximum LIP water surface elevation equal to 218.6 feet NGVD29.

## 3.2 Probable Maximum Flood in Streams and Rivers

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

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

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

Alternative 1 – Combination of:

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

Alternative 2 – Combination of:

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

Alternative 3 – Combination of:

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

The precipitation input for Alternative 1, the all-season PMF, and Alternatives 2 and 3, the cool-season PMF, are determined in Calculation LM-0698, Probable Maximum Precipitation (PMP) - Fukushima Flood Hazard Assessment (Exelon 2014d).

The PMF peak discharges and flow hydrographs for the Schuylkill River, Possum Hollow Run, and Sanatoga Creek are determined in Calculation LM-0700, Probable Maximum Flood - Hydrology - Fukushima Flood Hazard Assessment (Exelon 2014f). The PMF water surface elevations for the Schuylkill River, Possum Hollow Run, and Sanatoga Creek are determined in Calculation LM-0701, Probable Maximum Flood - Hydraulics - Fukushima Flood Hazard Assessment (Exelon 2014g).

### 3.2.1 Inputs

The inputs for the analysis are described below.

#### 3.2.1.1 Watershed Delineation

Subwatershed delineations are developed using U.S. Geological Survey (USGS) Pennyslvania StreamStats internet-based application (USGS 2013b) and existing USGS topographic data in the form of 10-foot contour topographic maps.

#### 3.2.1.2 All-Season PMP

The all-season PMP estimates for LGS 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.

#### 3.2.1.3 Cool-Season PMP

The cool-season PMP estimates for LGS are derived based on the charts presented in the generalized seasonal hydrometeorological report HMR No. 53 (HMR No. 53). The cool-season PMP estimates are derived based on the site location and all-season PMP/seasonal PMP ratios.

#### 3.2.1.4 100-Year Rainfall

The 100-year rainfall estimates are obtained from National Oceanic and Atmospheric Administration (NOAA) Precipitation Frequency Data Server (NOAA 2013e) for the LGS location.

#### 3.2.1.5 Hourly Dew Point Temperature and Wind Speed Data

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

#### 3.2.1.6 Snow Data

Daily Snow Depth data at selected Global Historical Network provided by NCDC (NOAA 2013b).

#### 3.2.1.7 Land Cover Data

Forest cover information based on land use data from the USGS National Land Cover Database for Pennsylvania (USGS 2011).

#### 3.2.1.8 Mean Monthly Baseflow

Baseflow is calculated for the Schuylkill River sub-watersheds using selected USGS Surface-Water Monthly Statistics data (USGS 2014). The Possum Hollow Run and Sanatoga Creek watersheds are ungaged. Baseflow is determined using ratios of drainage areas with surrogate gaged sub-watersheds of the Schuylkill River.

#### 3.2.1.9 Soil Data

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

#### 3.2.1.10 Surface Roughness Coefficients

Manning's n values for the Schuylkill River, Sanatoga Creek, and Possum Hollow Run are initially developed using orthoimagery photos (ESRI 2013b) and using published guidance for selection of Manning's Roughness Coefficients based on "Open-Channel Hydraulics" (Chow 1959). The Schuylkill River HEC-RAS model, including adjustment of roughness coefficients, is calibrated using observed historical water surface elevations from USGS gage data (Exelon 2014g).

#### 3.2.1.11 Ground Surface Topography

Cross section locations were established at locations where significant changes occur in slope, shape or roughness within the river and overbank areas, or where bridges and dams cross the water course. Floodplain geometry for overbank areas of the Schuylkill River, Sanatoga Creek, and Possum Hollow Run is developed using Digital Elevation Model topographic information from the Pennsylvania Spatial Data Access (PASDA) (PAMAP 2008).

The Schuylkill River channel was conservatively represented as a triangle shaped channel based on available channel invert elevations contained in the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for Berks, Montgomery, and Philadelphia counties (FEMA 2012, FEMA 2001, and FEMA 2007). Bathymetry information on Sanatoga Creek and Possum Hollow Run are conservatively ignored.

#### 3.2.1.12 Bridge Data

Information for Schuylkill River bridge structures is determined from Pennsylvania Department of Transportation drawings and the FEMA hydraulic model for the Schuylkill River. Bridge information for the two bridges over Sanatoga Creek is obtained from the published FEMA FIS profile (FEMA 2001). Bridge information for the railroad bridge over Possum Hollow Run is obtained from LGS drawings (LGS 1983).

### 3.2.2 Methodology

The PMP for the Schuylkill River, Sanatoga Creek, and Possum Hollow Run watersheds are determined using BOSS HMR52 computer software. The PMF peak discharges and flow hydrographs for the Schuylkill River, Sanatoga Creek, and Possum Hollow Run are determined using USACE HEC-HMS version 3.5 computer software. The PMF water surface elevations for the Schuylkill River, Sanatoga Creek, and Possum Hollow Run are determined using USACE HEC-RAS version 4.1 computer software. Detailed descriptions of the methods utilized in the PMF analyses are presented below.

#### 3.2.2.1 Watershed Delineation

The Schuylkill River watershed at LGS is delineated using the USGS StreamStats (USGS 2013b). The watershed delineation, including five sub-watersheds, is verified using USGS topographic maps and published USGS gage drainage areas. The Sanatoga Creek and Possum Hollow Run watersheds are delineated in the same manner with the exception that there are no USGS gage drainage areas for comparison.

#### 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. The all-season PMP estimates for LGS 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 location and watershed area.

The all-season PMP estimates for the Schuylkill River, Sanatoga Creek, and Possum Hollow Run watersheds are derived using the BOSS HMR52 software. The depth-area-duration values for 6, 12, 24, 28 and 72-hours and 10, 200, 1,000, 5,000, 10,000 and 20,000 $mi^2$ (HMR No. 51, Figures 18 through 47) are determined. These values are input into the BOSS HMR52 software. Using the methods presented in HMR No. 51 and HMR No. 52, this program utilizes algorithms to calculate maximum rainfall depths and hyetographs for a given watershed. BOSS HMR52 is an enhanced version of an original program HMR52 developed by the USACE.

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

#### 3.2.2.3 Alternative 2 Precipitation and Snow Water Equivalent 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 2013e) for the LGS location. Conservatively, all-season mean values are utilized without adjustment for lesser cool-season values.

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).

Temperature of saturated air (i.e., dew point) is determined using historical climatological data available in the watershed (NOAA 2013c). The snowmelt estimate conservatively used average monthly maximum dewpoint values. Monthly maximum dewpoint data points are extracted from recorded hourly data, for cool-season months (November through April) at the selected station and then used to calculate an average monthly maximum for each month. Dewpoint data are considered only for those periods coincident with rainfall to reflect realistic values for rain-on-snow periods. Note that the temperature of saturated air in the snowmelt equation is measured at 3 meters (10 feet) above the snow surface. However, according to NOAA's National Climatic Data Center, the temperature of saturated air is measured approximately 6 feet above the ground. As a conservative approach, the temperature of

saturated air input in the snowmelt calculations is the temperature values measured by the climatological gage station (i.e., the 6 feet above ground level is typically slightly warmer than the 10 feet above ground level).

Wind velocity is calculated using historical climatological data (NOAA 2013c) available from the same gage used for the dewpoint calculation. The snowmelt calculation used average monthly wind velocity values. The average wind velocity is calculated based on recorded hourly data, for cool-season months (November through April) at each selected station for each month.

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. The months of May through October are not included as months that could generate significant snowmelt due to the low or absent values of recorded maximum monthly snow cover.

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 and Snow Water Equivalent Input

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

The cool-season PMP estimates for LGS are determined using the charts provided in Hydrometeorological Report No. 53 for each cool-season month (November to April). The cool-season monthly PMPs are calculated using HMR No. 53 to develop the seasonal PMP depths for the 72-hour, 10 square mile storm by month of occurrence. HMR No. 53 does not provide guidance for watershed areas other than 10 square miles. Thus, the seasonal variation of the PMP (i.e., all-season PMP/seasonal PMP ratio) is considered to be constant for other watershed sizes, to calculate the seasonal PMP depths for the watershed and sub-watersheds based on the 10-mi$^2$ PMPs presented in HMR No. 51 and HMR No. 53.

The snowmelt rate is determined for each cool-season month using the energy budget equation for the rain-on-snow condition (USACE 1998). The meteorological parameters used as the input to the energy budget equation, dew point temperatures and wind speeds, are determined from the historical data obtained from the meteorological stations in the watersheds evaluated.

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 the watersheds evaluated.

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

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

1. The Soil Survey Geographic (SSURGO) maps for counties in the watershed were downloaded from the NRCS website (NRCS 2013). For counties without SSURGO maps, data are acquired through the PASDA website. The soil database contains information for each delineated soil type (or classification), represented by a GIS shape file.
2. The hydrologic soil group (HSG) classification (A, B, C, or D from lowest runoff potential to highest runoff potential) is determined from the soil database (NRCS 2013). Distinct shapefiles are created in ArcGIS for each HSG within each subwatershed.
3. The minimum initial infiltration rate associated with each HSG classification is conservatively utilized (Maidment 1993). Area-weighted constant infiltration rates for each subwatershed are calculated for each soil group within each watershed.

Constant loss rates are developed using an "iterative process" process during the calibration and verification of the watershed runoff model and applied only to Alternative 1. As a conservative bounding approach to account for frozen ground in the cool season months, a constant loss rate of zero is used for Alternative 2 and Alternative 3.

A more conservative rainfall-runoff simulation approach is used for the Sanatoga Creek and Possum Hollow Run watersheds. A maximized curve number (CN) representing impervious cover (i.e., maximum runoff) is adopted for the two nearby streams.

#### 3.2.2.6 Unit Hydrograph

The Snyder Unit Hydrograph (UH) transform method is used for the Schuylkill River subwatersheds. To account for nonlinear hydrologic response, the calculated UH for each sub-watershed is adjusted by increasing the peak discharge by one-fifth and decreasing the time-to-peak by one third as per NUREG/CR-7046.

The NRCS Dimensionless Unit Hydrograph transform method is used for Sanatoga Creek and Possum Hollow Run. A nonlinear hydrologic response adjustment is not applied because the minimum acceptable lag time is conservatively selected to maximize the peak flow rate potential.

#### 3.2.2.7 River Routing

Reach routing conveys flows downstream and accounts for travel time and flow translation within the river. The Muskingum reach routing method is used for the lower portion of the Schuylkill River watershed between USGS gages at Reading and Pottstown. As a conservative measure, upstream areas of the watershed are not channel routed.

#### 3.2.2.8 HEC-HMS Model Calibration

Model calibration is the process of selecting and refining HEC-HMS input parameters to produce a simulated hydrograph for a given flood that shows good agreement with an observed hydrograph for the same flood. Model verification is the process of testing the calibrated HEC-HMS input parameters to demonstrate the modeled hydrograph for a given flood shows good agreement with the observed flood hydrograph.

USGS daily and hourly peak stream flow data for the gaged subwatersheds of the Schuylkill River are used to identify candidate storms, representing the Alternative 1 PMF combination. Multiple calibration events including the flood of record at Pottstown in June 1972 (Hurricane Agnes) are used as calibration events. The other calibration events occurred during October 1996, September 1999, and September 2011. The verification events occurred during June 2003, August 2004, September 2004, June 2006, and August 2011.

A HEC-HMS simulation for each gaged subwatershed is performed for the calibration floods using initial user-input calculated unit hydrograph parameters, initial loss, and constant loss rates beginning with the most upstream gaged subwatershed of the Schuylkill River. Reach routing travel time is initially calculated by computing the travel time for the peak of the hydrograph to arrive at the next downstream gage for a historic flood. The initial reach routing weighting factor is selected arbitrarily. Parameter variation is performed for the calibration floods for each of the subwatersheds by varying the parameters through a trial-and-error method. The calibration is performed iteratively until the calibrated peaks are similar but conservative in comparison to the observed peaks. A bounding range of calibrated parameters are then used as the initial parameters for the verification simulations. HEC-HMS simulations for each gaged subwatershed are performed for the verification floods using parameters within the bounding range.

Input parameters for ungaged subwatersheds are based on the parameters from surrogate subwatersheds judged to have similar hydrologic characteristics.

#### 3.2.2.9 HEC-HMS Dam Modeling

Dams are not incorporated into the HEC-HMS modeling for the PMF. Unit hydrograph parameters, lag times, and constant loss rates are calculated through direct calibration of the June 1972 storm. This approach is conservative because it does not account for the attenuation effects of dams constructed after the storm calibration event. Attenuation effects of dams constructed after the storm calibration event are incorporated into the calibrated parameters.

#### 3.2.2.10 Storm Center

Storm center variation is performed for the Schulykill River using the centroid of the overall watershed and the centroid of each of the five subwatersheds. Storm centering variation is not performed for Sanatoga Creek or Possum Hollow Run given the small watershed areas.

3.2.2.11 Temporal Distribution

The temporal order of the PMP is considered using four distributions. The default temporal order of the twelve 6-hour increments is: 12, 10, 8, 6, 4, 2, 1, 3, 5, 7, 9, 11, where the numbers indicate the largest precipitation intensity (1) to the smallest (12). The distribution variations are substituting the maximum 6-hour increment at the 9th position, shifting the four maximum 6-hour increments to the first 24-hour period, and shifting the four maximum 6-hour increments to the final 24-hour period.

3.2.2.12 Hydrologic Modeling

Hydrologic modeling is performed using USACE HEC-HMS computer software. Base flow is included as a constant flow rate. The various precipitation estimates are applied to the Schuylkill River watershed model for each alternative. Unit hydrographs adjusted for the effects of nonlinear basin response transform rainfall to runoff.

Based on comparison of precipitation estimates, Alternative 1 provides significantly greater precipitation than combined rainfall and snowmelt for Sanatoga Creek and Possum Hollow Run. Therefore, only Alternative 1 precipitation estimates are applied to the Sanatoga Creek and Possum Hollow Run watershed models. As previously discussed, unit hydrograph rainfall-runoff transformation is maximized for Sanatoga Creek and Possum Hollow Run.

The HEC-HMS models produce flow hydrograph results for each of the streams and rivers for future use in the hydraulic models.

3.2.2.13 Hydraulic Modeling

Hydraulic modeling for the Schuylkill River, Sanatoga Creek, and Possum Hollow Run is performed using USACE HEC-RAS computer software. Cross sections for the floodplains are developed from the digital elevation model (DEM) data using ArcGIS. FEMA river bed elevations and PAMAP DEM data are used to develop the Schuylkill River cross sections assuming a triangular shape. The bathymetry for Sanatoga Creek and Possum Hollow Run are conservatively ignored. Bridge structures are added into the HEC-RAS hydraulic models using bridge plans obtained from the Commonwealth of Pennsylvania Department of Transportation (PennDOT) and LGS. The bridges for Possum Hollow Run and Sanatoga Creek are conservatively modeled as dams with no conveyance below the roadway grade. Manning's roughness values and ineffective flow areas are added to appropriate cross sections and bridges. The Schuylkill River HEC-RAS model is calibrated using observed historical water surface elevations from USGS gage data. HEC-RAS steady-state modeling is used to determine the water surface elevation at each cross section by analyzing the peak flow obtained from the HEC-HMS hydrologic models.

### 3.2.3 Results

The Alternative 1 PMF combination produces the maximum effects. The maximum PMF water surface elevation for the Schuylkill River at LGS based on the peak discharge of 397,200 cfs is 163.9 feet NGVD29. The maximum PMF water surface elevation for Sanatoga Creek at LGS based on the peak discharge of 83,500 cfs is 153.7 feet NGVD29. The maximum PMF water surface elevation for Possum Hollow Run at LGS based on the peak discharge of 17,300 cfs is 167.8 feet NGVD29. These maximum PMF water surface elevations do not include the effects of dam failure or the effects due to the wind activities. The maximum water surface elevations due to dam failure are discussed in Section 3.4, Dam Failure. The maximum water surface elevations due to the wind-generated wave are discussed in Section 3.5, Combined Events.

### 3.2.4 Conclusions

The Schuylkill River CLB maximum PMF water surface elevation at LGS is 174 feet NGVD29. The reevaluated maximum water surface elevation due to the Schuylkill River PMF at LGS is equal to 163.9 feet NGVD29. The reevaluated maximum water surface elevation is 10.1 feet below the Schuylkill River CLB maximum water surface elevation.

The Sanatoga Creek CLB maximum PMF water surface elevation at LGS is not identified in the UFSAR. The reevaluated maximum water surface elevation due to the Sanatoga Creek PMF at LGS is equal to 153.7 feet NGVD29. The reevaluated maximum water surface elevation is 20.3 feet below the Schuylkill River CLB maximum water surface elevation.

The Possum Hollow Run CLB maximum PMF water surface elevation at LGS is 159 feet NGVD29. The reevaluated maximum water surface elevation due to the Possum Hollow Run PMF at LGS is equal to 167.8 feet NGVD29. The reevaluated maximum water surface elevation is 8.8 feet above the Possum Hollow Run CLB maximum water surface elevation. However, the reevaluated maximum water surface elevation is 6.2 feet below the Schuylkill River CLB maximum water surface elevation and does not affect the lowest grade level entrance to any safety-related structure of 217 feet NGVD29.

## 3.3 Storm Surge, Seiche, and Tsunami Screening

The Probable Maximum Storm Surge (PMSS), Probable Maximum Seiche (PMS), and Probable Maximum Tsunami (PMT) are analyzed in Calculation LM-0695, Probable Maximum Surge, Seiche, and Tsunami – Fukushima Flood Hazard Assessment (Exelon 2014a).

Storm surge, seiche, and tsunami evaluations are performed following the guidance outlined in NUREG/CR-7046, JLD-ISG-2012-06, and NUREG/CR-6966. NUREG/CR-7046, Appendix H.4, describes the combined events criteria for an enclosed body of water, which is appropriate for analyzing surge and seiche flooding along the Schuylkill River and the spray pond at LGS. NRC JLD-ISG-2012-06 requires that "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." LGS is not a coastal location; however, the Schuylkill River and the spray pond at LGS are analyzed to determine if they could be subjected to storm surge (wind setup and wave setup) due to severe wind storms. The assumptions associated with Storm Surge, Seiche, and Tsunami screening are listed in Calculation LM-0695 (Exelon 2014a).

### 3.3.1 Inputs

Inputs for the analysis in addition to the site location and plant grade elevation of 217 feet NGVD29 are provided below.

#### 3.3.1.1 Historical Tsunami Data

The Global Historical Tsunami Database (NOAA 2013d), maintained by the National Oceanic Atmospheric Administration's National Geophysical Data Center (NGDC), is reviewed to determine the history of tsunamis in the region. The NGDC tsunami-source-event database is global in extent with information dating from 2000 Before Common Era (B.C.E.) to the present.

#### 3.3.1.2 USGS Earthquake Data

The USGS Earthquake Hazard Data (USGS 2013a) is reviewed to determine the history and intensity of earthquakes in the region.

### 3.3.2 Methodology

The methodology applied to each mechanism is described below.

#### 3.3.2.1 Storm Surge

The distance to an open coast as well as the site elevation relative to sea level and the level of other nearby water bodies are documented to demonstrate that surge is not applicable at LGS. The physical setting of the river is also evaluated to demonstrate that surge is not applicable to the Schuylkill River.

#### 3.3.2.2 Seiche

A description of the site's physical setting is used to demonstrate that the effects of a potential seiche in the Schuylkill River will be physically confined by the geometries of the floodplains. Seiche potential is also analyzed for the spray pond at LGS. The natural periods of oscillation of the spray pond are determined from an analytical solution to the primitive equations for fluid motions in an idealized basin. In accordance with NUREG/CR-7046, Merian's formula for an enclosed basin is used to evaluate the primary seiche mode in the spray pond.

Meteorological, seismic forcing mechanisms, as well as wind generated waves can cause waves in a semi-enclosed or enclosed body of water. However, for seiches to present a flood risk to LGS, the forcing mechanisms should resonate with the natural frequency of the surface water bodies. The periods of external forcing mechanisms are developed based on reported ranges for these mechanisms.

#### 3.3.2.3 Tsunami

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

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

A regional survey considered tsunami-like waves in the area around LGS, extending from 34° to 46° N Latitude and 69° to 81° W, an approximately 800 mile by 600 mile area. The regional survey and assessment include the potential far-field sources and mechanisms that generate tsunamis. The Global Historical Tsunami Database (NOAA 2013d), maintained by the NGDC, is reviewed to determine the history of tsunamis in the region. An assessment of the mechanisms likely to cause a tsunami is also performed.

A substantial amount of slip and a large rupture area is required to generate a major tsunami. Generally, large earthquakes with magnitudes greater than 6.5 generate observable tsunamis (NRC NUREG/CR-6966, Section 1.3.1). The USGS Pennsylvania Earthquake History website (USGS 2012) is reviewed to determine the largest earthquake in Pennsylvania history.

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

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

The results from the regional survey are used to identify the primary effects of a potential tsunami wave near LGS. A site screening evaluation is then performed on the basis of available vertical margin and distance from tsunami-prone areas to determine the potential effects to LGS.

### 3.3.3 Results

#### 3.3.3.1 Storm Surge

Flooding of LGS due to storm surge is not anticipated because LGS is not located near a water body subject to coastal flooding. The spray pond is not subject to storm surge due to its limited fetch.

#### 3.3.3.2 Seiche

The site setting of LGS, elevation margin from the Schuylkill River, and the alignment of the Schuylkill River in the vicinity of LGS preclude flooding of the primary LGS site due to seiche. Seiche in the spray pond at LGS is not expected to affect the site due to the natural periods of the pond not coinciding with the periods of external forcing mechanisms, limited volume of water in the pond, and topographic relief on the north side of the pond which is away from the site.

#### 3.3.3.3 Tsunami

The most significant earthquake to occur in the region is 5.5 magnitude in Rockaway Beach, New York in 1884. The USGS Pennsylvania Earthquake History website (USGS 2012) shows that the largest earthquake in Pennsylvania occurred in 1840 and had a magnitude of 5.2. These earthquakes do not meet the threshold for tsunami generation.

The river channel in the vicinity of LGS is judged unlikely to generate a subaqueous landslide which would result in a tsunami-like wave that could affect the LGS site. Landslides that may occur at other steeper sections of the Schuylkill River distant from LGS would not result in tsunami waves with the potential to affect LGS.

The geographical areas where subaerial landslides occur are generally limited to areas of steep shoreline topography. LGS is located within an area considered to have low landslide incidence.

The screening analysis for tsunami concluded that subaerial landslide incidence is low and the potential for subaqueous landslide with a significant velocity is low. While there is a

potential for tsunami in the region near the Great Lakes and Atlantic coast, the site screening for tsunami concluded that flooding of LGS due to tsunami is not anticipated.

### 3.3.4 Conclusions

Based on the screening analysis performed, surge, seiche, and tsunami are not considered to be potential contributors to flooding at LGS. This is consistent with the CLB.

## 3.4 Dam Failure

The dam failure analysis is performed in Calculation LM-0702, Dam Failure - Fukushima Flood Hazard Assessment (Exelon 2014h). The assumptions associated with the dam failure analysis are listed in Calculation LM-0702.

### 3.4.1 Inputs

The inputs for the analysis are described below.

#### 3.4.1.1 Data for Upstream Dams

Dam height, reservoir storage capacity, and locations of the dams in the upstream watershed of LGS are obtained from the National Inventory of Dams (USACE NID) database maintained by the USACE. Data for the proposed Maiden Creek Dam is obtained from the USACE report of the Delaware River Basin (USACE 2008b).

#### 3.4.1.2 Schuylkill River PMF

The PMF peak flow on the Schuylkill River at LGS is determined in the PMF-Hydrology calculation (Exelon 2014f).

The HEC-RAS hydraulic model in support of the PMF analysis is developed in Calculation LM-0701, Probable Maximum Flood – Hydraulics – Fukushima Flood Hazard Assessment (Exelon 2014g).

### 3.4.2 Methodology

Upstream dam failures may result from a hydrologic event (i.e. PMF), a seismic event, or embankment failure due to piping through the embankment (sunny-day failure). The seismic and sunny-day dam failure modes are bounded by failure during the PMF scenario, as discussed below. Warning time is not relevant because no manual actions are required to respond to a dam failure flood.

The "Peak Outflow with Attenuation Method" and "Representing Clusters of Dams" described in the NRC's Guidance for Assessment of Flooding Hazards Due to Dam Failure (NRC JLD-ISG-2013-01) is used to conservatively calculate the increase in the PMF peak flow at LGS due to upstream dam failures. The adopted methodology involves the use of a single representative, hypothetical dam having the storage volume equal to the sum of the dams upstream of LGS within the Schuylkill River watershed and calculating the attenuation of peak outflow. The height of the tallest, most representative dam is used for the calculations of the dam breach flow. The hypothetical dam is then modeled at the location of the nearest dam upstream from LGS.

The methodology used for the dam failure evaluation is summarized as follows:

1. Identify dams upstream of LGS using the NID database.

2. Determine the total storage and storage-weighted height for the hypothetical dam using the heights and storages of the upstream dams; assuming maximum storage for each dam.
3. Adjust the height upward if necessary based on the most representative tallest dam in the watershed.
4. Calculate the peak dam breach outflow using a regression equation.
5. Calculate the attenuation of peak outflow from dam break to LGS using a regression equation.
6. Use the attenuated constant peak outflow in the steady state HEC-RAS model to determine the maximum water surface elevation due to dam failure during the PMF at LGS.

#### 3.4.2.1 Identifying Upstream Dams

The NID database is used to obtain dam coordinates that were then mapped into the Schuylkill River watershed to identify dams upstream of LGS. Maiden Creek Dam is a proposed USACE dam dating back to the 1960s and is not included in the NID database. Maiden Creek Dam is included in the analysis.

Based on the NID database there are no dams located in the Sanatoga Creek or Possum Hollow Run watersheds.

#### 3.4.2.2 Calculate Storage Volume and Representative Height

The dams identified are combined and modeled as one hypothetical dam located at the location of the closest dam, 7.8 miles upstream from LGS. The storage volume is calculated using the sum of maximum storage for the selected dams. The storage-weighted average height is calculated and then adjusted upward to be representative of the height of the largest single reservoir that best represents the dams within the Schuylkill River watershed.

#### 3.4.2.3 Calculate Peak Dam Breach Outflow

The peak breach outflow for the hypothetical dam is calculated using three different equations; the U.S. Bureau of Reclamation (USBR) peak breach outflow equation (USBR 1982), the NRCS peak breach outflow equation (NRCS 1985), and the Froehlich peak breach outflow equation (Froehlich 1995). The highest resulting peak breach outflow is then selected for use in modeling the resultant water surface elevation.

#### 3.4.2.4 Calculate Attenuated Peak Outflow

The distance between the nearest upstream dam and LGS along the Schuylkill River and tributaries is measured along the river using the measurement tool in ArcGIS. Attenuation of the peak dam breach outflow is calculated using the USBR empirical attenuation formula (USBR 1982).

#### 3.4.2.5 Hydraulic Simulation on the Schuylkill River

The Schuylkill River peak water surface elevation resulting from the combination of upstream dam breach and the PMF at LGS from the PMF-Hydrology calculation (Exelon 2014f) is calculated using the steady state mode HEC-RAS model from the PMF-Hydraulics calculation (Exelon 2014g).

#### 3.4.2.6 On-Site Spray Pond Failure Assessment

The spray pond was constructed by excavation only with no built up embankment. In the unlikely event of failure, the topographic relief of the area surrounding the spray pond is analyzed. The spray pond volume is not added to the volume of the upstream dams. The spray pond is located at LGS and the effects of potential flooding would pass before the upstream dam failures would have any effect.

The spray pond functions as the UHS. The spray pond construction was by excavation to elevation 241 feet NGVD29. The normal operating level of the spray pond is 251 feet NGVD29. The emergency spillway is set at 252 feet NGVD29 and the roadway crest elevation around the spray pond is 255 feet NGVD29. Because the spray pond is an excavation, there is no defined embankment. In the unlikely event of exceeding the capacity of the spray pond, overtopping flows may have the potential to cause localized erosion around the rim of the spray pond. However, because the spray pond is constructed by excavation, the function of the UHS would not be compromised.

### 3.4.3 Results

The attenuated dam breach outflow from upstream dams in the Schuylkill River watershed is 431,000 cfs at LGS. The resultant peak water surface elevation from the combined dam breach peak outflow and PMF in the Schuylkill River at LGS is 192.5 feet NGVD29.

The topographic relief around the spray pond indicates that if failure were to occur, the potential overtopping flows would drain away from the power block are through the north and west sides of the pond towards Sanatoga Creek and the Schuylkill River, respectively.

### 3.4.4 Conclusions

The Schuylkill River CLB dam failure peak water surface elevation at LGS is 201 feet NGVD29. The reevaluated maximum water surface elevation due to the combined effects of dam failure and the Schuylkill River PMF at LGS is equal to 192.5 feet NGVD29. The reevaluated maximum water surface elevation is 8.5 feet below the Schuylkill River CLB dam failure maximum water surface elevation.

## 3.5 Combined Events Flood

The combined events flooding is analyzed in the Calculation LM-0707, Combined Events - Fukushima Flood Hazard Assessment (Exelon 2014k). NUREG/CR-7046, Appendix H states that the following alternative combinations should be evaluated to determine the highest flood water elevation at the site:

H.1 Floods Caused By Precipitation Events

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

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

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

H.2 Floods Caused By Seismic Dam Failures

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

As identified in Section 3.4.2, the dam failure results in Calculation LM-0702 indicate that the maximum water surface elevations caused by seismic dam failures are bounded by the PMF with coincident hydrologic dam failure on the Schuylkill River at LGS. Therefore, combinations listed in H.2 are bounded by H.1 and are not evaluated further. Combinations of events that include surge, seiche, and tsunami (NRC NUREG/CR-7046, H.3, H.4, and H.5 Combinations) are not evaluated because these mechanisms are screened out in Calculation LM-0695.

### 3.5.1 Inputs

Inputs for the analysis in addition to the site location and plant grade elevation of 217 feet NGVD29 are provided below.

#### 3.5.1.1 Peak Hydrologic Dam Failure Water Surface Elevation

The peak hydrologic dam failure with PMF Stillwater elevation of 192.5 feet NGVD29 is from the Dam Failure calculation (Exelon 2014h).

#### 3.5.1.2 2-Year Wind Speed

The fastest 2-minute wind speed from the NOAA NCDC Global Historical Climatology Network-Daily (GHCND) Station GHCND: USW00014712 (NOAA 2013a).

#### 3.5.1.3 Topography

The digital elevation model of Pennsylvania (PAMAP 2005) is used to calculate fetch length based on the dam failure PMF stillwater elevation.

### 3.5.2 Methodology

The analysis of the combined events is performed based on the guidelines outlined in ANSI/ANS-2.8-1992 and NUREG/CR-7046. The combined event evaluation for LGS used the following steps:

1. Calculate the straight line fetch in the critical direction.
2. Calculate the sustained 2-year return period wind speed by using the fastest 2-minute wind speed and applying the Gumbel Distribution to the observed data.
3. Calculate wave height and period using CEDAS-ACES version 4.03 wave prediction application, including duration and temperature difference sensitivity.
4. Determine the wind setup using USACE EM 1110-2-1420 (USACE 1997).
5. Determine the wave setup using CEDAS-ACES wave setup across the surf zone application.
6. Determine the wave runup using the CEDAS-ACES irregular wave runup on smooth slope linear beaches application.
7. Calculate the Probable Maximum Water Elevation on the Schuylkill River at LGS resulting from the combined-effect flood.

CEDAS-ACES uses the equations developed in the Coastal Engineering Manual (CEM) (USACE 2008a) to determine wind generated wave height, setup, and runup.

#### 3.5.2.1 Determine the Straight Line Fetch

Due to the irregular geometry of the inundated areas of the Schuylkill River during the dam failure scenario, two wave runup transects are established to capture a range of conservative fetch lengths at LGS. The over-water fetch lengths are calculated using the probable maximum water elevation from the HEC-RAS model as described in the Dam Failure calculation (Exelon 2014h) and the digital elevation model of Pennsylvania (PAMAP 2005). Fetch lengths are calculated using the measure tool in ArcMAP.

#### 3.5.2.2 Calculate the Sustained Wind Speed

The 2-year annual recurrence interval wind speed is required for the coincident wind wave calculations as part of the combined-effects flood analysis per NUREG/CR-7046 (NRC, NUREG/CR-7046). The fastest 10-meter altitude, 2-minute duration wind speed recorded at nearby NCDC GHCND stations is used. The Reading Spaatz Field, PA gage (GHCND USW00014712) is selected because it has the best available data in the vicinity of LGS. The 2-year return period, 2-minute wind speed is calculated to be 39.7 miles per hour using the Gumbel Distribution.

#### 3.5.2.3 Determine Wave Height and Period

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

The deepwater wave growth formulas are utilized. The simplified wave growth formulas predict deepwater wave growth in accordance to fetch and duration-limited criteria. These formulas are bounded (at the upper limit) by the estimates for a fully developed spectrum and

accredit wave growth importance to the fetch length. The following variables are developed as input to the program to calculate wave height and period:

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

A sensitivity analysis of temperature difference and wind speed duration is performed by independently varying temperature difference and wind speed duration. The combination of temperature difference and wind speed duration that resulted in the highest wave height is utilized in the calculation of the wave height for LGS.

#### 3.5.2.4 Determine Wind Setup

Wind setup is defined as the effect of horizontal stress on water due to wind driving the water in the direction of the wind. Wind setup is calculated in accordance with USACE EM 1110-2-1420 (USACE 1997).

#### 3.5.2.5 Determine Wave Setup

The "Wave Setup across the Surf Zone" application of CEDAS-ACES is utilized to develop wave setup along each transect. Wave setup is calculated using the significant wave height. A refraction coefficient for an incident wave angle of 0 degrees is utilized. The beach slope is calculated using the PAMAP DEM for the vicinity of LGS and a conservative slope estimated within the inundated site area.

#### 3.5.2.6 Determine Wave Runup

The "Runup and overtopping on impermeable slopes" application of the CEDAS-ACES software, based on empirical runup equations developed by Ahrens and Titus (USACE 1992), is used to develop the wave runup at LGS. Wave runup is calculated using the significant wave height. Nearshore slopes are estimated from the PAMAP DEM for the vicinity of LGS.

### 3.5.3 Results

The bounding stillwater elevation for riverine flood events is 192.5 ft NGVD29 at LGS. This flood level results from the hydrologic dam failure in conjunction with the PMF flooding event, as described in the Dam Failure calculation (Exelon 2014h).

The wind setup resulting from a 2-year wind speed applied along the critical direction is calculated to be 0.8 feet.

The wind generated wave setup resulting from a 2-year wind speed applied along the critical direction (per Appendix H of NRC NUREG/CR-7046) on the Schuylkill River at the LGS is calculated to be 2.3 feet.

The wind generated wave runup resulting from a 2-year wind speed applied along the critical direction (per Appendix H of NRC NUREG/CR-7046) on the Schuylkill River at the LGS is calculated to be 6.6 feet. The water elevation resulting from the one-percent wave height is bounded by the wave runup.

The probable maximum water elevation resulting from combined events on the Schuylkill River at LGS is the sum of the bounding flood (H.1 Combination Alternative 1 water surface elevation 192.5 feet NGVD29 due to dam failures during the PMF), the 2-year wind setup (0.8 feet), the 2-year wave setup (2.3 feet), and the 2-year wind wave runup (6.6 feet), which results in a water elevation of 202.2 feet (NGVD29).

### 3.5.4 Conclusions

The maximum combined events flood elevations reported in the UFSAR including significant and 1-percent wave actions are 206 feet NGVD29 and 207 feet NGVD29, respectively (UFSAR). The reevaluated maximum combined events flood elevation is equal to 202.2 feet NGVD29. The reevaluated maximum combined events flood elevation is 4.8 feet below the CLB maximum combined events flood elevation.

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

## 3.6 Ice-Induced Flooding

The ice-induced flooding is analyzed in the Calculation LM-0697, Ice Jams - Fukushima Flood Hazard Assessment (Exelon 2014c).

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

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

An evaluation of upstream and downstream structures and historical ice jam events near LGS is performed to demonstrate that ice jam flooding is bounded by the PMF at LGS, which produces a water surface elevation of 163.9 feet NGVD29 (Exelon 2014g). The assumptions associated with ice-induced flooding are listed in Calculation LM-0697.

### 3.6.1 Inputs

The inputs for the analysis are described below.

#### 3.6.1.1 Historical Ice Jam Data

The USACE ice jam database (USACE 2014).

#### 3.6.1.2 Bridge Geometry

The bridge geometry is extracted from Schuylkill River HEC-RAS model developed as part of the PMF-Hydraulics calculation (Exelon 2014g).

#### 3.6.1.3 Schuylkill River Flow Data

USGS flow and gage height data are used to estimate gage river stage prior to the ice jam event on the Schuylkill River.

#### 3.6.1.4 FEMA 100-year flood elevations

Flooding due to collapse of an upstream ice jam or backwater flooding due to a downstream ice jam is conservatively assumed to occur during a FEMA 100-year flood event. The 100-year flood elevations from the FEMA Flood Insurance Study (FEMA 2001) for Montgomery County, PA are used.

### 3.6.2 Methodology

The method of analysis is summarized below.

1. Identify the largest historic ice-induced flooding event from the ice jam database on the Schuylkill River, Sanatoga Creek, and Possum Hollow Run and calculate the approximate resulting ice jam height.
2. Estimate the peak water surface elevation at LGS resulting from failure of upstream ice jam that hypothetically formed at the first bridge upstream of LGS in each analyzed stream.
3. Estimate the peak water surface elevation at LGS from backwater effects resulting from an ice jam that hypothetically formed at the first bridge downstream of LGS in each analyzed stream.

#### 3.6.2.1 Identify Historical Ice Jams

The USACE Cold Regions Research and Engineering Laboratory ice jam database (USACE 2014) was queried to obtain the record of ice jams that have occurred on the Schuylkill River, Sanatoga Creek, and Possum Hollow Run. The period of record available is from March 1920 through March 2014. The ice jam that resulted in the highest ice thickness in the Schuylkill River is selected for use as the ice jam forming on the upstream and downstream structures.

No records of ice jams on Sanatoga Creek or Possum Hollow Run were found in the USACE ice jam database. There is a potential for ice jam formation on these streams, specifically at the confluence of these streams with the Schuylkill River. It is expected that flooding resulting from potential ice jams formed at the confluence of Sanatoga Creek or Possum Hollow Run with the Schuylkill River will be bounded by flooding resultant from a downstream ice jam on the Schuylkill River.

#### 3.6.2.2 Calculate Ice Jam Height of Historic Ice Jam

The ice jam database typically reports river stage relative to a local gage datum. The reported river stages during the ice jam event are converted to river elevations by adding the reported river stage to the "zero" stream gage elevation (gage datum). The zero gage elevation is determined based on information from the USGS.

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

To calculate the height of the ice jam, it is necessary to estimate what the river elevation would have been without the ice jam. The river elevation is estimated based on the daily flow at the gage the day before the ice jam event to avoid possible ice-induced influences at the

stream gage. Recent available flow-stage relations for the USGS stream gages are used to translate the flow obtained around the ice jam event to river stage. The river stage is then selected as the river elevation the day before the ice jam (i.e., the river elevation without the ice jam). The height of the ice jam is then calculated as follows:

*Ice Jam Height = River Elevation during Ice Jam – River Elevation day before Ice Jam*

3.6.2.3 Calculate Water Surface Elevation from Upstream Ice Jam

The first significant stream obstruction upstream of the site is utilized as the location of a hypothetical upstream ice jam. The first upstream structure of LGS is a railroad bridge. The hypothetical ice jam is considered to breach when the water level behind the ice jam reached the top of the ice jam. The peak water surface elevation at LGS due to an upstream ice jam failure is calculated as the reported FEMA 100-year flood elevation (FEMA 2001) at LGS combined with the ice jam height at the upstream bridge. The ice jam height is conservatively translated directly to LGS without attenuation.

3.6.2.4 Calculate Water Surface Elevation from Downstream Ice Jam

A roadway bridge downstream of LGS is selected for the location of a downstream ice jam on the Schuylkill River. The backwater at LGS caused by the ice jam at the downstream bridge is calculated by adding the height difference between the FEMA 100-year flood elevation at LGS and the FEMA 100-year flood elevation at the downstream bridge to the maximum water surface elevation due to the ice jam at the downstream bridge.

### 3.6.3 Results

The historic ice-induced flood on the Schuylkill River is calculated to be the result from the March 6, 1920 ice jam occurring in Reading, Pennsylvania. A river stage of 18.2 feet was recorded with a calculated ice jam height of 11.8 feet. The flood wave effects from an upstream ice jam and the backwater effects from a downstream ice jam both result in a water surface elevation of 137.8 feet NGVD29.

### 3.6.4 Conclusions

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

The CLB indicates ice-induced flooding is not applicable to LGS. The reevaluated result of ice-induced flooding is 137.8 feet NGVD29 and is bounded by the Schuylkill River PMF elevation at LGS, 163.9 feet NGVD29. The reevaluated results of ice-induced flooding are 26.1 feet below the Schuylkill River PMF elevation.

## 3.7 Channel Migration

The channel migration potential phenomenon is analyzed in the Calculation LM-0696, Channel Migration and Diversion - Fukushima Flood Hazard Assessment (Exelon 2014b).

NUREG/CR-7046 notes that natural channels may migrate or divert either away from or toward the site. There are no well-established predictive models for channel diversion. Historical records and hydrogeomorphological data should be used to determine whether an adjacent channel, stream, or

river has exhibited the tendency to meander towards the site. The assumptions associated with Channel Migration are listed in Calculation LM-0696.

### 3.7.1 Inputs

The inputs for the analysis are described below.

#### 3.7.1.1 Historical Topographic Map

The 1906 and 2013 USGS Topographic maps of LGS are examined to illustrate general continuity of the river shore for that period.

#### 3.7.1.2 Geologic Data

Bedrock depth information is obtained from Environmental Systems Research Institute (ESRI 2013a) maps representing NRCS SSURGO datasets. Bedrock geology is obtained from the Pennsylvania Geological Survey.

### 3.7.2 Methodology

The evaluation approach is summarized in general terms below.

- Review historical records and geologic data to assess whether the Schuylkill River exhibits the tendency to migrate towards the site.
- Evaluate the foundation type at critical structures to assess potential susceptibility to erosion caused by possible channel migration.
- Review historic landslides in the vicinity of the site and review landslide potential for causing the channel to migrate towards the site.
- Evaluate present-day channel stabilization and maintenance measures in place to mitigate channel migration of the Schuylkill River.

### 3.7.3 Results

A review of historical data and site information indicates that the Schuylkill River has not exhibited a tendency to meander towards LGS. All seismic Category I plant structures are founded on bedrock with the exception of part of the spray pond. Channel diversion impacts at LGS are not anticipated to occur as a result of landslide due to the lack of susceptible topographic and geological features. The USACE has an active role in maintaining the current alignment of the Schuylkill River.

### 3.7.4 Conclusions

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

## 3.8 Error/Uncertainty

The Error/Uncertainty is analyzed in Calculation LM-0704 Error/Uncertainty - Fukushima Flood Hazard Assessment (Exelon 2014j). The analysis evaluates the errors and uncertainties associated with the Effects of LIP calculation. The flood due to the LIP event at LGS is the controlling flood hazard mechanism because it results in the highest water surface elevations at the site. However, none of the peak flood elevations calculated for the LIP (or any other flood mechanism) exceed the CLB. The assumptions associated with error/uncertainty analysis are listed in Calculation LM-0704.

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

- Error/uncertainty associated with the selected Manning's roughness coefficients
- Error/uncertainty associated with the topographical survey used to develop the ground surface elevations in the computational model used to evaluate flooding due to the LIP (FLO-2D model).

### 3.8.1 Inputs

#### 3.8.1.1 Two-Dimensional Hydrodynamic Model

FLO-2D model developed as part of the LIP Calculation LM-0699, Local Intense Precipitation (LIP) - Fukushima Flood Hazard Assessment (Exelon 2014e).

#### 3.8.1.2 Range of Manning's n Parameter

Range of Manning's n Parameter for paved areas is 0.02 to 0.05, forest and shrubs is 0.30 to 0.40, grass cover is 0.05 to 0.4, and 0.025 for bodies of water.

#### 3.8.1.3 Topographic Accuracy Information

The ground survey vertical accuracy is reported in the site survey information. The LiDAR vertical accuracy is reported in the LiDAR metadata.

### 3.8.2 Methodology

LIP error/uncertainty analysis is done by performing sensitivity analyses on the FLO-2D model developed as part of the LIP Calculation LM-0699 (Exelon 2014e).

#### 3.8.2.1 Manning's n Parameter Sensitivity Analysis

A sensitivity analysis is performed for the Manning's n parameter used in the FLO-2D model developed as part of the LIP Calculation (Exelon 2014e). The high and low ends of the recommended range of Manning's n for each surface type (e.g., paved area, grassed areas, etc.) used in the FLO-2D model are assessed in the FLO-2D model. The recommended ranges for Manning's n parameter for the surface types included in the FLO-2D model are as follows:

| | |
|---|---|
| "Average grass cover" used for grass areas: | 0.05 – 0.40; |
| "Shrubs and forest litter, pasture" used for forest areas: | 0.30 – 0.40; |
| "Asphalt or concrete" used for paved areas: | 0.02 – 0.05. |

The FLO-2D model included in the LIP Calculation (Exelon 2014e) utilized the low end values of the recommended ranges of Manning's n-values (0.02 for the paved areas, 0.30 for forest and shrub areas, 0.05 for grass areas, and 0.025 for the water areas).

A sensitivity analysis is performed by changing the Manning's n values included in the LIP FLO-2D model to reflect the high end range values of 0.05 for paved areas, 0.40 for forest and shrub areas, 0.40 for grass areas, and 0.025 for water areas.

3.8.2.2 Topographic Survey Accuracy

The elevation grid for the project area is calculated based on two datasets including site survey spot elevations and Pennsylvania LiDAR data for Montgomery County. Ground survey was performed within the site secure area limits and is used to compute the grid elevations in this area. Publicly available LiDAR information is used to compute the grid elevations located outside of the secure area. The ground survey data are reportedly more accurate than the LiDAR data. Therefore, the ground survey information is used within the secure area instead of the LiDAR data. The level of accuracy of the LiDAR data is judged acceptable for the purpose of modeling flow characteristics outside the secure area.

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

### 3.8.3 Results

The sensitivity analyses indicate that the FLO-2D model is relatively insensitive to changes in the Manning's n-values. Based on highly conservative Manning's n-values (0.40 for grass areas and 0.05 for paved areas), the sensitivity analyses resulted in a maximum increase in the computed water surface elevations of 0.3 feet at the critical door locations.

The FLO-2D model grid elevations within the secure area have a minimum level of uncertainty of +/-0.05 feet.

### 3.8.4 Conclusions

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

## 3.9 Associated Effects

The associated effects for the flooding due to the LIP are determined in Calculation LM-0703 Associated Events - Fukushima Flood Hazard Assessment (Exelon 2014i). The assumptions associated with the Associated Effects analysis are listed in Calculation LM-0703.

### 3.9.1 Inputs

The inputs for the analysis are described below.

3.9.1.1 Flood Depth

Flood depths during the local intense precipitation event (Exelon 2014e).

3.9.1.2 Flood Velocity

Flood velocities during the local intense precipitation event (Exelon 2014e).

### 3.9.2 Methodology

The HHA approach described in NUREG/CR-7046 (NRC NUREG/CR-7046) is used for the calculation.

The "Guidance for Performing the Integrated Assessment for External Flooding Guidance" (NRC JLD-ISG-2012-05) defines "flood height and associated effects" as follows:

The maximum stillwater surface elevation plus the following factors:

- Wind waves and run-up effects;
- Hydrodynamic loading, including debris;
- Effects caused by sediment deposition and erosion;
- Concurrent site conditions, including adverse weather conditions;
- Groundwater ingress; and
- Other "pertinent factors" including flood event duration and warning time

The "flood height and associated effects" evaluation is performed for the secure area at LGS for the effects of LIP. Surface water from the Schuylkill River does not adversely affect SSCs at LGD due to the flood margin from the calculated maximum water elevation.

The method of analysis is described in general terms below.

- Calculate hydrostatic loading
- Calculate hydrodynamic (i.e., impact) loading
- Evaluate debris impact loading
- Evaluate ground water ingress
- Evaluate sediment deposition and erosion
- Evaluate concurrent site conditions
- Evaluate wind-wave effect
- Evaluate flood duration
- Evaluate warning time

#### 3.9.2.1 Hydrostatic Loading

Hydrostatic loads are those caused by unbalanced water above or below the ground surface, free or confined. These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts. Hydrostatic pressure is equal in all directions and always acts perpendicular to the surface on which it is applied. Hydrostatic loads can be subdivided into vertical downward loads, lateral loads, or vertical upward loads (uplift or buoyancy). Hydrostatic lateral forces are calculated by multiplying one-half of the maximum hydrostatic pressure by the instantaneous maximum flow depth reported in the LIP calculation to obtain force per linear foot. Hydrostatic loads are associated with the LIP flood depths above the ground surface. Saturated conditions below the ground surface are not expected for the LIP event, as discussed in Section 3.9.3.

#### 3.9.2.2 Hydrodynamic Loading

Water flowing around a building (or structure) imposes loads on the building. Hydrodynamic loads, which are a function of flow velocity and structure geometry, include frontal impact on the upstream fact, drag along the sides, and suction at the downstream side. Hydrodynamic loads are calculated by multiplying the hydrodynamic pressure by the instantaneous maximum flow depth reported in the LIP calculation to obtain force per linear foot.

#### 3.9.2.3 Debris Impact Loading

Debris impact loading is not applicable at the secure area at LGS due to the shallow depths and relatively low velocities in the power block area resulting from the local intense precipitation flooding calculated in the LIP analysis.

#### 3.9.2.4 Ground Water Ingress

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

1. Site information sources, such as available drawings and the Flood Walkdown Report, are utilized to identify SSCs which might be impacted by groundwater.
2. The potential for external flood mechanisms (primarily LIP) to affect groundwater levels is examined.
3. The results of the riverine combined events calculation are used to contrast water elevations with those of safety-related structures to evaluate the potential for groundwater ingress.

#### 3.9.2.5 Sediment Deposition and Erosion

High velocity flood flows may result in scour or erosion. Locations where there is an abrupt change in velocity vectors have the potential for sediment deposition. Vector direction and size are identified to assess the potential for both scour and deposition. Maximum velocities in the secure are taken from the LIP calculation and then compared with typical permissible velocities for selected ground cover materials (Fischenich 2001).

#### 3.9.2.6 Concurrent Site Conditions

Concurrent site conditions include adverse weather such as ice conditions forming on site and high winds. Effects due to ice formation have been previously identified in Section 3.6. Effects from high winds include wind wave generation, which has been previously identified in Section 3.5. Wind-wave effects due to flooding from the LIP are not expected due to the duration of the flooding event, shallow flooding depths, and obstructions to wind fetch in the secure area.

#### 3.9.2.7 Wind-Wave Effects

Wind-wave effects including wave loads and wave runup due to the LIP are not considered due to the short duration of the flood event, shallow flooding depths, and obstructions to wind fetch in the secure area.

#### 3.9.2.8 Flood Duration

Flood duration can vary depending on many factors, including rainfall duration and intensity, watershed area and topography, and riverine characteristics. The LIP maximum water elevation is compared to the licensing basis flood elevation for on-site flooding mechanisms.

#### 3.9.2.9 Warning Time

Warning time is not relevant for all flood-causing mechanisms, including LIP, because no manual actions are credited with providing flood protection or mitigation.

### 3.9.3 Results

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

- The maximum hydrostatic unit load for the critical doors in the secure area at LGS is calculated to be 165 pounds per linear foot due to the LIP flooding event.
- The maximum hydrodynamic (impact) unit load at the secure area due to the LIP is calculated to be 40 pounds per linear foot.
- Debris impact loading is not applicable at the secure area due to the shallow depths and limited velocities at critical door locations. Large floating debris is not expected to be generated by an LIP flood within the secure area.
- Groundwater ingress is not anticipated to adversely affect safety-related equipment at LGS due to the low-permeability of much of the site ground cover and the short duration of the elevated water surface levels caused by the LIP. Elevated river flood conditions do not reach the levels of below grade features that require protection.
- The highest velocity in the secure area at LGS due to the LIP is calculated to be 10 feet per second in the enclosed area of the Refueling Water Tank and Condensate Tank. Erosion is not expected to occur in this area due to the paved nature of the surface. Flow velocities are calculated to be upwards of 17.9 feet per second in a paved area north of the Unit 1 Cooling Tower at LGS. The maximum permissible mean velocity threshold for concrete areas is greater than 18 feet per second.

  Local high velocities up to 8.5 feet per second due to LIP are modeled on the adjacent slopes of the Circ. Water Pumphouse, a non-safety related SSC. The slopes are vegetated, but generally located within areas of rock excavation.

  While there are changes in flow velocity direction in the secure area with flow velocities up to 10 feet per second, deposition is not expected to occur on the paved and concrete surfaces. Generally, the non-erodible nature of the site cover will limit the extent of sediment generation.

  Sediment Deposition and erosion is not anticipated to significantly adversely affect the LGS secure area.
- Concurrent site conditions and wave effects, due to LIP are not applicable to LGS.
- The maximum water elevation at the secure area during the LIP is below the CLB.
- The warning time associated with a local intense precipitation event is negligible.

### 3.9.4 Conclusions

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

# 4. FLOOD PARAMETERS AND COMPARISON WITH CURRENT DESIGN BASIS

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

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 (NRC 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 JLD-ISG-2012-05), including:

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

2. Flood event duration parameters (per Figure 6, below, of NRC JLD-ISG-2012-05)
    a. Warning time (may include information from relevant forecasting methods (e.g., products from local, regional, or national weather forecasting centers) and ascension time of the flood hydrograph to a point (e.g. intermediate water surface elevations) triggering entry into flood procedures and actions by plant personnel);
    b. Period of site preparation (after entry into flood procedures and before flood waters reach site grade);
    c. Period of inundation; and
    d. Period of recession (when flood waters completely recede from site and plant is in safe and stable state that can be maintained).

3. Plant mode(s) of operation during the flood event duration

4. Other relevant plant-specific factors (e.g. waterborne projectiles)

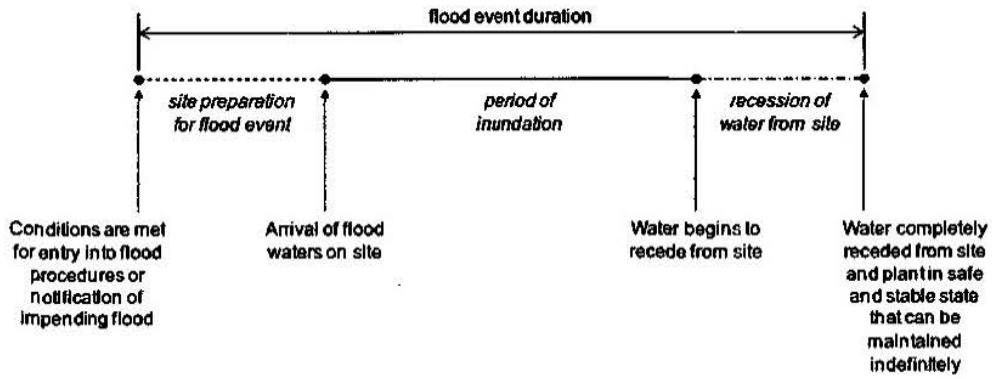


**Figure 4.0.1 - Illustration of Flood Event Duration (NRC JLD-ISG-2012-05, Figure 6)**

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

For LGS, the following flood-causing mechanisms are either determined to be implausible or completely bounded by other mechanisms or the current design basis:

1. Surge, Seiche, and Tsunami;
2. Ice Induced Flooding; and
3. Channel Migration or Diversion
4. Combinations in Section H.1 of NUREG/CR-7046 (Floods Caused by Precipitation Events) for the Schuylkill River (including hydrologic dam failure)
5. Combinations in Section H.1 of NUREG/CR-7046 (Floods Caused by Precipitation Events) for Sanatoga Creek
6. Seismically-Induced Dam Failure (Combination H.2)

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

1. Local Intense Precipitation

2. Combinations in Section H.1 of NUREG/CR-7046 (Floods Caused by Precipitation Events) for Possum Hollow Run

The tables below summarize the parameters for each flood hazard and provide comparisons with the current design basis flood.

**Table 4.0.1 - Summary of Licensing Basis and External Flooding Study Parameters**

| Parameter | Current Licensing Basis Value/Methodology | Reevaluation Study Value/Methodology |
|---|---|---|
| Probable Maximum Precipitation for Schuylkill River | | |
| Methodology | NRC Regulatory Guide 1.59, Appendix B | HMR 51 and HMR 52 |
| Storm Duration | n/a | 72 hours |
| Cumulative PMP (inches) | n/a | 27.2 |
| Probable Maximum Precipitation for Sanatoga Creek | | |
| Methodology | Not Evaluated | HMR 51 and HMR 52 |
| Storm Duration | n/a | 72 hours |
| Cumulative PMP (inches) | n/a | 40.3 |
| Probable Maximum Precipitation for Possum Hollow Run | | |
| Methodology | NOAA Letter 1976 | HMR 51 and HMR 52 |
| Storm Duration | 72 hours | 72 hours |
| Cumulative PMP (inches) | 39.7 | 40.3 |
| Probable Maximum Flood for Schuylkill River | | |
| Nonlinear Basin Response | No | Yes |
| Hydrologic Modeling | NRC Regulatory Guide 1.59, Appendix B | Synthetic UH |
| Total area (sq.mi.) | 1,170 | 1,167 |
| Probable Maximum Flood for Sanatoga Creek | | |
| Nonlinear Basin Response | No | No(1) |
| Hydrologic Modeling | Not Evaluated | Synthetic UH |
| Total area (sq.mi.) | <10 | 7.1 |
| Probable Maximum Flood for Possum Hollow Run | | |
| Nonlinear Basin Response | No | No(1) |
| Hydrologic Modeling | Synthetic UH | Synthetic UH |
| Total area (sq.mi.) | 1.3 | 1.4 |
| Wind Wave Activity coincident with PMF for Schuylkill River | | |
| Wind speed | 40 mph | 39.7 mph |
| Wave parameters determination | Hand Calculation | CEDAS-ACES Computer Software and Hand Calculation |
| Local Intense Precipitation | | |
| Methodology | NOAA Letter 1976 | HMR 51 and HMR 52 |
| LIP Duration (hours) | 72 | 1 and 6 |
| LIP (inches) | 39.7 | 17.9 and 26.9 |
| Effects Local Intense Precipitation | | |
| Methodology | Rational Method and Backwater Calculations | Hydrodynamic Modeling using FLO-2D Computer Software |

| Parameter | Current Licensing Basis Value/Methodology | Reevaluation Study Value/Methodology |
|---|---|---|
| Potential Dam Failure | | |
| Dam Failure Scenarios Considered | Seismic | Hydrologic, Seismic, Sunny-Day |
| Combined Events | | |
| Methodology | Precipitation with Dam Failure and Wind Wave | Precipitation with Dam Failure and Wind Wave |
| Surge, Seiche, and Tsunami | | |
| Methodology | Not Evaluated | Screening Analysis |
| Ice-Induced Flooding | | |
| Methodology | Not Evaluated | Hand Calculated Based on Historical Data |
| Channel Migration | | |
| Methodology | Not Evaluated | Screening Analysis |
| Associated Effects | | |
| Methodology | Not Evaluated | Hydrostatic and Hydrodynamic Loads |

(1) A nonlinear hydrologic response adjustment is not applied because the minimum acceptable lag time is conservatively selected to maximize the runoff potential.

**Table 4.0.2 – Local Intense Precipitation**

| | Flood Scenario Parameter | CDB | Reevaluated | Bounded (B) or Not Bounded (NB) |
|---|---|---|---|---|
| Flood Level and Associated Effects | 1. Max Stillwater Elevation (ft. MSL) | 218.6 | 218.4 | B |
| | 2. Max Wave Run-up Elevation (ft. MSL) | N/A | N/A | N/A |
| | 3. Max Hydrodynamic / Debris Loading (lb/ft) | Not Determined | 40 lb/ft | B |
| | 4. Effects of Sediment Deposition/Erosion | N/A | See Note | B |
| | 5. Concurrent Site Conditions | Not Determined | See Note | N/A |
| | 6. Effects on Groundwater | N/A | See Note | N/A |
| Flood Event Duration | 7. Warning Time (hours) | N/A | N/A | N/A |
| | 8. Period of Site Preparation (hours) | N/A | N/A | N/A |
| | 9. Period of Inundation (hours) | N/A | N/A | N/A |
| | 10. Period of Recession (hours) | N/A | N/A | N/A |
| Other | 11. Plant Mode of Operations | Any | Any | Any |
| | 12. Other Factors | N/A | N/A | N/A |

Notes for corresponding parameter:

1. The reevaluated flood elevation is bounded by the current design basis.
2. Consideration of wind-wave action for the LIP event is not explicitly required by NUREG/CR-7046 and is judged to be a negligible associated effect because of limited fetch lengths and flow depths.
3. The hydrodynamic and hydrostatic loads are determined as force per unit length of structure (lb/ft). To determine the force for the entire structure the loads need to be multiplied by the structure length. The hydrodynamic and hydrostatic loads are bounded by the design basis missile protection loads. The debris load for the LIP event is assumed to be negligible due to the absence of heavy objects at the plant site and due to low flow velocity, the factors combination of which could lead to a hazard due to debris load. Additionally, the water depth around the buildings due to LIP is shallow.
4. The flow velocities due to the LIP event are determined to be below the suggested velocities (Fischenich 2001) for the ground cover type (concrete) at the plant area. Therefore, the erosion is not a plausible hazard for LGS.
5. High winds could be generated concurrent to a LIP event. However, manual actions are not required to protect the plant from LIP flooding so this concurrent condition is not applicable.
6. The majority of the plant area is paved and results in minimal infiltration, if any. Additionally, the event is a short-duration which limits the amount of soil infiltration.
7. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the LIP flood.
8. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the LIP flood.
9. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the LIP flood.
10. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the LIP flood.
11. The reevaluated peak flood elevation is bounded by the current design basis. Current plant operations and procedures will still govern.
12. There are no other factors, including waterborne projectiles, applicable to the LIP flood.

**Table 4.0.3 – Combinations in Section H.1 of NUREG/CR-7046 for Possum Hollow Run Including Dam Failure**

| | Flood Scenario Parameter | CDB | Reevaluated | Bounded (B) or Not Bounded (NB) |
|---|---|---|---|---|
| Flood Level and Associated Effects | 1. Max Stillwater Elevation (ft. MSL) | 159 | 167.8 | NB |
| | 2. Max Wave Run-up (Including Setup) Elevation (ft. MSL) (significant/maximum) | See Note | See Note | B |
| | 3. Max Hydrodynamic and Debris Loading (lb) | 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 | See Note | See Note | B |
| Flood Event Duration | 7. Warning Time (hours) | N/A | N/A | N/A |
| | 8. Period of Site Preparation (hours) | N/A | N/A | N/A |
| | 9. Period of Inundation (hours) | N/A | N/A | N/A |
| | 10. Period of Recession (hours) | N/A | N/A | N/A |
| Other | 11. Plant Mode of Operations | Any | Any | Any |
| | 12. Other Factors | N/A | N/A | N/A |

Notes for corresponding parameter:

1. Reevaluated elevation is bounded by the CLB for the Schuylkill River. The reevaluated flood result is well below the plant finished floor elevation of 217 feet MSL.
2. Reevaluated elevation is bounded by the CLB for the Schuylkill River. The reevaluated flood result is well below the plant finished floor elevation of 217 feet MSL.
3. Reevaluated elevation is bounded by the CLB for the Schuylkill River. The reevaluated flood result is well below the plant finished floor elevation of 217 feet MSL.
4. Reevaluated elevation is bounded by the CLB for the Schuylkill River. The reevaluated flood result is well below the plant finished floor elevation of 217 feet MSL.
5. Reevaluated elevation is bounded by the CLB for the Schuylkill River. The reevaluated flood result is well below the plant finished floor elevation of 217 feet MSL.
6. The stillwater level is bounded by the CLB for the Schuylkill River. Therefore, impact to groundwater ingress is considered to be bounded
7. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the PMF flood.
8. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the PMF flood.
9. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the PMF flood.
10. SSC's important to safety are currently protected by means of permanent/passive measures. Therefore, flood event duration parameters are not applicable to the PMF flood.
11. The reevaluated peak flood elevation is bounded by the current design basis. Current plant operations and procedures will still govern.
12. N/A

# 5. REFERENCES

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

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

(ESRI 2013a) ESRI, Bedrock Depth - Minimum, Modified: October 2013, Accessed: December 2013.

(ESRI 2013b) ESRI, USA Topo Maps, Modified: October 2013, Accessed: February 2014.

(ESRI 2014) ESRI, World Imagery, Modified: July 2014.

(Exelon 2014a) Exelon, Calculation LM-0695, Probable Maximum Surge, Seiche, and Tsunami - Fukushima Flood Hazard Assessment.

(Exelon 2014b) Exelon, Calculation LM-0696, Channel Migration and Diversion - Fukushima Flood Hazard Assessment.

(Exelon 2014c) Exelon, Calculation LM-0697, Ice Jams - Fukushima Flood Hazard Assessment.

(Exelon 2014d) Exelon, Calculation LM-0698, Probable Maximum Precipitation (PMP) - Fukushima Flood Hazard Assessment.

(Exelon 2014e) Exelon, Calculation LM-0699, Local Intense Precipitation (LIP) - Fukushima Flood Hazard Assessment.

(Exelon 2014f) Exelon, Calculation LM-0700, Probable Maximum Flood – Hydrology - Fukushima Flood Hazard Assessment.

(Exelon 2014g) Exelon, Calculation LM-0701, Probable Maximum Flood – Hydraulics - Fukushima Flood Hazard Assessment.

(Exelon 2014h) Exelon, Calculation LM-0702, Dam Failure - Fukushima Flood Hazard Assessment.

(Exelon 2014i) Exelon, Calculation LM-0703, Associated Events - Fukushima Flood Hazard Assessment.

(Exelon 2014j) Exelon, Calculation LM-0704, Error/Uncertainty - Fukushima Flood Hazard Assessment.

(Exelon 2014k) Exelon, Calculation LM-0707, Combined Events - Fukushima Flood Hazard Assessment.

(Exelon 2014l) Exelon, Limerick Topographic Survey.

(FEMA 2001) Federal Emergency Management Agency (FEMA), Flood Insurance Study – Montgomery County, Pennsylvania, Revised October 19, 2001.

(FEMA 2007) Federal Emergency Management Agency (FEMA), Flood Insurance Study – Philadelphia County, Pennsylvania, Revised January 17, 2007.

(FEMA 2012) Federal Emergency Management Agency (FEMA), Flood Insurance Study – Berks County, Pennsylvania, Revised July 3, 2012.

(Fischenich 2001) Stability Thresholds for Stream Restoration Materials, Fischenisch, C., EMRRP Tehncial Notes Collection (ERDC TN-EMRRP-SR-29), U.S. Army Engineer Research and Development Center, Vicksburg, MS, May 2001.

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

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

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

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

(LGS 1983) Railroad Bridge, General Plan and Elevation, Drawing No. C-801 through C-806, Bechtel, November 9, 1983.

(Maidment 1993) Maidment, David R., Handbook of Hydrology, February, 1993.

(NOAA 2013a) National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center (NCDC), Available at: http://www.ncdc.noaa.gov/oa/ncdc.html, Accessed March 2014.

(NOAA 2013b) National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center (NCDC), Global Historical Climatology Network, Daily Summaries, Available at: http://www.ncdc.noaa.gov/cdo-web/, Accessed November 2013.

(NOAA 2013c) National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center (NCDC), Surface Data Hourly Global. Available at: http://www.ncdc.noaa.gov/cdo-web/, Accessed November 2013.

(NOAA 2013d) National Oceanic and Atmospheric Administration (NOAA), National Geophysical Data Center, Tsunami Events Search, Available at http://www.ngdc.noaa.gov/, Accessed December 3, 2013.

(NOAA 2013e) National Oceanic and Atmospheric Administration (NOAA), National Weather Service, Hydrometeorological Design Studies Center, NOAA Atlas 14, Precipitation Frequency Data Server (PFDS), Available at http://hdsc.nws.noaa.gov/hdsc/pfds, Accessed November 2013.

(NRC March 2012) U.S. Nuclear Regulatory Commission, Letter to Licensees, Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding

Recommendations 2.1, 2.3, and 9.3 of the Near Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, March 12, 2012.

(NRC JLD-ISG-2012-05) U.S. Nuclear Regulatory Commission, NRC JLD-ISG-2012-05, Guidance for Performing the Integrated Assessment for External Flooding, Japan Lessons-Learned Project Directorate Interim Staff Guidance, Revision 0, Novemer 2012.

(NRC JLD-ISG-2012-06) U.S. Nuclear Regulatory Commission, NRC JLD-ISG-2012-06, Guidance for Performing a Tsunami, Surge and Seiche Flooding Safety Analysis, Japan Lessons-Learned Project Directorate Interim Staff Guidance, Revision 0, January 2013.

(NRC JLD-ISG-2013-01) United States Nuclear Regulatory Commission, Guidance for Assessment of Flooding Hazards Due to Dam Failure - JLD-ISG-2013-01, Revision 0, July 29, 2013.

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

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

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

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

(NRC RG 1.102) U.S. Nuclear Regulatory Commission, Regulatory Guide 1.102, Rev. 1, Flood Protection for Nuclear Power Plants, Washington, D.C., 1976.

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

(NRCS 2013) Natural Resources Conservation Service (NRCS), Soil Survey Geographic (SSURGO) Database Available at: http://datagateway.nrcs.usda.gov, Accessed November 2013.

(PAMAP 2005) PAMAP Program 3.2 ft Digital Elevation Model of Pennsylvania, Pennsylvania Spatial Data Access Available at http://www.pasda.psu.edu/uci/MetadataDisplay.aspx?entry=PASDA&file=PAMAP_DEM.xml&dataset=1247.

(PAMAP 2008) PAMAP Program Processed LiDAR data, Pennsylvania Spatial Data Access Available at http://www.pasda.psu.edu/uci/MetadataDisplay.aspx?entry=PASDA&file=PAMAP_DEM.xml&dataset=1247.

(UFSAR) Limerick Updated Safety Analysis Report (UFSAR), Revision 16.

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

(USACE 1997) U.S. Army Corps of Engineers (USACE), EM 1110-2-1420, Engineering and Design, Hydrologic Engineering Requirements for Reservoirs, Washington, DC 20314-1000, October 1997.

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

(USACE 2008a) U.S. Army Corps of Engineers (USACE), Coastal Engineering Manual, EM 1110-2-1100 (Part II), Chapter 2, Meteorology and Wave Climate, August 2008 (Change 2).

(USACE 2008b) U.S. Army Corps of Engineers (USACE), Enhancing Multi-jurisdictional Use and Management of Water Resources for the Delaware River Basin, NY, NJ, PA, and DE, December 2008.

(USACE 2014) U.S. Army Corps of Engineers (USACE), Cold Regions Research and Engineering Laboratory Ice Engineering Research Group, available at http://icejams.crrel.usace.army.mil/, Accessed March 2014.

(USACE NID) U.S. Army Corps of Engineers (USACE), National Inventory of Dams (NID), available at http://geo.usace.army.mil/pgis/f?p=397:12, Accessed November 2013.

(USBR 1982) U.S. Bureau of Reclamation (USBR), Guidelines for Defining Inundated Areas Downstream from Bureau of Reclamation Dams, Reclamation Planning Instruction No. 82-11, June 1982.

(USGS 2011) U.S. Geological Survey (USGS), The National Land Cover Database, Available at: http://www.mrlc.gov/nlcd06_data.php, Accessed November 2013.

(USGS 2012) U.S. Geological Survey (USGS), Pennsylvania Earthquake History, Available at: http://earthquake.usgs.gov/earthquakes/sates/pennsylvania/history.php, Accessed December 2013.

(USGS 2013a) U.S. Geological Survey (USGS), Earthquake Hazards Program, Magnitude/Intensity Comparison, Available at: http://earthquake.usgs.gov/learn/topics/mag_vs_int.php, Accessed December 2013.

(USGS 2013b) U.S. Geological Survey (USGS), The Streamstats Program - Pennsylvania, Available at: http://water.usgs.gov/osw/streamstats/pennsylvania.html, Data downloaded on December 2013.

(USGS 2014) U.S. Geological Survey (USGS), Surface-Water Monthly Statistics for the Nation, Available at: http://waterdata.usgs.gov/nwis/monthly/?referred_module=sw, Accessed February 2014.

**Enclosure 2**

**CD-R labeled:**

Limerick Generating Station
Flood Hazard Reevaluation
Pertinent Site Data