FENOC
First Energy Nuclear Operating Company

Perry Nuclear Power Plant
P.O. Box 97
10 Center Road
Perry, Ohio 44081

Frank R. Payne
General Plant Manager, Nuclear

440-280-7300

March 24, 2016
L-16-091

10 CFR 50.54(f)

ATTN: Document Control Desk
U.S. Nuclear Regulatory Commission
11555 Rockville Pike
Rockville, MD 20852

SUBJECT:
Perry Nuclear Power Plant
Docket No. 50-440, License No. NPF-58
Revision of Flood Hazard Reevaluation Report in Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding the Flooding Aspects of Recommendation 2.1 of the Near-Term Task Force (NTTF) Review of Insights from the Fukushima Dai-ichi Accident (TAC No. MF6099)

On March 12, 2012, the Nuclear Regulatory Commission (NRC) issued a letter titled, "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" [Agencywide Documents Access and Management System (ADAMS) Accession No. ML12053A340], to all power reactor licensees and holders of construction permits in active or deferred status. Enclosure 2 of the 10 CFR 50.54(f) letter addresses NTTF Recommendation 2.1 for flooding. One of the required responses is for licensees to submit a Hazard Reevaluation Report (HRR) in accordance with the NRC's prioritization plan. By letter dated May 11, 2012 (ADAMS Accession No. ML12097A509), the NRC placed the Perry Nuclear Power Plant (PNPP) in Category 3 requiring a response by March 12, 2015. The flood HRR for PNPP was submitted by letter dated March 10, 2015 (ADAMS Accession No. ML15069A056).

By letter dated December 11, 2015, FirstEnergy Nuclear Operating Company (FENOC) informed the NRC staff of a planned revision to the PNPP flood HRR to reflect the impact of recent plant and modeling changes, including major physical modifications to the streams and watersheds at the site. FENOC hereby submits Revision 1 of the PNPP flood HRR, which has been revised to reflect these site modifications.

There are no regulatory commitments contained in this letter. If there are any questions or if additional information is required, please contact Mr. Thomas A. Lentz, Manager – Fleet Licensing, at 330-315-6810.

I declare under penalty of perjury that the foregoing is true and correct. Executed on March 24, 2016.

Respectfully,

Frank R. Payne

Enclosure:
Flood Hazard Reevaluation Report, Revision 1

cc: Director, Office of Nuclear Reactor Regulation (NRR)
NRC Region III Administrator
NRC Resident Inspector
NRR Project Manager

Enclosure
L-16-091

Flood Hazard Reevaluation Report, Revision 1
(59 pages follow)

FLOOD HAZARD REEVALUATION REPORT

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

for the

PERRY NUCLEAR POWER PLANT
10 Center Road
North Perry, OH 44081

FirstEnergy

First Energy Corporation
76 South Main Street
Akron, OH 44308

Revision 1

March 2, 2016

	Printed Name	Affiliation	Signature	Date
Preparer:	Kurt Roblyer	ENERCON	Kurt Roblyer	3/2/2016
Verifier:	Suraj Balan	ENERCON		3/2/2016
Verifier:	Freddy Dahmash	ENERCON		3/2/2016
Approver:	Ray Sacramo	ENERCON		3/2/2016
Lead Responsible Engineers:	John Sabo	FENOC		03/04/2016
	George J. Moore III	FENOC		03/04/16
Design Engineering Supervisor	Bossi for Culler	FENOC		3/4/16
Design Engineering Manager	Ben Huck	FENOC		3/4/16
Director of Engineering	Doug Reeves	FENOC		3/4/16

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1.0 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) which included six (6) enclosures:

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

In Enclosure 2 of the NRC-issued information request (Reference 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 FirstEnergy Nuclear Operating Company (FENOC) for the Perry Nuclear Power Station (PNPP), this Flood Hazard Reevaluation Report (Report) provides the information requested in the March 12, 2012 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, or other additional future assessments as necessary, 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, FENOC is requested to perform a reevaluation of all appropriate external flooding sources for PNPP, including the effects from local intense precipitation (LIP) on the site, the probable maximum flood (PMF) on streams and rivers, lake flooding from storm surges, seiches, and tsunamis, and dam failures. It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for ESPs, combined operating license reviews, 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).

The NRC prioritization of responses letter (Reference NRC May 2012) identifies PNPP as a Category 3 site. Licensees in this category are expected to report the results of the reevaluation within three years of the March 12, 2012 50.54(f) letter issuance.

For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan documenting planned actions or measures implemented to address the reevaluated hazards.

Subsequently, addressees shall perform an integrated assessment, or other additional future assessments as necessary, of the plant to fully identify vulnerabilities and detail 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 sink (UHS) that could be adversely affected by flood conditions (the loss of UHS from non-flood associated causes is 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 the NRC-issued information request 50.54(f) letter, the Report should provide documented results, as well as pertinent PNPP information and detailed analysis, and include the following:

1. 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 include the following:

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

2. 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, tsunamis, 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. A basis for inputs and assumptions, methodologies and models used, including input and output files, and other pertinent data should be provided.

3. 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.

4. Interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to the completion of the integrated assessment or any future assessments.

5. Additional actions beyond requested information item 1.d taken or planned to address flooding hazards, if any.

2.0 SITE INFORMATION

PNPP is located in Lake County, Ohio, approximately seven miles northeast of Painesville. The southern plant site boundary line is 3,100 feet from the shoreline of Lake Erie on the west side of the site and 8,000 feet on the east side (USAR, Section 2.4.1.1). Lake Erie is the major hydrologic feature of the location. In the vicinity of the site, the coastal watershed is drained by several small streams (USAR, Section 2.4.1.2). Three nameless streams run close to the plant area. The Major Stream has a drainage basin of 7.44 square miles (Reference PNPP 2015p) and runs northwestward within 1,000 feet of the southwest corner of the plant. The Minor Stream has a drainage area of only 0.11 square miles (Reference PNPP 2015b) and borders the plant area to the east. The Diversion Stream has a drainage area of 0.59 square miles (Reference PNPP 2015b) and is located east of the Minor Stream. The Diversion Stream diverts runoff flow upstream of the Minor Stream east around the outside of the security barriers and north directly to Lake Erie. The safety-related structures of the plant are located within an isolated drainage area and within the drainage basin of the Minor Stream. Final grade elevations in the immediate plant area vary from 617 to 620 feet (USGS) (USAR, Section 2.4.2.2). The floors at plant grade are set at Elevation 620.5 feet (USGS) (USAR, Section 2.4.2.3). Note that the Updated Safety Analysis Report (USAR) presents elevations using a USGS datum that is equivalent to the National Geodetic Vertical Datum of 1929 (NGVD 29). For reference, conversion for the North American Vertical Datum of 1988 (NAVD 88) and for the International Great Lakes Datum of 1985 (IGLD 85) are provided in Table 1 (Reference PNPP 2015b and PNPP 2015h). The present-day site layout is shown in Figure 2.0.1.

Table 1 – Datum Conversions (feet)		
NGVD 29	**NAVD 88**	**IGLD 85**
0.00	-0.72	-0.94

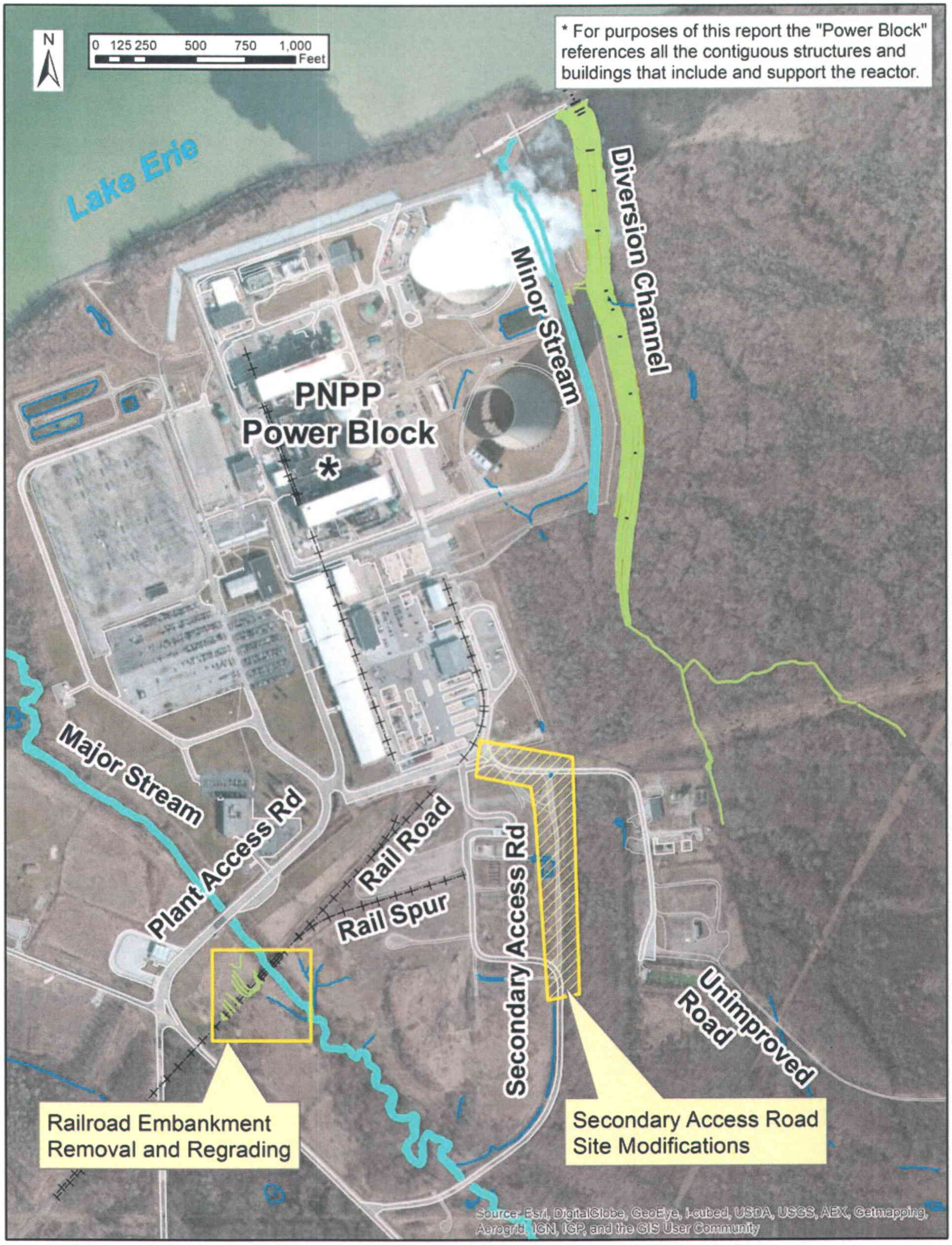

Figure 2.0.1 – Present-Day Site Layout

2.1. Current Design Basis

The current design basis as referenced by the PNPP Updated Safety Analysis Report (USAR) is provided in Section 2.1. The following is a list of flood-causing mechanisms and their associated water surface elevations that were considered for the PNPP current design basis. Section 2.3 includes discussion of the updated design basis analyses.

2.1.1. Local Intense Precipitation (LIP)

The USAR indicates that the precipitation value of 26.7 inches over a 6-hour period, with the maximum hourly rainfall of 13.1 inches occurring during the first hour, is utilized for the LIP analysis. In the case of complete blockage of the storm drainage system, the plant site has been graded so that overland drainage will occur away from the plant site buildings and will not allow the accumulated storm water to exceed Elevation 620.5 feet (USGS) (USAR, Section 2.4.2.3).

2.1.2. Flooding in Streams and Rivers

The USAR identifies a flow rate of 31,250 cubic feet per second (cfs) for the Major Stream and 7,000 cfs for the Minor Stream (USAR, Section 2.4.3). The Major Stream water surface elevation upstream of the plant access road for the PMF was found to be 624.0 feet (USGS) until this surface met the normal depth of flow in the existing stream. This water surface elevation will safely pass beneath the railroad bridge (USAR, Section 2.4.3.5). The Minor Stream water surface elevation was found to be 619.5 feet (USGS) (USAR, Figure 2.4-8).

2.1.3. Dam Breaches and Failures

The USAR identifies no impoundments upstream of the plant. Therefore, dam failure is not included as a design condition (USAR, Section 2.4.4).

2.1.4. Storm Surge and Seiche

The probable maximum meteorological event in Lake Erie results in a maximum stillwater surface elevation of 580.5 feet (USGS) (USAR, Section 2.4.5.2.2). The probable maximum storm (PMS) was assumed to occur over a 72-hour period, during which the winds increased from 20 miles per hour to the maximum speed of approximately 103 miles per hour over the lake and then decreased to less than 35 miles per hour in the Perry area (USAR, Section 2.4.5.1.3).

2.1.5. Tsunami

Since the site is located on Lake Erie, an inland lake, tsunami occurrence is not applicable (USAR, Section 2.4.6).

2.1.6. Ice-Induced Flooding

Ice flooding cannot occur because of the high bluffs between the buildings and the lake. Also, safety-related onshore buildings are set back from the top of the 45-foot high bluff to preclude ice forces being a problem (USAR, Section 2.4.7.4).

2.1.7. Channel Migration or Diversion

Channel diversion is not applicable to PNPP since no cooling water channels exist from which flow could be diverted (USAR, Section 2.4.9).

2.1.8. Combined Effect Flood (including Wind-Generated Waves)

Wind wave activity, including runup, was evaluated as part of the surge analysis. Runup occurring coincidentally with the probable maximum setup would extend to about elevation 607.9 feet (USGS) on the bluff at the lake shore (USAR, Section 2.4.2.2).

2.1.9. Low Water

No water is taken from the Major Stream or Minor Stream. Therefore, low flows in the streams will not affect PNPP operation.

The UHS for PNPP is Lake Erie. Submerged offshore intakes supply water to the emergency service water pumphouse. All safety-related pumps and equipment are located above elevation 586.5 feet (USGS) in the emergency service water pumphouse (USAR, Section 2.4.10). The emergency service water pumphouse and emergency service water pumps are designed to provide service capacity under all lake level conditions down to 565.26 feet (USGS) level caused by the probable maximum setdown superimposed on the minimum monthly mean lake level (USAR, Section 2.4.11.6). Two vertical shafts convey water into the intake tunnel. The intake heads are covered with a velocity cap to prevent/minimize whirlpooling. With the inverts of the intake ports at an average elevation of 552.65 feet (USGS), inflow of sufficient cooling water is assured (USAR, Section 2.4.11.5).The corresponding water level in the emergency service water pump chamber would be at elevation 562.09 feet (USGS).

2.2. Flood-Related Changes to the License Basis

There were no changes to the flood-related license basis since the initial license issuance.

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

An engineering change package (Reference PNPP ECP 13-0802) implemented in 2015 established the updated design basis for the LIP and the flooding in streams and rivers as discussed in the following subsections. The analyses supporting the engineering change package are based on the 2012 aerial survey conducted by Sanborn. The Sanborn survey is referenced to the NAVD 88 datum. Therefore, the analyses were conducted using the NAVD 88 datum with final results converted to a datum consistent with the plant. The basis of the conversion originally used a general conversion from NAVD 88 to NGVD 29. Condition Report CR 2015-05079 (Reference PNPP CR 2015-05079) addressed that the conversion factor is more accurate when based on the local onsite plant monuments. The conversion was adjusted by 0.21 feet and a new conversion factor is used (NAVD 88 + 0.93 ft = Perry Local Datum and IGLD 85 + 1.15 ft = Perry Local Datum). The results presented herein are in the Perry Local Datum. The Perry Local datum was originally based on and within the tolerance levels of the NGVD 29 datum.

The engineering change package included site modifications to the Major Stream and established the Diversion Stream to reroute the south reaches of the Minor Stream as shown in Figure 2.0.1. The engineering change package incorporated removal of a portion of the existing abandoned rail line embankment to the southwest of the rail line bridge crossing the Major Stream. Removal of the embankment allows greater conveyance of flow in the overbanks of the Major Stream, preventing flow from overtopping the rail line. Furthermore, the secondary access road was raised to prevent any remaining backwater from overtopping the road. This modification identified in Figure 2.0.1 resulted in the concentration of Major Stream runoff to be carried downstream to Lake Erie, rather than contributing to runoff to other areas of the site.

The engineering change package incorporated an upstream diversion of the Minor Stream. The Diversion Stream diverts runoff flow east around the outside of the security barriers and north directly to Lake Erie. A berm was constructed adjacent to the Diversion Stream between the stream and the site. Under PMF conditions the berm will maintain all flood water within the Diversion Stream watershed away from the site. These modifications significantly reduce the runoff flow received by the Minor Stream.

Based on aerial images of the watershed, other changes to the watershed include commercial development within the watershed area, which is a small percentage of the overall watershed area. The changes to the local area sub-watershed for PNPP include buildings that have been added or removed and security barrier upgrades that have been added to the site since license issuance.

2.3.1. Probable Maximum Precipitation (Reference PNPP 2015a and PNPP 2016b)

The design basis rainfall is determined using the U.S. Bureau of Reclamation Design of Small Dams, Second Edition guidance (Reference USBR 1973) as referenced by the USAR. The guidance incorporates Hydrometeorological Report No. 33 (Reference NOAA 1956) and includes a recommended area reduction factor for small drainage areas. The Corrected PMP utilizing the area reduction factor is a front loaded temporal distribution event having 10.5 inches of rainfall occurring the first hour and 21.4 inches over the entire 6-hour period. The Uncorrected PMP does not incorporate the area reduction factor and is characterized by 13.1 inches in the first hour and a 6-hour accumulation of 26.7 inches (USAR, Section 2.4.3.1). Table 2 provides a description of the PMP events.

Table 2 – PMP Comparison

Duration (hours)	1	2	3	4	5	6
Uncorrected PMP (inches)	13.1	17.1	20.0	22.4	24.6	26.7
Corrected PMP (inches)	10.5	13.7	16.0	17.9	19.7	21.4

2.3.2. Local Intense Precipitation (LIP) (Reference PNPP 2016a and PNPP 2016b)

The effects of LIP were analyzed using FLO-2D hydraulics software to establish the updated design basis, including the impacts of site stream modifications. The software uses shallow water equations to route stormwater throughout the site. The Major Stream is incorporated into the model as a static water level boundary condition. The Minor Stream watershed is incorporated into the modeling. The Diversion Stream and associated watershed do not affect the area of the power block analyzed for the effects of LIP. The Diversion Stream and associated watershed also do not affect the Major Stream or the Minor Stream. The Diversion Stream is not included in the FLO-2D modeling.

Buildings and security wall barriers are incorporated into the model as obstructions. The site storm drainage network is incorporated into the model using the SWMM software incorporated into the FLO-2D software. The elements of the roof drain system that connect directly to the storm drainage network or groundwater are incorporated into the model. Roof drains that discharge directly to the ground are not modeled. The site also includes an underdrain system that is independent of the storm drainage network. Sixteen underdrain manhole locations are modeled as open inlets. A forthcoming modification is in development for the underdrain system to allow for manhole structures to intake surface water. The modification is required to validate the LIP analysis discussed in this section. The plant is currently operating under PFA 2015-08036 (Reference PNPP CR 2015-08036) due to the potential flooding at the site.

FLO-2D software presents an efficient physical process model that assess the established concepts of unsteady flow analysis. FLO-2D is a volume conservation flood routing model incorporating a grid system to evaluate topography. FLO-2D solves finite difference approximations of the continuity and momentum equations to compute flow in eight directions for each grid cell. The continuity and momentum equations are the underlying equations for unsteady flow analysis. Unsteady flow is referenced for streamcourse modeling by the design basis guidance standard of the American National Standards Institute (ANSI) (Reference ANSI N170-1976).

To determine the associated flooding elevations, the Corrected PMP identified in Table 2 is applied evenly across the site. No rainfall losses are included in the modeling. The effects of LIP produce maximum water surface elevations at the power block ranging from 619.8 feet Perry Local Datum to 620.7 feet Perry Local Datum. In general, the south end of the power block around the Unit 2 Turbine Building results in higher maximum water surface elevations. The resulting maximum water surface elevations exceed 26 door threshold elevations of the power block. The maximum water depths at the 26 door locations range from 0.1 feet to 0.4 feet above the door threshold elevations for durations ranging from 0.3 hours to 6 hours.

Maximum water surface elevations north of the power block at the Service Water Pumphouse Building and the Emergency Service Water Pumphouse Building range from 619.1 feet plant datum to 620.0 feet plant datum. The resulting maximum water surface elevations do not exceed door threshold elevations at the Service Water Pumphouse. The resulting maximum water surface elevations exceed two door thresholds and one louver opening of the Emergency Service Water Pumphouse. The maximum water depths at the three door locations range from 0.3 feet to 0.5 feet above the opening elevations for durations of 6 hours. The doors of the Circulating Water pumphouses at each cooling tower are not affected by the effects of LIP flooding.

2.3.3. Flooding in Streams and Rivers

As a result of the site modifications in 2015, the flooding in streams and rivers were analyzed to establish the updated design basis.

2.3.3.1. Major Stream (Reference PNPP 2015c, PNPP 2015p, and PNPP 2015q)

The Major Stream PMF is the flood resulting from the 72-hour duration site-specific, all-season probable maximum precipitation (PMP). Front, one-third, center, two-thirds, and end-loading temporal distributions are considered in an effort to capture the distribution that maximizes runoff. Cool-season alternatives are bounded by the all-season results.

The U.S. Army Corps of Engineers (USACE) HEC-HMS hydrologic software is used to convert rainfall to runoff. A rainfall hyetograph is applied to the Major Stream watershed and the National Resources Conservation Service (NRCS) unit hydrograph methodology is used for rainfall-to-runoff transformation. The unit hydrograph is modified to account for the effects of nonlinear basin response by increasing the peak of the unit hydrograph by one-fifth and reducing the time-to-peak by one-third. An estimated base flow is included in the analysis and no precipitation losses are incorporated.

The unsteady flow simulation module within the USACE HEC-RAS software is used to transform the resulting flow hydrograph into a water surface elevation hydrograph. The cross sections reflect the removal of a portion of the rail line embankment. The plant access road and culvert is included in the model. The upstream rail line bridge is incorporated into the model. The upstream secondary access road is included as an inline structure and all flow overtops the road. A bounding water level in Lake Erie is used as the downstream boundary condition.

The area between the rail line spur and the secondary access road in the overbank on the northeast side of Major Stream would be subject to the effects of localized PMP rainfall. The area is evaluated using a HEC-HMS level-pool storage routing model. The same site-specific PMP is applied to the model. No losses are incorporated into the analysis. Discharge from the area is determined based on weir flow over the rail line.

The analysis does not result in overtopping of the rail line bridge structure. Furthermore, Major Stream flooding upstream of the rail line is prevented from encroaching on adjacent areas by the elevated embankment of the rail line spur in the overbank on the northeast side of the Major Stream. The resulting maximum water levels for the Major Stream are not adjacent to the power block. The raised secondary access road is also not overtopped by the localized flooding of the area between the rail line spur and the secondary access road. Therefore, the Major Stream does not contribute to flooding effects in the Minor Stream, the Diversion Stream, or the power block area. The one-third, center, and two-thirds temporal distributions produce identical peak results and the maximum water levels. The maximum water surface elevation at the rail line bridge structure is 628.5 feet Perry Local Datum, with a maximum flow of 34,200 cfs at the bridge. The maximum water surface elevation produced for the area between the rail line spur and the secondary access road is 631.4 feet Perry Local Datum.

A bounding approach is taken in the analysis of the flooding effects of Major Stream. The bounding approach determined a more conservative case between design basis and beyond design basis into one analysis for the development and implementation of stream modifications. The bounding PMP applied is the beyond design basis site-specific PMP as defined in Table 3.

Table 3 – Site-Specific PMP

Duration (hours)	**1**	**2**	**3**	**4**	**5**	**6**	**12**	**24**	**48**	**72**
PMP (inches)	12.1	13.6	14.8	15.9	17.0	18.1	24.0	25.5	26.1	26.6

In initial appearances the Uncorrected HMR 33 PMP appears to bound the site-specific PMP as it has the larger hourly and cumulative storm duration PMP totals. However, the site-specific PMP is evaluated using 5-minute increments based on the Hydrometeorological Report No. 52 (Reference NOAA 1982) sub-hourly ratios, which are not distributed linearly as in the design basis methodology utilizing Hydrometeorological Report No. 33. Due to the small drainage areas and short response time, the sub-hourly PMP increments are critical in determining the PMF, by producing more conservative inflow hydrographs and ultimately produces higher flow rates.

As shown in Table 2, the Uncorrected PMP is greater than the Corrected PMP. Therefore, use of the site-specific PMP is a bounding approach for the Major Stream updated design basis input rainfall.

2.3.3.2. Minor Stream (Reference PNPP 2015a and PNPP 2015b)

As a bounding approach, the Minor Stream PMF is the flood resulting from the 48-hour duration Uncorrected Hydrometeorological Report No. 33 PMP. Ratios provided by Hydrometeorological Report No. 52 are used to define sub-hourly increments. Intermediate 5-minute incremental PMP depths are determined for point precipitation (1 square mile). Front, one-third, center, two-thirds, and end-loading temporal distributions are considered in an effort to capture the distribution that maximizes runoff. Cool-season alternatives are bounded by the all-season results.

The USACE HEC-HMS hydrologic software is used to convert rainfall to runoff. The Minor Stream watershed is subdivided into three (3) sub-watersheds (a small remaining area south of the site, the Minor Stream and site area contributing to the Minor Stream at the lateral swale between the two cooling towers, and the small remaining area of the unnamed tributary east of the security barriers) based on the topography of the site and reflecting the construction of the Diversion Stream. A rainfall hyetograph is applied to the sub-watersheds and NRCS unit hydrograph methodology is used for rainfall-to-runoff transformation. The unit hydrographs are modified to account for the effects of nonlinear basin response by increasing the peak of the unit hydrograph by one-fifth and reducing the time-to-peak by one-third. An estimated base flow is included in the analysis and no precipitation losses are incorporated.

The unsteady flow simulation module within the USACE HEC-RAS software is used to transform the resulting flow hydrograph into a water surface elevation hydrograph. One crossing is incorporated into the Minor Stream model. The downstream Lockwood Road crossing is modeled as an inline structure and all flow overtops the road. A bounding water level in Lake Erie is used as a downstream boundary condition for the HEC-RAS software program.

The one-third, center, and two-thirds temporal distributions produce identical peak results and the maximum water levels. The maximum water surface elevation occurs at the cross section corresponding to the south end of the Unit 2 Turbine Building and is 619.7 feet Perry Local Datum. The maximum water surface elevation profile decrease to 618.7 feet Perry Local Datum at the cross section corresponding to the north end of the Unit 1 Turbine Building. The maximum flow overtopping Lockwood Road downstream is approximately 1,100 cfs. Peak flooding does not overflow the Minor Stream channel banks and does not reach the power block.

2.3.3.3. Diversion Stream (Reference PNPP 2015a and PNPP 2015b)

As a bounding approach, the Diversion Stream PMF is the flood resulting from the 48-hour duration Uncorrected Hydrometeorological Report No. 33 PMP. Ratios provided by Hydrometeorological Report No. 52 are used to define sub-hourly increments. Intermediate 5-minute incremental PMP depths are determined for point precipitation (1 square mile). Front, one-third, center, two-thirds, and end-loading temporal distributions are considered in an effort to capture the distribution that maximizes runoff. Cool-season alternatives are bounded by the all-season results.

The USACE HEC-HMS hydrologic software is used to convert rainfall to runoff. A rainfall hyetograph is applied to the Diversion Stream watershed and the NRCS unit hydrograph methodology is used for rainfall-to-runoff transformation. The unit hydrograph is modified to account for the effects of nonlinear basin response by increasing the peak of the unit hydrograph by one-fifth and reducing the time-to-peak by one-third. An estimated base flow is included in the analysis and no precipitation losses are incorporated.

The unsteady flow simulation module within the USACE HEC-RAS software is used to transform the resulting flow hydrograph into a water surface elevation hydrograph. The culvert at the outfall of the Diversion Stream is assumed to be completely blocked and modeled as an inline structure. A bounding water level in Lake Erie is used as a downstream boundary condition for the HEC-RAS software program.

The one-third, center, and two-thirds temporal distributions produce identical peak results and the maximum water levels for the Diversion Stream. The PMF inflow hydrograph has a peak of 4,200 cfs. The maximum water surface elevation occurs at the upstream fill area and is 629.2 feet Perry Local Datum. The fill area and berm are designed with a crest elevation of 630.9 feet Perry Local Datum. The berm constructed adjacent to the stream prevents flooding of the site.

The Diversion Stream berm was assessed for piping failure and the potential for impact on the Minor Stream results. The HEC-RAS models used to evaluate the effects of the PMF are used to assess the failure effects. The HEC-RAS models are evaluated using unsteady-state flow simulation.

The peak of the failure flow hydrograph added to the Minor Stream modeling is 670 cfs. Because of the difference in response times for the Minor Stream and the Diversion Stream, the addition of the failure flow hydrograph extends the duration of flooding in the Minor Stream, but the peak water surface elevations are identical to the PMF flooding for the Minor Stream.

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

As a result of the updated design basis (Reference PNPP 2016b), the LIP maximum water surface elevations exceed the door threshold elevations at several locations of the power block. A summary of the updated design basis results are provided in Section 2.3.2. The door schedule for the updated design basis LIP flood hazard results is provided in Attachment 1. The plant is currently operating under Prompt Functionality Assessment (PFA) 2015-08036 (Reference PNPP CR 2015-08036) due to the potential flooding at the site.

3.0 SUMMARY OF FLOOD HAZARD REEVALUATION

NUREG/CR-7046, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America* (Reference NUREG/CR-7046), by reference to the American Nuclear Society (ANS), states that a single flood-causing event is inadequate as a design basis for power reactors and recommends that combinations should be evaluated to determine the highest flood water elevation at the site. For PNPP, the combination that produces the highest flood water elevation at the site safety-related structures is the effects of LIP, as provided below.

With the exception of the Lake Erie analyses, the beyond design basis analyses are based on the 2012 aerial survey conducted by Sanborn. The Sanborn survey is referenced to the NAVD 88 datum. Therefore, the analyses were conducted using the NAVD 88 datum with final results converted to a datum consistent with the plant. The basis of the conversion originally used a general conversion from NAVD 88 to NGVD 29. Condition Report CR 2015-05079 addresses that the conversion factor is more accurate when based on the local onsite plant monuments. The conversion was adjusted by 0.21 feet and a new conversion factor is used (NAVD 88 + 0.93 feet = Perry Local Datum).

The Lake Erie analyses were conducted using the IGLD 85 datum with final results converted to a datum consistent with the plant. The conversion also incorporated the general conversion from NAVD 88 to NGVD 29. Therefore, a conversion adjustment of 0.21 feet is also applicable (IGLD 85 + 1.15 feet = Perry Local Datum). The results presented herein are in the Perry Local Datum. The Perry Local datum is originally based on and within the tolerance levels of the NGVD 29 datum.

The beyond design basis Calculation 50:66.000 *PNPP Site Modifications Beyond Design Basis Local Intense Precipitation* (Reference PNPP 2016c) defines the maximum water surface elevations resulting from the LIP event at the power block. The maximum water surface elevations at the power block due to the LIP event range from 619.9 feet Perry Local Datum to 621.3 feet Perry Local Datum. The LIP maximum water surface elevations exceed the door threshold elevations at several locations of the power block. The elevations of the site doors are referenced to the Perry Local Datum.

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

- NRC Standard Review Plan, NUREG-0800, revised March 2007 (Reference NUREG-0800)
- NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," dated November 2011 (Reference NUREG/CR-7046)
- NUREG/CR-6966, "Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America", dated March 2009 (Reference NUREG/CR-6966)
- "American National Standard for Determining Design Basis Flooding at Power Reactor Sites", dated July 28, 1992 (Reference ANSI/ANS-2.8-1992)
- NEI Report 12-08, "Overview of External Flooding Reevaluations" (Reference NEI August 2012)
- NRC JLD-ISG-2012-06, "Guidance for Performing a Tsunami, Surge or Seiche Flooding Hazard Assessment", Revision 0, dated January 4, 2013 (Reference JLD-ISG-2012-06)
- NRC JLD-ISG-2013-01, "Guidance for Assessment of Flooding Hazards due to Dam Failure", Revision 0, dated July 29, 2013 (Reference JLD-ISG-2013-01)

The flood hazard reevaluation, including inputs and methodology, are beyond the current PNPP design and license basis. Consequently, the analytical results project beyond the capability of the current design basis. The following provides the flood-causing mechanisms and their associated water surface elevations that are considered in the PNPP flood hazard reevaluation:

3.1. Flooding in Streams and Rivers (Reference PNPP 2015a, PNPP 2015b, PNPP 2015c, PNPP 2015j, PNPP 2015n, PNPP 2015o, PNPP 2015p, and PNPP 2015q)

The updated design basis analyses incorporated the beyond design basis events. Therefore, the beyond design basis analyses are identical to the updated design basis analyses for the Major Stream, the Minor Stream, and the Diversion Stream. The PMF in rivers and streams adjoining the site is determined by applying the PMP to the drainage basin in which the site is located. Because the watersheds analyzed are small, the representative PMP is point precipitation.

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

The PMF is a function of the combined events defined in NUREG/CR-7046 for floods caused by precipitation events. Cool-season alternatives are determined to be bounded by the all-season alternative.

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 All-Season PMP
- Waves induced by 2-year wind speed applied along the critical direction

Alternative 2 – Combination of:

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

3.1.1. Basis of Inputs:

The inputs used in the PMP and PMF analyses are based on the following:

All-Season PMP Analysis

- PNPP, Major Stream, Minor Stream, and Diversion Stream watershed locations;
- Probable Maximum Precipitation Study for the State of Ohio;
- Site-specific, all-season PMP point rainfall short duration estimates determined using a storm-based approach in accordance with National Oceanic and Atmospheric Administration (NOAA) Hydrometeorological Reports and the World Meteorological Organization approach;
- Uncorrected Hydrometeorological Report No. 33 PMP point rainfall estimates;
- Hydrometeorological Report No. 52 ratios for PMP point rainfall estimates for durations less than 1 hour.

PMF Analysis-Hydrologic and Hydraulic Analysis

- PNPP, Major Stream, Minor Stream, and Diversion Stream watershed locations, areas, boundaries and configurations;
- Precipitation for the subject watershed area;
- Base flow: Historic flow rate data collected by USGS at gauge 04212100 on the Grand River, which is used to determine the base flow for the Major Stream, Minor Stream, and Diversion Stream;

- Site topography developed from aerial photogrammetry;
- Digital terrain model (DTM) developed from site topography;
- Manning's roughness coefficients are based on a visual assessment of aerial photography and selected using standard applicable engineering guidance references; and
- Site data for the rail line bridge crossing the Major Stream;
- Construction drawings and as-built drawings for modifications to the Major Stream rail line embankment, the secondary plant access road, and the Diversion Stream.

3.1.2. Computer Software Programs

PMP

- ArcGIS Desktop 10.1
- Microsoft Excel 2007 and 2013

PMF analysis

- AutoCAD Civil 3D 2012
- ArcGIS Desktop 10.0
- ArcGIS Desktop 10.1
- HEC-HMS 3.5
- HEC-RAS 4.1
- HEC-GeoRAS 10.1
- Microsoft Excel 2010 and 2013

3.1.3. Methodology

The site-specific PMP analysis included the following steps:

- An extensive storm search to identify storms which could be used for PMP studies in the region.
- Largest precipitation events which were determined to be transpositionable to the PNPP site were then maximized in-place and transpositioned to the site.

Site-Specific, All-Season PMP

All the storms evaluated in the previous PMP studies in the region, and considered to be transpositionable to the PNPP site were evaluated. This resulted in 20 events that were evaluated to determine the short duration 1-hour depth for the site-specific, all-season PMP. Ten of these storms were previously analyzed in Hydrometeorological Report No. 33 and Hydrometeorological Report No. 51 (Reference NOAA 1978) by the National Weather Service (NWS) and the USACE. The remaining 10 were analyzed for the Probable Maximum Precipitation Study for the State of Ohio.

Each storm is then maximized by an in-place maximization factor to represent what the storm would have looked like had the atmospheric conditions and moisture available for rainfall production been at maximum levels when the storm occurred versus what was actually observed. The in-place maximized values for each storm are then adjusted to transpose the storm from its original location to the PNPP site. The transposition calculation adjusts for differences in available moisture at the site versus the original storm location. The ratios to determine sub-hourly increments in Hydrometeorological Report No. 52, are applied to the resulting site-specific, 1-hour PMP estimate to determine 5-, 15-, and 30-minute duration estimates.

The PMF analysis includes the following steps:

- Delineate watershed and sub-watersheds and calculate sub-watershed areas for input into the USACE HEC-HMS rainfall-runoff hydrologic computer model.
- Determine rainfall.
- Estimate HEC-HMS rainfall-runoff model input parameters: NRCS unit hydrograph method.
- Adjust unit hydrograph to account for the effects of nonlinear basin response.
- Perform PMF simulation with PMP input using HEC-HMS model with no precipitation losses.
- Estimate water surface elevation using HEC-RAS unsteady-state model by using runoff hydrograph from the HEC-HMS model as an input.

Watershed Delineation

For the purposes of the hydrologic modeling effort, the Major Stream is evaluated using one watershed. The Minor Stream watershed is subdivided into three (3) sub-watersheds (a small area south of the site, the Minor Stream and site area contributing to the Minor Stream at the lateral swale between the two cooling towers, and the small remaining area of the unnamed tributary east of the security barriers) based on the topography of the site. The Diversion Stream is evaluated using one watershed.

All-Season PMP

For the Major Stream analysis, the all-season PMP is determined by using the site-specific PMP estimates defined by the PMP study for the State of Ohio (Reference ODNR 2013) for durations from 6 to 72 hours. The PNPP site-specific analysis defines the PMP estimates for 1 hour and sub-hourly increments. Intermediate 5-minute incremental PMP depths are determined for point precipitation (1 square mile).

As a bounding approach, the Minor Stream and Diversion Stream are modeled using the Uncorrected Hydrometeorological Report No. 33 all-season PMP. Hydrometeorological Report No. 33 defines PMP estimates for durations from 1 hour to 48 hours. Ratios provided by Hydrometeorological Report No. 52 are used to define the sub-hourly increments. Intermediate 5-minute incremental PMP depths are determined for point precipitation (1 square mile).

The temporal distribution of the PMP is arranged in accordance with the recommendations in Hydrometeorological Report No. 52, wherein individual rainfall increments decrease progressively to either side of the greatest rainfall increment. Various temporal distributions for each rainfall scenario are then evaluated to further maximize the runoff. Front, one-third, center, two-thirds, and end-loading temporal distributions are considered in an effort to capture the distribution that maximizes runoff.

Note that an antecedent rainfall occurs prior to the all-season PMP. Because of the small drainage areas of the Major Stream, Minor Stream, and Diversion Stream and the dry period between the antecedent storm and the PMP event, the streams return to baseflow conditions prior to the PMP event. Therefore, the antecedent storm is determined to have no influence on the PMP storm.

Hydrologic Model (HEC-HMS)

The PMF is the flood resulting from the all-season PMP. USACE HEC-HMS hydrologic software is used to convert rainfall to runoff. A rainfall hyetograph is applied to the Major Stream watershed, the Diversion Stream watershed, and each sub-watershed of the Minor

Stream and transformed to runoff using unit hydrograph methodology. Generally, a unit hydrograph is developed using historical data obtained from various rain and stream gauges in the watershed. The Major Stream, Minor Stream, and Diversion Stream watersheds are ungauged. Thus, no historical observations are available to use as a basis to create a unit hydrograph. Therefore, a synthetic unit hydrograph is developed. NRCS unit hydrograph methodology is used for rainfall-to-runoff transformation.

ANSI/ANS-2.8-1992 suggests that base flow should be based on mean monthly flow. As mean monthly flow is not available for the Major Stream, Minor Stream, or Diversion Stream, the base flow is approximated based on the mean monthly flow in an adjacent watershed. The USGS gauge station on the Grand River near Painesville, OH is used. The Grand River has the same hydrologic unit code accounting level as the Major Stream, Minor Stream, and Diversion Stream and are adjacent hydrologic unit code sub-watershed levels. Therefore, the rivers and streams exhibit similar watershed characteristics, and it is an acceptable approach to use the base flow information for the Grand River as the basis for estimation of the base flow for Major Stream, Minor Stream, and Diversion Stream.

To conservatively maximize runoff, no precipitation losses are incorporated into the all-season PMF alternative analysis.

The unit hydrographs for the Major Stream watershed, each sub-watershed of the Minor Stream, and the Diversion Stream are modified to account for the effects of nonlinear basin response in accordance with NUREG/CR-7046. The peak of each unit hydrograph is increased by one-fifth and the time-to-peak is reduced by one-third. The remaining hydrograph ordinates are adjusted to preserve the runoff volume to a unit depth over the drainage area.

The area between the rail line spur and the secondary access road in the overbank on the northeast side of Major Stream would be subject to the effects of localized PMP rainfall. The area is evaluated using a HEC-HMS level-pool storage routing model. The same site-specific PMP is applied to the model. No losses are incorporated into the analysis. Discharge from the area is determined based on weir flow over the rail line.

Hydraulic Model (HEC-RAS)

The unsteady flow simulation module within the USACE HEC-RAS software is used to transform the resulting flow hydrographs from the controlling alternative into a water surface elevation hydrograph under unsteady flow conditions.

Channel and floodplain geometry for the Major Stream, the Minor Stream, and the Diversion Stream is modeled by developing cross sections of the streams. The cross sections are placed at locations that define geometric characteristics of the stream and overbanks. Cross sections are also placed at representative locations where changes occur in discharge, slope, shape, and roughness, as well as at hydraulic and inline structures (e.g., bridges culverts). Stream banks, blocked obstructions, and ineffective flow areas are also incorporated into the HEC-RAS model.

The cross sections of the Major Stream reflect the removal of a portion of the rail line embankment. Three crossings are incorporated into the Major Stream model. The plant access road includes a culvert and is modeled using topographic survey data and USAR data. The upstream rail line bridge is modeled using topographic survey data and site records. The upstream secondary access road is included as an inline structure and all flow overtops the road.

One crossing is incorporated into the Minor Stream model. The downstream Lockwood Road crossing is modeled as an inline structure and all flow overtops the road. The culvert at the outfall of the Diversion Stream is assumed to be completely blocked and modeled as an inline structure.

The PMF flow hydrographs obtained from the time-series outflow results of the HEC-HMS models are entered into the HEC-RAS models. A bounding water level in Lake Erie is used as a downstream boundary condition for the HEC-RAS software program.

The Diversion Stream berm was assessed for failure and the potential for impact on the Minor Stream results. Failure of the berm is assumed to occur during the peak water surface elevation in the Diversion Stream, at a location that maximizes the depth. Because the berm crest does not overtop during the PMF, piping failure is considered to be the failure mode. No tailwater is assumed for the analysis. Breach parameters are selected in accordance with JLD-ISG-2013-01 to maximize the breach and the resulting failure flow hydrograph. The HEC-RAS models used to evaluate the effects of the PMF are used to assess the failure effects. The berm failure is modeled as a lateral breach in the HEC-RAS model for the Diversion Stream. The resulting failure flow hydrograph is then added to the HEC-RAS model for the Minor Stream. The HEC-RAS models are evaluated using unsteady-state flow simulation.

3.1.4. Results

Major Stream

The Major Stream is not directly adjacent to PNPP safety-related structures. The analysis does not result in overtopping of the rail line bridge structure. Major Stream flooding upstream of the rail line is prevented from encroaching on adjacent areas in the overbank on the northeast side by the elevated embankment of the rail line spur. The raised secondary access road is also not overtopped by the localized flooding of the area between the rail line spur and the secondary access road. Therefore, the Major Stream does not contribute to flooding effects in the Minor Stream, the Diversion Stream, or the power block area. The one-third, center, and two-thirds temporal distributions produce identical peak results and the maximum water levels. The maximum water surface elevation at the rail line bridge structure is 628.5 feet Perry Local Datum, with a maximum flow of 34,200 cfs at the bridge. The resulting maximum water levels for the Major Stream are not adjacent to the power block. The maximum water surface elevation produced for the area in the overbank on the northeast side of Major Stream between the rail line spur and the secondary access road is 631.4 feet Perry Local Datum. The secondary access road was raised to exceed the maximum water surface elevation as part of the modifications performed in 2015. Therefore, overflow from the area is maintained within the Major Stream watershed. As previously indicated, the updated design basis analyses incorporated the beyond design basis events. Therefore, the beyond design basis results for the Major Stream are identical to the updated design basis.

Minor Stream

The one-third, center, and two-thirds temporal distributions produce identical peak results and the maximum water levels. The maximum water surface elevation occurs at the Unit 2 Turbine Building and is 619.7 feet Perry Local Datum. The maximum flow overtopping Lockwood Road downstream is 1,100 cfs.

The maximum water surface elevations at each safety-related structure adjacent to the Minor Stream flooding are provided in Table 4. As previously indicated, the updated design basis analyses incorporated the beyond design basis events. Therefore, the beyond design basis results for the Minor Stream provided in Table 4 are identical to the updated design basis.

Table 4 – Minor Stream Beyond Design Basis Flooding Elevations and Durations at PNPP

Structure	Maximum Water Surface Elevation (feet Perry Local Datum)	Flood Duration above 620.5 feet (Perry Local Datum)
Unit 2 Turbine Building	619.7	n/a
Unit 2 Auxiliary Building	619.4	n/a
Unit 2 Reactor Building	618.9	n/a
Intermediate Building	618.9	n/a
Unit 1 Reactor Building	618.9	n/a
Unit 1 Auxiliary Building	618.9	n/a
Unit 1 Turbine Building	618.7	n/a

<u>Diversion Stream</u>

The one-third, center, and two-thirds temporal distributions produce identical peak results and the maximum water levels for the Diversion Stream. The PMF inflow hydrograph has a peak of 4,200 cfs. The maximum water surface elevation occurs at the upstream fill area and is 629.2 feet Perry Local Datum. The fill area and berm are designed with a crest elevation of 630.9 feet Perry Local Datum. The berm constructed adjacent to the stream prevents flooding of the site.

Failure of the Diversion Stream berm results in a failure flow hydrograph with a peak of 670 cfs added to the Minor Stream modeling. Because of the difference in response times for the Minor Stream and the Diversion Stream, the addition of the failure flow hydrograph extends the duration of flooding in the Minor Stream, but the peak water surface elevations are identical to the PMF flooding identified in Table 4. As previously indicated, the updated design basis analyses incorporated the beyond design basis events. Therefore, the beyond design basis results for the Diversion Stream are identical to the updated design basis.

3.2. Dam Assessment (Reference PNPP 2015d)

3.2.1. Basis of Inputs

Inputs used for the dam assessment evaluation include:

- PNPP, Major Stream, and Minor Stream watershed locations.
- USGS topographic quadrangle maps.
- The USACE National Inventory of Dams (NID) database is used to identify any watershed dams.

3.2.2. Computer Software Programs

- None

3.2.3. Methodology

The PNPP Major Stream, Minor Stream, and Diversion Stream watershed locations are approximated using USGS topographic quadrangle maps. The USACE NID database is reviewed to determine that no dams are located in the Major Stream, Minor Stream, or Diversion Stream watersheds.

3.2.4. Results

No dams are located in the combined watershed of the Major Stream, Minor Stream, and Diversion Stream. Therefore, dam failure is not applicable.

3.3. Ice-Induced Flooding (Reference PNPP 2015e)

As identified by NUREG/CR-7046, ice jams and ice dams can form in rivers and streams adjacent to a site, and may lead to flooding by two mechanisms:

- Collapse of an ice jam or an ice dam upstream of the site can result in a dam breach-like flood wave that may propagate to the site; and
- An ice jam or an ice dam downstream of a site may impound water upstream of itself, thus causing a flood via backwater effects.

3.3.1. Basis of Inputs

- USACE ice jam database.
- Site topography.
- Bridge geometry (upstream and downstream of PNPP) determined using site topography and site records.

3.3.2. Computer Software Programs

- Microsoft Excel 2013

3.3.3. Methodology

Per NUREG/CR-7046, ice-induced flooding is assessed by reviewing the USACE ice jam database to determine the most severe historical events that have occurred. No historical records are available for the Major Stream, Minor Stream, or Diversion Stream. The nonexistence of ice jam records is explained by the absence of stream monitoring stations on the two streams. Based on ice jam occurrences recorded for rivers within adjacent watersheds, it is determined that ice jam events are possible.

The maximum ice jam is determined by selecting the historic event that produced the maximum flood stage relative to the normal water surface elevation at that location. Regardless of the specific conditions that produced the historic flood stage at a specific location, the full height is conservatively assumed to represent the ice jam.

Historical ice jam data for the adjacent watersheds for the Grand River and the Ashtabula River are considered. Although no records are available for the actual height of the ice jams, the maximum recorded stage is used to represent the ice jam. The ice jam is transposed to the site and compared with other flooding causing mechanisms.

3.3.4. Results

The maximum reported ice jam stage in the regional vicinity of PNPP is estimated to be 18 feet. For the Minor Stream, no significant upstream crossings or structures are present that could facilitate significant ice jam formation. The only upstream access road is now located in the Diversion Stream watershed. The access road is an unimproved road with a low profile and a small drainage structure. The elevation characteristics of the area at that drainage structure indicate in the unlikely event an ice jam did occur, it would be limited to less than 2 feet in height. Furthermore, any ice jam and subsequent failure would only release a flow rate at the maximum capacity of the small drainage structure. By qualitative comparison, the PMF analysis includes rainfall runoff contribution from the entire drainage area resulting in a peak flow rate magnitudes greater than the capacity of a small drainage structure. Any upstream ice jam collapse is bounded by the PMF analysis.

The only downstream location on the Minor Stream conducive to an ice jam is at Lockwood Road. The assumed transposition of the maximum ice jam would produce a maximum water surface elevation of 608.9 feet Perry Local Datum. Assuming the culvert at Lockwood Road is completely blocked, any coincident flow would eventually overtop Lockwood Road. The PMF analysis for Minor Stream assumes the Lockwood Road culvert is completely blocked and the PMF overtops the road. Therefore, by qualitative comparison with the PMF analysis, any downstream ice jam flooding is bounded by the PMF analysis.

Downstream ice jam flooding was not specifically examined for the Diversion Stream. The Diversion Stream is located east of the Minor Stream and is not adjacent to the immediate area of the plant. Similar to the Minor Stream, the downstream end of the Diversion Stream also has a culvert. Assuming the culvert is completely blocked, any coincident flow would eventually overtop the culvert embankment. The culvert embankment is significantly lower than the berm located between the Diversion Stream and the site.

For the Major Stream, the upstream rail line bridge has a clear height of greater than 20 feet from natural grade to the bottom of the low chord. The assumed transposition of the maximum estimated 18 feet ice jam would allow normal flows overtopping the ice jam to flow through the remaining bridge opening below the low chord. In addition, flow would pass through the removed portion of the abandoned rail line embankment to the southwest of the rail line bridge crossing the Major Stream. Any ice jam failure at this location would be transposed downstream to the plant access road.

The access road includes a large elliptical culvert 35'-11" by 23'-5". Based on site topography the clear height is approximately 18 feet above the normal water surface elevation. Assumed transposition of the maximum estimated 18 feet ice jam would produce a maximum water surface elevation below the plant access road. Assuming the culvert at the plant access road is completely blocked, any coincident flow would eventually overtop the plant access road. The PMF analysis for Major Stream results in overtopping of the plant access road. Therefore, by qualitative comparison with the PMF analysis, any ice jam flooding is bounded by the PMF analysis.

The downstream sediment control structure is the only other location possibly conducive to an ice jam. The assumed transposition of the maximum ice jam would produce a maximum water surface elevation of 604.9 feet Perry Local Datum. The results of the PMF analyses for the Major Stream exceed the estimated ice jam elevation.

Ice-induced flooding at PNPP is bounded by the PMF analyses, and no further consideration is required.

3.4. Channel Migration or Diversion (Reference PNPP 2015e)

NUREG/CR-7046 indicates historical records and hydrogeomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to migrate towards the site.

3.4.1. Basis of Inputs

- Ohio Department of Natural Resources surficial geology map (ODNR 2006)
- USGS topographic quadrangle maps, including aerial images (Reference USGS 1943, USGS 1992, USGS 1994, and USGS 2010)

3.4.2. Computer Software Programs

- None

3.4.3. Methodology

Surficial geology along with historic and current topographic quadrangle maps that include aerial images are reviewed to examine the condition and alignment of rivers and streams over time.

3.4.4. Results

The surficial geology map indicates the area in the immediate vicinity of PNPP is characterized by layers of sand, silt and clay, and till or clayey to silty till over shale bedrock. This characterization represents the entire watershed of the Minor Stream and the majority of the Major Stream. The upstream portion of Major Stream is characterized by additional areas of sand and gravel or clayey to silty till over shale bedrock. Alluvium or organic material, which are more susceptible to erosion, are not present in the Major Stream, the Minor Stream, or Diversion Stream watersheds.

Topographic maps for the years of 1905 (Reprinted 1943), 1960 (Revised 1992), 1994, and 2010 are reviewed to assess historic channel migration. The Minor Stream was rerouted as part of the plant construction. In 2015 site modifications constructed the Diversion Stream to divert runoff flow upstream of the Minor Stream, east around the outside of the security barriers, and north directly to Lake Erie. The topographic and aerial images provide no evidence of oxbows, braided streams, or alluvial fans that could indicate a potential for channel migration of the Major Stream or the Minor Stream. The Diversion Stream is newly constructed and does not exhibit evidence of a potential for channel migration. The Diversion Stream is located east of the Minor Stream and is not adjacent to the immediate area of the plant.

The streams in the vicinity of PNPP do not exhibit characteristics of channel migration.

3.5. Storm Surge (Reference PNPP 2015f, PNPP 2015g, PNPP 2015h, PNPP 2015k, and PNPP 2015l)

Probable Maximum Storm Surge (PMSS)

In accordance with JLD-ISG-2012-06, all coastal nuclear power plant sites and nuclear power plant sites adjacent to cooling ponds or reservoirs subject to potential hurricanes, windstorms and squall lines must consider the potential for inundation from storm surge and waves. JLD-ISG-2012-06 also suggests that for the storm surge hazard assessment, historical storm events in the region should be augmented by a synthetic storm parameterized to account for conditions more severe than those in the historical records and considered reasonably possible on the basis of technical reasoning.

3.5.1. Basis of Inputs

The inputs used in PMSS analysis are based on the following:

- Historical wind and pressure field data from NOAA for the Great Lake Region
- Probable maximum windstorm (PMWS)
- Lake Erie bathymetry from the NOAA geophysical database
- Supporting GIS data

3.5.2. Computer Software Programs

- ArcMap 10.1
- Delft3D software suite (Delft3D-FLOW Revision 1, Delft3D-WAVE Revision 0, Delft3D-RGFGRID Revision 1, and Delft3D-QUICKIN Revision 1)
- IGLD85 Height Conversion Tool
- R Computer Language 2.15.1
- Microsoft Excel 2007, 2010, and 2013

3.5.3. Methodology

Several physical processes contribute to the generation of a storm surge. The contribution of wind to a storm surge is often called wind setup. Wind blowing over the water causes a shear stress that is exerted on the surface of the water, pushing water in the direction of the wind. Atmospheric pressure gradients are another forcing mechanism that contributes to changes in water level, as water is forced from regions of high atmospheric pressure toward regions of low atmospheric pressure.

The following describes the methodologies used in the PMSS calculation:

Development of the PMWS

The PMWS storm-based approach is specific to the characteristics of the site. Past extreme events in a region are analyzed and considered transpositionable. As part of the PMWS, different storm types (such as synoptic, squall line, and hybrid) that impacted the Great Lakes region are considered in order to determine the storm event that will generate the maximum surge and seiche. Each storm's input parameters are quantified and plotted based on the location of low/high pressure centers, concurrent wind/pressure fields, and how they evolve through time and space.

Most of the synoptic storms occur in association with deep areas of low pressure which move through the region from southwest to northeast. The general synoptic pattern is one in which the deep area of low pressure results in a very strong pressure gradient force between its low pressure center and a corresponding region of higher pressure to the north or west. The larger the gradient between the two systems over a given distance is, the stronger the resulting winds.

Squall line (or derecho) events create a widespread straight-line windstorm that is associated with a fast-moving band of severe thunderstorms. These winds have produced some of the highest instantaneous gusts on record, but last for only a short time (less than 30 minutes) at a given location. The short duration of these events, as they quickly traverse a given location, means they will not control the PMSS. Further, these events do not occur within deep low pressure systems or remnant tropical systems. Therefore, their wind and pressure data are not combined with the other storm types in this analysis, as this would result in a PMWS that is not physically possible.

Although deep low pressure systems often produce the longest duration large-scale winds, other storm types also produce strong winds over the region. In rare cases, land-falling tropical systems along the Gulf Coast or Atlantic Seaboard move inland across the Appalachians or up the Mississippi and Ohio River valleys. By the time these storms reach the Great Lakes region, they are no longer tropical systems, but instead have transitioned into extra-tropical cyclones. Their general circulation and center of deep low pressure persists. Much like the deep low pressure scenario previously discussed, strong and persistent winds can result. The remnants of Hurricane Hazel (October 1954) and Hurricane Sandy (October 2012) are classic examples of this storm type. This storm scenario provided some of the strongest winds from the northwest through the northeast directions over Lake Erie (with durations of 12 hours or more).

Delft3D Calibration

The Delft3D hydrodynamic model is set up using the Delft3D software suite. The wave setup contribution to the total storm surge values are modeled by coupling the Delft3D-WAVE and Delft3D-FLOW surge models. The general approach to storm surge modeling using coupled Delft3D-FLOW and Delft3D-WAVE models consists of the following steps:

- Developing the bathymetric dataset and model grid mesh for the lake system;
- Assembling input files for atmospheric forcing (wind and pressure fields);
- Assembling input files for initial water level, boundary conditions, and the physical and numerical parameters of the model;
- Assembling measured water levels and wave data for model calibration and verification;
- Testing and refining the initial model setup;
- Validating the model for historical extreme storm events; and
- Assessing model sensitivity to various factors and adjustable parameters such as bottom friction and wind drag coefficient.

The Delft3D model is calibrated based on historical data obtained from NOAA meteorological and water level recording stations located in the Lake Erie region.

Review of historical data shows that various parts of Lake Erie respond differently to any one particular storm. The storm that produces extreme water levels in one part of Lake Erie might not, and probably does not, produce extreme levels in other parts. Therefore the number of calibration and validation storms selected, to assess model prediction accuracy, covered all parts of the lake shoreline.

Calibration and verification of the coupled Delft3D-FLOW and Delft3D-WAVE models is performed by a time series comparison of measured and predicted/modeled storm surge values at different water level recording stations on Lake Erie. A similar time series comparison is also performed for wave heights.

The Delft3D models are calibrated using extreme historic wind and pressure data from multiple meteorological and water level recording stations. Calibration and verification of the coupled Delft3D-FLOW and Delft3D-WAVE models demonstrates that the hydrodynamic model is capable of computing the storm surge and seiche dynamics for Lake Erie, as well as the significant wave heights and periods at PNPP from PMWS events.

PMSS

The calibrated Delft3D model is used to determine the PMSS. The historic wind and pressure field data is replaced with candidate PMWS events, and the model is run to determine the critical PMWS.

JLD-ISG-2012-06 and ANSI/ANS 2.8-1992 require the antecedent (pre-storm) water level equal to the 100-year maximum recorded water level to be applied as the initial storm surge model still water level. The 100-year water level of 575.8 feet Perry Local Datum (574.6 feet IGLD 85) is used as the initial condition/antecedent water level in all the Delft3D-FLOW models. Since the probable minimum low water level at PNPP could occur at a time when the monthly mean lake level is at the long-term mean low probable level, the antecedent water level for low water evaluation is set to the long-term low probable level at Lake Erie, which is equal to 569.2 feet Perry Local Datum (568 feet IGLD 85).

Various topographic features affect the storm surge propagation towards PNPP. The site is located on the bluffs adjacent to the shores of Lake Erie. The bluffs at the site are more than 40 feet above the 100-year water level of the lake.

Maximum Historical and 25-year Storm Surge

The historical maximum storm surge is the largest of the determined yearly maximum storm surge heights. The historical maximum storm surge height is used in combined flooding scenarios as discussed in Section 3.7.

Storm surges are calculated from monthly data as the difference between monthly maximum and monthly mean based on guidance provided by USACE. The Log Pearson Type III distribution is the commonly accepted frequency procedure for annual maximum water levels. A frequency analysis on the yearly maximum storm surge heights obtained from the Fairport and Erie stations is performed using a Log Pearson III statistical analysis. The 25-year storm surge height is used in combined flooding scenarios.

3.5.4. Results

Simulations of all the candidate PMWS events showed that the critical PMWS event is the transpositioned September 1989 wind storm event maximized over the PNPP site. This storm is the most intense of all the PMWS events with a maximum wind speed of 106 miles/hour and is aligned along the north-south direction. The maximum PMSS resulting from this PMWS event produced a maximum water surface elevation of 582.8 feet Perry Local Datum, which is well below the site. Wave runup effects are evaluated with combined flooding scenarios as discussed in Section 3.7.

Surge, Seiche, and Resonance

Results show that the level of the rise due to a seiche is significantly less than the calculated surge height. For this reason, seiches are not the controlling flood event at PNPP.

Resonance generated by waves can cause problems in enclosed water bodies such as harbors and bays when the period of oscillation of the water body is equal to the period of the incoming waves. However, the PNPP site is not located in an enclosed embayment. Additionally, the period of oscillation of Lake Erie near the PNPP site is determined to be in the range of 11 to 15 hours. This is much greater than that of the peak spectral period of the incident shallow water storm waves. Consequently, resonance is not a detriment at PNPP during the critical PMWS event.

Probable Minimum Low Water Level resulting from the PMWS

Simulations of all the candidate PMWS storm events show that the critical PMWS event that would result in probable minimum low water level (drawdown) is the transpositioned November 1998 storm event maximized over the PNPP site. This storm had the most intense southeasterly winds of the examined storm events, with a maximum wind speed of 93 miles/hour. The probable minimum low water elevation (drawdown) associated with the transposed November 1998 storm produces a probable minimum low water level of 563.2 feet Perry Local Datum near the PNPP site. The inverts of the PNPP intake ports are at an average elevation of 552.65 feet Perry Local Datum (USAR 2.4.11.2.2.2).

3.6. Tsunami Assessment (Reference PNPP 2015i)

NUREG/CR-6966 identifies that earthquakes, landslides, and volcanoes can initiate tsunamis, with earthquakes being the most frequent cause. Dip-slip earthquakes (due to vertical movement) are more efficient at generating tsunamis than strike-slip earthquakes (due to horizontal movement). To generate a major tsunami, a substantial amount of slip and a large rupture area is required. Consequently, only large earthquakes with magnitudes greater than 6.5 generate observable tsunamis.

3.6.1. Basis of Inputs

- National Center for Earthquake Engineering Research (NCEER) database
- Natural Resources Canada seismicity data
- NOAA natural hazards tsunami database
- NOAA natural hazards volcano database
- Ohio Department of Natural Resources (ODNR) seismicity data
- USGS earthquake hazards program database

3.6.2. Computer Software Programs

- None

3.6.3. Methodology

As identified by NUREG/CR-7046, tsunami assessment is referenced to NUREG/CR-6966 and NOAA Technical Memorandum OAR PMEL-136 (Reference NOAA 2007). In addition, the more recently issued NRC guidance, JLD-ISG-2012-06, also addresses tsunami assessment. However, a screening assessment is applied and JLD-ISG-2012-06 provides guidance on detailed tsunami modeling that is beyond the scope of a screening assessment. Technical Memorandum OAR PMEL-136 reflects a similar tsunami screening assessment described by NUREG/CR-6966.

The NUREG/CR-6966 screening assessment is based on a regional screening and a site screening. The regional screening consists of researching historical records for tsunami records and the potential for tsunami-generating sources. The site screening evaluates the site based on the horizontal distance from a coast, the longitudinal distance measured along a river, and the grade elevation in comparison to the effects of a tsunami. This assessment approach is based on a review of historical records and databases.

NUREG/CR-6966 identifies that tsunamis are generated by rapid, large-scale disturbances of a body of water. The most frequent cause of tsunamis is an earthquake; however, landslides and volcanoes can also initiate tsunamis. Because of the tsunami-generation sequence associated with earthquakes, dip-slip earthquakes (due to vertical movement) are more efficient at generating tsunamis than strike-slip earthquakes (due to horizontal movement). Furthermore, to generate a major tsunami, a substantial amount of slip and a large rupture area is required. Consequently, only large earthquakes with magnitudes greater than 6.5 generate observable tsunamis.

As part of the assessment, the NOAA natural hazards tsunami database is used to review historical tsunami events and associated runups for the east coast of the United States and Canada. Of the total events, there were 7 tsunami events that produced 14 runups occurring in the Great Lakes region from 1755 to 1954. The USGS hazard fault database findings are reviewed for strong earthquakes or the vertical displacements necessary to induce a tsunami. Additionally, the USGS earthquake hazards program, the NCEER database, and the Natural Resources Canada database are reviewed for historical earthquakes in the region.

ODNR data is also reviewed for earthquake-generated tsunami and landslide-induced tsunami. Lastly, the NOAA natural hazards volcano database is reviewed to assess volcanoes in the Lake Erie region.

3.6.4. Results

According to ODNR, an earthquake-generated tsunami in Lake Erie would require a very large earthquake on the order of magnitude 7.0 or greater and significant vertical displacement. Historically, in the Lake Erie region, the largest earthquakes are in the magnitude 5.0 range. Preliminary analysis of post-glacial sediments in the region has not yielded evidence of a large earthquake in the last few thousand years. Furthermore, earthquakes in the region, for which sufficient data are available, show primarily horizontal rather than vertical movement, which is not as conducive to tsunami generation.

Tsunamis can also be generated by the downslope movement of a very large volume of rock or sediment, either from a rockfall above the water or from a submarine landslide. Although large amounts of unconsolidated sediments are washed into Lake Erie each year when shoreline bluffs are undercut by wave action, these masses lack sufficient volume and rapid collapse to displace a volume of water that would create a tsunami. Lake Erie also has a very gentle bottom profile, particularly in the western and central basins. The eastern basin has steeper slopes, but not steep enough for a large amount of sediment to suddenly flow downslope in a submarine landslide.

The NOAA natural hazards tsunami database identifies only two occurrences of non-seiche (or non-wind-induced) tsunami events in the Great Lakes region. The two occurrences yielded slight or small wave effects.

The USGS Earthquake Hazards Program fault database contains no known Quaternary faults (or current faults) in this region because geologists have not found any faults at the Earth's surface. Consequently, no potential exists for strong earthquakes or the vertical displacement necessary to induce a tsunami. Various earthquake databases, including the USGS Earthquake Hazards Program earthquake database, the National Center for Earthquake Engineering research catalog, and Natural Resources Canada, identify that the largest events in the vicinity are no greater than magnitude 5.0.

Lastly, no volcanoes are located in the Lake Erie region, according to the NOAA natural hazards volcano database.

Tsunami is not the controlling flood event at PNPP.

3.7. Combined Effect Flood (including Wind-Generated Waves) (Reference PNPP 2015m)

Evaluation of floods caused by precipitation events is covered in Appendix H.1 of NUREG/CR-7046. The three alternatives are addressed in flooding on streams and rivers (Section 3.1), which identifies the resulting water surface elevations. Combined effect flooding evaluates the component of added waves induced by 2-year wind speed along the critical direction.

Evaluation of floods along the shores of enclosed bodies of water is covered in Appendix H.4.1 of NUREG/CR-7046 and includes one alternative:

Combination of:

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

Three alternatives are specified in Appendix H.4.2 of NUREG/CR-7046 for streamside locations of enclosed bodies of water. Each of the alternatives considered has three components contributing to the water surface elevation.

- **Alternative 1 – Combination of:**
- The lesser of one-half of the PMF or the 500-year flood;
- Surge and seiche from the worst regional hurricane or windstorm with wind-wave activity; and
- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

- **Alternative 2 – Combination of:**
- PMF in the stream;
- A 25-year surge and seiche with wind-wave activity; and
- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

- **Alternative 3 – Combination of:**
- A 25-year flood in the stream;
- Probable maximum surge and seiche with wind-wave activity; and
- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

3.7.1. Basis of Inputs

Inputs include the following:

- PMSS from Calculation 50:47.000
- Wave parameters for Lake Erie at PNPP from Calculation 50:47.000
- 100-year or maximum controlled water level in Lake Erie from Calculation 50:46.000
- Lake Erie historical maximum surge from Calculation 50:46.000
- Lake Erie 25-year surge from Calculation 50:46.000
- 2-year wind speed
- Site topography

3.7.2. Computer Software Programs

- ArcGIS Desktop 10.1
- HEC-GeoRAS 10.1
- HEC-RAS 4.1
- Microsoft Excel

3.7.3. Methodology

Each combination includes coincident wind-wave activity. Coincident wind-wave activity is determined for the critical flooding combination using the USACE guidance outlined in USACE *Coastal Engineering Manual* (Reference USACE 2008). Wind setup is the effect of horizontal stress on the water surface. Runup is the maximum elevation of wave uprush above stillwater level.

3.7.3.1. H.4.1 Combination

Probable maximum surge and seiche is discussed in Section 3.5. Wind wave activity includes wave height, wind setup, and wave runup. Wave height and wind setup are included as part of the PMSS developed using Delft3D model. Wave runup is determined in accordance with USACE guidance on the shoreline and bluff slopes adjacent to Lake Erie.

3.7.3.2. H.4.2 Combination Alternatives

As a bounding approach, the maximum Lake Erie water level from all three alternatives, including the initial water level and surge effects, is used as the downstream boundary condition for Major Stream and Minor Stream flooding, and combined with the PMF for each stream. It is determined that the maximum Lake Erie water level has no significant effect on the PMF analyses. Based on the analyses and results for the Major Stream and Minor Stream, the Diversion Stream was not specifically analyzed for combined effect flooding. The Diversion Stream is not adjacent to the power block. Therefore, wind wave activity has no specific consequence.

3.7.4. Results

The H.4.1 Combination maximum water surface elevation is 582.8 feet Perry Local Datum, including wind setup, and occurs west of the power block along the shoreline bluff slopes. The maximum effects due to wind wave activity occur at a location just east of the power block along a section of shoreline with steeper bluff slopes. Wave action analysis concludes that a maximum wave runup of 27.5 feet may be generated on the shoreline bluff slopes. The PMSS maximum water surface elevation at this location is 581.9 feet Perry Local Datum. The maximum wave runup elevation during the controlling PMSS is equal to 609.5 feet Perry Local Datum, which is well below the site.

The maximum combined water surface elevation for the Major Stream and the Minor Stream are the PMF results.

The Major Stream and Diversion Stream are not directly adjacent to PNPP safety-related structures. Therefore, wind wave activity has no specific consequence. The Minor Stream PMF flooding is maintained within the stream banks, eliminating the potential for wind wave effects at the power block.

3.8. Local Intense Precipitation (Reference PNPP 2015c, PNPP 2016a, and PNPP 2016c)

The LIP is an extreme precipitation event at a given location. The LIP evaluation is performed in accordance with NUREG/CR-7046. Cool-season alternatives are determined to be bounded by the all-season alternative.

3.8.1. Basis of Inputs

- Site topography
- Site-specific, all-season PMP point rainfall estimates
- Site storm drainage network and roof drain drawings

- Site underdrain system drawings
- Manning's roughness coefficients
- Major Stream maximum water surface elevation results

3.8.2. Computer Software Programs

- ArcGIS Desktop 10.1
- FLO-2D Pro, Build 14-08-09 (incorporating SWMM 5.0.022)
- HEC-RAS 4.1
- MATLAB 8.6.0.267246
- Microsoft Excel 2013

3.8.3. Methodology

The effects of LIP are analyzed using FLO-2D software. The software uses shallow water equations to route stormwater throughout the site. FLO-2D depicts site topography using a digital elevation model (DEM) capturing all terrain features, such as grading, slopes, drainage divides, and low areas of the site.

The Major Stream is incorporated into the model as a static water level boundary condition. The grid model DEM is developed from ground surface information combined with the maximum water surface elevations determined for the Major Stream as a boundary condition. The entire drainage area of the Minor Stream is incorporated into the model domain. The Minor Stream eight foot diameter culvert discharging to Lake Erie is included in the modeling. The culvert feature is modeled using the SWMM software incorporated into the FLO-2D software. HEC-RAS software is used to derive a stage-discharge relationship assigned to the culvert. The Diversion Stream and associated watershed do not affect the area of the power block analyzed for the effects of LIP. The Diversion Stream and associated watershed also do not affect the Major Stream or the Minor Stream. The Diversion Stream is not included in the FLO-2D modeling.

Manning's roughness coefficient values are selected based on land cover type using the guidance provided in the FLO-2D manual. Two types of obstructions are modeled: buildings/structures that completely block the water passage, and security wall barriers that could be overtopped if the water depth increases above the top of the wall. The levee function within FLO-2D is used to incorporate site security features, which could impact the natural drainage characteristics of the site.

The site storm drainage network is incorporated into the model using the SWMM software incorporated into the FLO-2D software. All inlets are modeled with a 50 percent area reduction and 25 percent perimeter reduction to account for inlet grating and potential debris accumulation. The pipe network was also modeled using a 10 percent reduction of pipe capacity. In addition, security restrictions affecting pipe capacity are also incorporated.

The elements of the roof drain system that connect directly to the storm drainage network or groundwater are incorporated into the model. Roof drains that discharge directly to the ground are not modeled. The roof drain system is incorporated into the modeling without reductions. Roof elevations are modeled as described by site drawings. Roof parapets are modeled using the levee function. Roof scuppers are incorporated into the parapet levees.

The site also includes an underdrain system that is independent of the storm drainage network. Sixteen underdrain manhole locations are modeled as open inlets. Inlet reductions are incorporated for the physical inlet grating. However, due to the large three feet to four feet diameter underdrain pipe sizes, no reduction of pipe capacity is included.

The site-specific rainfall is applied as a 6-hour storm event based on 5-minute increments to capture the highest intensity of the distribution. Front, one-third, center, two-thirds, and end-loading temporal distributions are considered in the evaluation. No rainfall losses are included in the modeling.

3.8.4. Results

The site-specific beyond design basis rainfall with an end loading distribution produces the critical results for the effects of LIP flooding. The maximum water surface elevations at the power block range from 619.9 feet Perry Local Datum to 621.3 feet Perry Local Datum. In general, the south end of the power block around the Unit 2 Turbine Building results in higher maximum water surface elevations. The resulting maximum water surface elevations exceed 57 door threshold elevations of the power block. The maximum water depths at the 57 door locations range from 0.1 feet to 1.0 feet above the door threshold elevations for durations ranging from 0.1 hours to 6 hours.

Maximum water surface elevations north of the power block at the Service Water Pumphouse Building and the Emergency Service Water Pumphouse Building range from 619.3 feet Perry Local Datum to 620.5 feet Perry Local Datum. The resulting maximum water surface elevations exceed one door threshold elevation at the Service Water Pumphouse. The maximum water depth is 0.1 feet above the door threshold elevation for a duration of 0.1 hours. The resulting maximum water surface elevations exceed the two door thresholds and one louver opening of the Emergency Service Water Pumphouse. The maximum water depths range from 0.7 feet to 1.0 feet above the opening elevations for durations ranging from 5.8 hours to 6 hours.

The resulting maximum water surface elevations exceed two door threshold elevations at the Unit 2 Circulating Water Pumphouse. The maximum water depths are 0.5 feet above the door threshold elevations for durations of 0.7 hours. The Unit 2 Circulating Water Pumphouse is an abandoned unit that does not communicate, in terms of flooding, with SSCs required for safe plant operation or shutdown. Therefore, flooding of this building is inconsequential to plant safety.

4.0 COMPARISON OF BEYOND DESIGN BASIS WITH UPDATED DESIGN BASIS

The reevaluated maximum water surface elevations due to the LIP, storm surge (PMSS), and combined effect flood for storm surge including wind-generated waves exceed the design basis.

For LIP flooding, the design basis and beyond design basis evaluations are identical except for the rainfall input. The design basis rainfall input is the Corrected PMP front loaded temporal distribution. The beyond design basis rainfall input is the site-specific PMP and is evaluated for the front, one-third, center, two-thirds, and end-loading temporal distributions.

The design basis LIP analysis results exceed 26 door threshold elevations of the power block and the two door thresholds and one louver opening of the Emergency Service Water Pumphouse. Door threshold elevations at the Service Water Pumphouse Building and the Circulating Water Pumphouses are not exceeded. The beyond design basis LIP results exceed 57 door threshold elevations of the power block, one door of the Service Water Pumphouse Building, the two door thresholds and one louver opening of the Emergency Service Water Pumphouse, and two doors of the Unit 2 Circulating Water Pumphouse. The design basis maximum water surface elevations at the power block range from 619.8 feet Perry Local Datum to 620.7 feet Perry Local Datum. The beyond design basis maximum water surface elevations at the power block exceed the design basis range by 0.1 feet to 0.6 feet.

For the lake storm surge flooding, the design basis assumes the surge acts only in one direction. A site-specific wind and pressure field is developed as part of the reevaluation. More recent storms provide the controlling wind for reevaluated surge flooding at PNPP.

As previously indicated, the PMSS maximum stillwater surface elevation exceeds the design basis elevation of 580.5 feet Perry Local Datum by a maximum value of 2.3 feet. The UHS for PNPP is Lake Erie. Although the design basis is exceeded, the PMSS Stillwater surface elevation remains below the safety-related pumps and equipment located above elevation 586.5 feet Perry Local Datum in the emergency service water pumphouse. Additionally, the PMSS minimum stillwater surface elevation is lower than the design basis of 565.26 feet Perry Local Datum by 2.0 feet. However, previous evaluation determined that there is adequate net positive suction head and submergence of the emergency service water pumps for a resulting elevation as low as 559.0 feet Perry Local Datum (Reference PNPP 2004).

For the combined effect flood from storm surge including wind-generated waves, the design basis incorporated the guidance outlined by the USACE *Shore Protection, Planning and Design*, Technical Report No. 4 (Reference USACE 1966). The beyond design basis coincident wind-wave activity is determined using the current USACE guidance outlined in USACE *Coastal Engineering Manual*.

The maximum surge runup exceeds the design basis elevation of 607.9 feet Perry Local Datum by a maximum value of 1.6 feet. However, the maximum effects of the PMSS event are maintained on the bluffs adjacent to Lake Erie. The site grade remains higher than the maximum effects of the PMSS event. The wave runup would cause the lake level to increase as well as decrease through the cycling of the waves. The long intake tunnel of the plant dampens the impact of the high and low wave action. The water levels within the pumphouse tend to equalize to the sustained average height of the lake. Therefore, the resulting beyond design basis PMSS effects are inconsequential to safety-related structures.

The comparisons of existing and reevaluated flood hazards are provided in Table 5.

Table 5 – Comparison of Existing and Reevaluated Flood Hazards at PNPP

Flood-Causing Mechanism	Design Basis	Comparison	Flood Hazard Reevaluation Results
Flooding in streams and rivers	Major Stream[(a)] (Reference PNPP 2015p) PMF Elevation is 628.5 feet Perry Local Datum (at rail line bridge). PMF Flow is 34,200 cfs. Cool-season bounded by all-season. Minor Stream[(a)] (Reference PNPP 2015b) PMF Elevation is 619.7 feet Perry Local Datum. PMF Flow is 1,100 cfs. Cool-season bounded by all-season. Diversion Stream[(a)] (Reference PNPP 2015b) PMF Elevation is 629.2 feet Perry Local Datum. PMF Flow is 4,200 cfs. Cool-season bounded by all-season.	Bounded	Major Stream (Reference PNPP 2015p) All-Season PMF Elevation is 628.5 feet Perry Local Datum (at rail line bridge). All-Season PMF Flow is 34,200 cfs. Cool-Season bounded by all-season. Minor Stream (Reference PNPP 2015b) All-Season PMF Elevation is 619.7 feet Perry Local Datum. All-Season PMF Flow is 1,100 cfs. Cool-Season bounded by all-season. Diversion Stream (Reference PNPP 2015b) PMF Elevation is 629.2 feet Perry Local Datum. PMF Flow is 4,200 cfs. Cool-season bounded by all-season.

Table 5 – Comparison of Existing and Reevaluated Flood Hazards at PNPP (Continued)

Flood-Causing Mechanism	Design Basis	Comparison	Flood Hazard Reevaluation Results
Dam breaches and failures	No upstream impoundments.	Bounded	Dam assessment indicates no watershed dams.
Storm surge	Water surface elevation is 580.5 feet Perry Local Datum (USAR, Section 2.4.5.2.2). Low water elevation is 565.26 feet Perry Local Datum (USAR, Section 2.4.11.6).	Not bounded. Exceeds current design basis.	Water surface elevation is 582.8 feet Perry Local Datum (Reference PNPP 2015h). Low water elevation is 563.2 feet Perry Local Datum (Reference PNPP 2015h).
Seiche	This flood-causing mechanism is not described specifically for the site in the USAR (USAR, Section 2.4.6).	Bounded	Bounded by storm surge (Reference PNPP 2015g).
Tsunami	This flood-causing mechanism is identified as not applicable in the USAR (USAR, Section 2.4.6).	Bounded	Tsunami assessment indicates a slight possibility of tsunamis in the Great Lakes region. However, the seismicity in the region suggests no potential exists for strong earthquakes or the vertical displacement necessary to induce a substantial tsunami (Reference PNPP 2015i).
Ice-induced flooding	This flood-causing mechanism is identified as not plausible in the USAR (USAR, Section 2.4.7).	Bounded	Ice-induced flooding is bounded by the all-season PMF event (Reference PNPP 2015e).
Channel migration or diversion	This flood-causing mechanism is identified as not applicable in the USAR (USAR, Section 2.4.9).	Bounded	Channel diversion is not characteristic for adjacent streams (Reference PNPP 2015e).

Table 5 – Comparison of Existing and Reevaluated Flood Hazards at PNPP (Continued)

Flood-Causing Mechanism	Design Basis	Comparison	Flood Hazard Reevaluation Results
Combined effect flood (including wind-generated waves)	Wave runup on Lake Erie shoreline bluffs is 607.9 Perry Local Datum (USAR, Section 2.4.2.2). Wind wave effects on streams are not applicable.	Not bounded. Exceeds current design basis.	Wave runup on Lake Erie shoreline bluffs is 609.5 feet Perry Local Datum (Reference PNPP 2015h). Wind wave effects on streams are not applicable.
LIP	Maximum water surface elevations at the power block range from 619.8 feet Perry Local Datum to 620.7 feet Perry Local Datum.[(a)(b)] (Reference PNPP 2016b)	Not bounded. Exceeds current design basis.	Maximum water surface elevations at the power block range from 619.9 feet Perry Local Datum to 621.3 feet Perry Local Datum.[(c)] (Reference PNPP 2016c)

(a) Values are based on the reconstituted design basis values discussed in Section 2.3.
(b) The door schedule for the design basis LIP flood hazard results are provided in Attachment 1.
(c) The door schedule for the beyond design basis LIP flood hazard results are provided in Attachment 2.

5.0 INTERIM AND PLANNED FUTURE ACTIONS

The Flooding Hazard Reevaluation Report evaluated the applicable flooding hazards for PNPP. Three of the postulated reevaluated flood hazard events (lake flooding, combined effects, and LIP) resulted in maximum flood water elevations higher than previously calculated for PNPP. These postulated flooding events are considered beyond design basis events and do not constitute an operability concern. The PMSS event and the combined effects for wave runup on the Lake Erie shoreline results are inconsequential. Therefore, no interim or future actions are planned for this event.

The interim action plan is to continue with actions associated with Condition Report CR 2015-08036 until all design basis modifications based on Calculation 50:64.000 LIP results are complete. Design basis modifications will be implemented for the storm sewer system, the underdrain system, and door specific protection or analysis. Implementation of the modifications for the storm sewer system and the underdrain system, as incorporated into the design basis LIP Calculation 50:64.000, are required for the beyond design basis LIP Calculation 50:66.000 to be valid. Door protection will be developed for the beyond design basis Calculation 50:66.000 LIP results through either analysis or physical barriers. Analysis will determine the volume of inflow through the doors for the duration of flooding. The inflow volume will be compared to internal flooding analyses to determine if external flooding is bounded or not. Physical barriers will be determined as either permanent barriers or through procedure.

6.0 ACRONYMS

ANS	American Nuclear Society
ANSI	American National Standards Institute
ArcMap	Arc Map Software
ArcGIS	Arc Geographic Information System Software
AutoCAD	Automated Computer Aided Design Software
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
CR	Condition Report
Delft3D	Delft 3-Dimensional Software
Delft3D-FLOW	Flow Module of Deft3D Software
Delft3D-WAVE	Wave Module of Deft3D Software
Delft3D-RGFGRID	Grid Generation Module of Delft3D Software
Delft3D-QUICKIN	Modification/generation Module of Delft3D Software
DEM	Digital Elevation Model
DTM	Digital Terrain Model
ECP	Engineering Change Package
ESP	Early Site Permits
FENOC	FirstEnergy Nuclear Operating Company
FLO-2D	FLO 2-Dimensional Software
GIS	Geographic Information System
HEC-GeoRAS	Hydrologic Engineering Center Geographic River Analysis System software
HEC-HMS	Hydrologic Engineering Center Hydraulic Modeling System software
HEC-RAS	Hydrologic Engineering Center River Analysis System software
IGLD 85	International Great Lakes Datum of 1985
JLD-ISG	Japan Lessons-Learned Project Directorate Interim Staff Guidance
LIP	Local Intense Precipitation
MATLAB	Matrix Laboratory Software
NAVD 88	North American Vertical Datum of 1988
NCEER	National Center for Earthquake Engineering Research
NGVD 29	National Geodetic Vertical Datum of 1929
NID	National Inventory of Dams
NOAA	National Oceanic and Atmospheric Administration
NRC	Nuclear Regulatory Commission

NRCS	National Resources Conservation Service
NTTF	Near Term Task Force
NUREG	US Nuclear Regulatory Commission Regulation
NUREG/CR	US Nuclear Regulatory Commission Regulation Contractors Report
NWS	National Weather Service
OAR PMEL	Oceanic and Atmospheric Research Pacific Marine Environmental Laboratory
ODNR	Ohio Department of Natural Resources
PFA	Prompt Functionality Assessment
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PMS	Probable Maximum Storm
PMSS	Probable Maximum Storm Surge
PMWS	Probable Maximum Windstorm
PNPP	Perry Nuclear Power Station
RG	Regulatory Guide
SSCs	Structure, Systems, and Components
SWMM	Storm Water Management Model software
UHS	Ultimate Heat Sink
USACE	U.S. Army Corps of Engineers
USAR	Updated Safety Analysis Report
USBR	U.S. Bureau of Reclamation
USGS	U.S. Geological Survey

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ATTACHMENT 1

Updated Design Basis LIP Flood Hazard Results
Front Temporal Distribution
(Reference PNPP 2016b)

Updated Design Basis LIP Flood Hazard Results at Door Locations for Front LIP Temporal Distribution

POI ID Number	Building	Door Number	Grid Number	Door Elev.	Max Flow Depth Above Ground Elevation	Max WSE		Margin Δ = Door Elev - Max WSE
				ft PNPP Datum	ft	ft NAVD88	ft PNPP Datum	(ft)
1	U1 Auxiliary Building	1AX 406	74326	620.504	0.13	619.29	620.22	0.3
2	U1 Auxiliary Building	1AX 404	74642	620.465	0.03	619.39	620.32	0.1
3	U1 Auxiliary Building	1AX 403	75274	620.456	0.58	619.29	620.22	0.2
4	U1 Auxiliary Building	1AX 407	68496	620.614	0.41	619.48	620.41	0.2
5	U1 Auxiliary Building	1AX 405	68499	620.506	0.09	619.53	620.46	0.0
6	U1 Turbine Building	1TB 307	59754	620.500	0.10	619.53	620.46	0.0
7	U1 Turbine Building	1TB 308	58385	620.500	0.03	619.21	620.14	0.4
8	U1 Turbine Building	1TB 306	61482	624.500	0.27	618.87	619.80	4.7
9	U1 Turbine Building	1TB 316	71777	620.392	0.03	619.35	620.28	0.1
10	U1 Heater Bay	1HB 306	64199	620.410	0.03	619.16	620.09	0.3
11	U1 Heater Bay	1HB 310	68856	620.385	0.03	619.29	620.22	0.2
12	U1 Heater Bay	1HB 314	67868	620.870	0.05	619.74	620.67	0.2
13	U1 Heater Bay	1HB 311	67870	620.416	0.05	619.36	620.29	0.1
14	U1 Heater Bay	1HB 309	70808	620.610	0.03	619.46	620.39	0.2
15	U1 Heater Bay	1HB 308	71778	620.261	0.12	619.25	620.18	0.1
16	U1 Turbine Power Complex	1TP 306	73051	620.420	0.20	619.34	620.27	0.2
17	U1 Turbine Power Complex	1TP 303	67841	620.516	0.03	619.53	620.46	0.1
18	U1 Off Gas Building	1OG 303	63836	620.467	0.11	619.43	620.36	0.1
19	U1 Off Gas Building	1OG 302	63499	620.431	0.08	619.43	620.36	0.1
20	U1 Off Gas Building	1OG 304	61123	620.467	0.05	619.54	620.47	0.0
21	U1 Water Treatment Building	0WT 101	55241	620.401	0.21	619.26	620.19	0.2
22	Water Treatment Building	0WT 103	55936	620.399	0.30	619.43	620.36	0.0
23	Water Treatment Building	0WT 102	58372	620.429	0.48	619.43	620.36	0.1
24	Diesel Generator Building	0DG 106	64479	620.512	0.18	619.63	620.56	0.0

Updated Design Basis LIP Flood Hazard Results at Door Locations for Front LIP Temporal Distribution

POI ID Number	Building	Door Number	Grid Number	Door Elev. ft PNPP Datum	Max Flow Depth Above Ground Elevation ft	Max WSE ft NAVD88	Max WSE ft PNPP Datum	Margin Δ = Door Elev - Max WSE (ft)
25	Diesel Generator Building	0DG 105	64146	620.547	0.05	619.62	620.55	0.0
26	Diesel Generator Building	0DG 104	63813	620.680	0.03	619.78	620.71	0.0
27	Diesel Generator Building	0DG 103	63478	620.650	0.03	619.75	620.68	0.0
28	Diesel Generator Building	0DG 102	63143	620.503	0.03	619.59	620.52	0.0
29	Diesel Generator Building	0DG 101	62808	620.487	0.03	619.52	620.45	0.0
30	Radwaste Building	0RW 422	62470	620.459	0.11	619.52	620.45	0.0
31	Radwaste Building	0RW 407	62132	620.393	0.14	619.50	620.43	0.0
32	Radwaste Building	0RW 411	61457	620.435	0.10	619.44	620.37	0.1
33	Radwaste Building	0RW 101	62819	620.473	0.40	619.43	620.36	0.1
34	Intermediate Building	0IB 315	76859	620.594	0.59	619.31	620.24	0.3
35	Intermediate Building	0IB 316	78439	620.570	0.36	619.49	620.42	0.1
36	Intermediate Building	0IB 317	78120	620.416	0.33	619.53	620.46	0.0
37	Service Building	0SB 113	71398	620.430	0.54	619.58	620.51	-0.1
38	Service Building	0SB 114	71721	620.434	0.40	619.57	620.50	-0.1
39	Service Building	0SB 107	70749	620.466	0.58	619.59	620.52	-0.1
40	Service Building	0SB 104	70098	620.436	0.55	619.60	620.53	-0.1
41	Service Building	0SB 105	68131	620.493	0.57	619.64	620.57	-0.1
42	Service Building	0SB 108	66808	620.477	0.52	619.66	620.59	-0.1
43	Service Building	0SB 103	66476	620.482	0.51	619.68	620.61	-0.1
44	Service Building	0SB 102	64810	620.461	0.51	619.66	620.59	-0.1
45	Service Building	0SB 101	64477	620.467	0.29	619.62	620.55	-0.1
46	U2 Auxiliary Building	2AX 403	78748	620.419	0.41	619.68	620.61	-0.2
47	U2 Auxiliary Building	2AX 404	78745	620.415	0.28	619.71	620.64	-0.2
48	U2 Auxiliary Building	2AX 406	79062	620.439	0.31	619.73	620.66	-0.2

Updated Design Basis LIP Flood Hazard Results at Door Locations for Front LIP Temporal Distribution

POI ID Number	Building	Door Number	Grid Number	Door Elev. ft PNPP Datum	Max Flow Depth Above Ground Elevation ft	Max WSE ft NAVD88	Max WSE ft PNPP Datum	Margin Δ = Door Elev - Max WSE (ft)
49	U2 Auxiliary Building	2AX 405	73000	620.546	0.30	619.55	620.48	0.1
50	U2 Auxiliary Building	2AX 407	72041	620.469	0.47	619.57	620.50	0.0
51	U2 Turbine Building	2TB-E	77146	620.499	0.34	619.77	620.70	-0.2
52	U2 Turbine Building	2TB-D	77780	620.426	0.43	619.77	620.70	-0.3
53	U2 Turbine Building	2TB 316	80003	620.440	0.63	619.80	620.73	-0.3
54	U2 Turbine Building	2TB-C	80320	620.311	0.70	619.80	620.73	-0.4
55	U2 Turbine Building	2TB 306	70722	624.500	0.43	619.71	620.64	3.9
56	U2 Turbine Building	2TB-B	66782	620.373	0.63	619.70	620.63	-0.3
57	U2 Turbine Building	2TB 308	67445	620.318	0.53	619.70	620.63	-0.3
58	U2 Turbine Building	2TB-A	65462	620.325	0.54	619.76	620.69	-0.4
59	U2 Turbine Building	2TB 307	66130	620.311	0.49	619.75	620.68	-0.4
60	U2 Heater Bay	2HB 303	78716	620.476	0.65	619.81	620.74	-0.3
61	U2 Heater Bay	2HB 306	74889	620.430	0.88	619.75	620.68	-0.2
62	U2 Turbine Power Complex	2TP 303	72999	620.377	0.28	619.55	620.48	-0.1
63	U2 Off Gas Building	2OG 304	66467	620.461	0.59	619.74	620.67	-0.2
64	U2 Off Gas Building	2OG 302	68454	620.422	0.47	619.59	620.52	-0.1
65	U2 Off Gas Building	2OG 303	68782	620.477	0.48	619.60	620.53	-0.1
66	Service Water Pump	SW 102	58081	620.000	0.15	618.55	619.48	0.5
67	Service Water Pump	SW105	61183	620.000	0.29	618.75	619.68	0.3
68	Service Water Pump	SW 103	57039	620.000	0.08	618.17	619.10	0.9
69	Service Water Pump	SW 104	56698	620.000	0.28	618.42	619.35	0.6
70	Service Water Pump	SW 101	61190	620.000	0.54	618.80	619.73	0.3
71	Emergency Water Pump	EW 202	65576	619.500	0.44	618.90	619.83	-0.3
72	Emergency Water Pump	EW 201	64916	619.500	0.54	619.09	620.02	-0.5
73	Emergency Water Pump	EW East	66916	619.500	0.65	619.10	620.03	-0.5
74	Circulating Water Pump	U2 CW 101	96539	621.500	0.39	620.59	621.52	0.0

Updated Design Basis LIP Flood Hazard Results at Door Locations for Front LIP Temporal Distribution

POI ID Number	Building	Door Number	Grid Number	Door Elev.	Max Flow Depth Above Ground Elevation	Max WSE		Margin Δ = Door Elev - Max WSE
				ft PNPP Datum	ft	ft NAVD88	ft PNPP Datum	(ft)
75	Circulating Water Pump	U2 CW 102	96901	621.500	0.80	620.60	621.53	0.0
76	Circulating Water Pump	U2 CW 103	97636	626.500	0.07	620.92	621.85	4.7
77	Circulating Water Pump	U2 CW 104	95463	626.500	0.03	624.38	625.31	1.2
78	Circulating Water Pump	U2 CW South	97994	624.500	0.40	620.61	621.54	3.0
79	Circulating Water Pump	U1 CW 101	83930	621.500	0.13	619.95	620.88	0.6
80	Circulating Water Pump	U1 CW 102	83929	621.500	0.03	619.95	620.88	0.6
81	Circulating Water Pump	U1 CW 103	85532	626.500	0.03	620.85	621.78	4.7
82	Circulating Water Pump	U1 CW 104	84900	626.500	0.03	624.55	625.48	1.0
83	Circulating Water Pump	U1 CW South	84887	624.500	0.27	620.22	621.15	3.3

Updated Design Basis LIP Flood Hazard Results
Flood Duration above Door Elevation for Front Temporal Distribution

POI ID Number	Building	Door Number	Flooding Duration Hours	Maximum Water Surface Elevation ft PNPP Datum
1	U1 Auxiliary Building	1AX 406	Door Not Flooded	620.2
2	U1 Auxiliary Building	1AX 404	Door Not Flooded	620.3
3	U1 Auxiliary Building	1AX 403	Door Not Flooded	620.2
4	U1 Auxiliary Building	1AX 407	Door Not Flooded	620.4
5	U1 Auxiliary Building	1AX 405	Door Not Flooded	620.5
6	U1 Turbine Building	1TB 307	Door Not Flooded	620.5
7	U1 Turbine Building	1TB 308	Door Not Flooded	620.1
8	U1 Turbine Building	1TB 306	Door Not Flooded	619.8
9	U1 Turbine Building	1TB 316	Door Not Flooded	620.3
10	U1 Heater Bay	1HB 306	Door Not Flooded	620.1
11	U1 Heater Bay	1HB 310	Door Not Flooded	620.2
12	U1 Heater Bay	1HB 314	Door Not Flooded	620.7
13	U1 Heater Bay	1HB 311	Door Not Flooded	620.3
14	U1 Heater Bay	1HB 309	Door Not Flooded	620.4
15	U1 Heater Bay	1HB 308	Door Not Flooded	620.2
16	U1 Turbine Power Complex	1TP 306	Door Not Flooded	620.3
17	U1 Turbine Power Complex	1TP 303	Door Not Flooded	620.5
18	U1 Off Gas Building	1OG 303	Door Not Flooded	620.4
19	U1 Off Gas Building	1OG 302	Door Not Flooded	620.4
20	U1 Off Gas Building	1OG 304	Door Not Flooded	620.5
21	U1 Water Treatment Building	0WT 101	Door Not Flooded	620.2
22	U1 Water Treatment Building	0WT 103	Door Not Flooded	620.4
23	U1 Water Treatment Building	0WT 102	Door Not Flooded	620.4
24	Diesel Generator Building	0DG 106	Door Not Flooded	620.6
25	Diesel Generator Building	0DG 105	Door Not Flooded	620.6
26	Diesel Generator Building	0DG 104	Door Not Flooded	620.7
27	Diesel Generator Building	0DG 103	Door Not Flooded	620.7

Updated Design Basis LIP Flood Hazard Results
Flood Duration above Door Elevation for Front Temporal Distribution

POI ID Number	Building	Door Number	Flooding Duration	Maximum Water Surface Elevation
			Hours	ft PNPP Datum
28	Diesel Generator Building	0DG 102	Door Not Flooded	620.5
29	Diesel Generator Building	0DG 101	Door Not Flooded	620.5
30	Radwaste Building	0RW 422	Door Not Flooded	620.5
31	Radwaste Building	0RW 407	Door Not Flooded	620.4
32	Radwaste Building	0RW 411	Door Not Flooded	620.4
33	Radwaste Building	0RW 101	Door Not Flooded	620.4
34	Intermediate Building	0IB 315	Door Not Flooded	620.2
35	Intermediate Building	0IB 316	Door Not Flooded	620.4
36	Intermediate Building	0IB 317	Door Not Flooded	620.5
37	Service Building	0SB 113	0.7	620.5
38	Service Building	0SB 114	0.6	620.5
39	Service Building	0SB 107	0.3	620.5
40	Service Building	0SB 104	0.8	620.5
41	Service Building	0SB 105	0.5	620.6
42	Service Building	0SB 108	0.9	620.6
43	Service Building	0SB 103	1.0	620.6
44	Service Building	0SB 102	1.0	620.6
45	Service Building	0SB 101	0.6	620.6
46	U2 Auxiliary Building	2AX 403	1.6	620.6
47	U2 Auxiliary Building	2AX 404	2.3	620.6
48	U2 Auxiliary Building	2AX 406	3.1	620.7
49	U2 Auxiliary Building	2AX 405	Door Not Flooded	620.5
50	U2 Auxiliary Building	2AX 407	Door Not Flooded	620.5
51	U2 Turbine Building	2TB-E	2.1	620.7
52	U2 Turbine Building	2TB-D	6.2 (use maximum of 6 hrs)	620.7
53	U2 Turbine Building	2TB 316	1.8	620.7
54	U2 Turbine Building	2TB-C	3.8	620.7
55	U2 Turbine Building	2TB 306	Door Not Flooded	620.6
56	U2 Turbine Building	2TB-B	6.8 (use maximum of 6 hrs)	620.6
57	U2 Turbine Building	2TB 308	9.7 (use maximum of 6 hrs)	620.6

Updated Design Basis LIP Flood Hazard Results
Flood Duration above Door Elevation for Front Temporal Distribution

POI ID Number	Building	Door Number	Flooding Duration Hours	Maximum Water Surface Elevation ft PNPP Datum
58	U2 Turbine Building	2TB-A	4.0	620.7
59	U2 Turbine Building	2TB 307	3.5	620.7
60	U2 Heater Bay	2HB 303	1.7	620.7
61	U2 Heater Bay	2HB 306	5.8	620.7
62	U2 Turbine Power Complex	2TP 303	1.3	620.5
63	U2 Off Gas Building	2OG 304	1.5	620.7
64	U2 Off Gas Building	2OG 302	0.9	620.5
65	U2 Off Gas Building	2OG 303	0.3	620.5
66	Service Water Pump	SW 102	Door Not Flooded	619.5
67	Service Water Pump	SW105	Door Not Flooded	619.7
68	Service Water Pump	SW 103	Door Not Flooded	619.1
69	Service Water Pump	SW 104	Door Not Flooded	619.4
70	Service Water Pump	SW 101	Door Not Flooded	619.7
71	Emergency Water Pump	EW 202	6.1 (use maximum of 6 hrs)	619.8
72	Emergency Water Pump	EW 201	10.0 (use maximum of 6 hrs)	620.0
73	Emergency Water Pump	EW East	7.3 (use maximum of 6 hrs)	620.0
74	Circulating Water Pump	U2 CW 101	Door Not Flooded	621.5
75	Circulating Water Pump	U2 CW 102	Door Not Flooded	621.5
76	Circulating Water Pump	U2 CW 103	Door Not Flooded	621.8
77	Circulating Water Pump	U2 CW 104	Door Not Flooded	625.3
78	Circulating Water Pump	U2 CW South	Door Not Flooded	621.5
79	Circulating Water Pump	U1 CW 101	Door Not Flooded	620.9
80	Circulating Water Pump	U1 CW 102	Door Not Flooded	620.9
81	Circulating Water Pump	U1 CW 103	Door Not Flooded	621.8
82	Circulating Water Pump	U1 CW 104	Door Not Flooded	625.5
83	Circulating Water Pump	U1 CW South	Door Not Flooded	621.2

ATTACHMENT 2

Beyond Design Basis LIP Flood Hazard Results
End Temporal Distribution
(Reference PNPP 2016c)

Beyond Design Basis LIP Flood Hazard Results at Door Locations for End LIP Temporal Distribution

POI ID Number	Building	Door Number	Grid Number	Door Elev. ft PNPP Datum	Max Flow Depth Above Ground Elevation ft	Max WSE ft NAVD88	Max WSE ft PNPP Datum	Margin Δ = Door Elev - Max WSE (ft)
1	U1 Auxiliary Building	1AX 406	74326	620.504	0.51	619.67	620.60	-0.1
2	U1 Auxiliary Building	1AX 404	74642	620.465	0.33	619.69	620.62	-0.2
3	U1 Auxiliary Building	1AX 403	75274	620.456	1.02	619.73	620.66	-0.2
4	U1 Auxiliary Building	1AX 407	68496	620.614	0.69	619.76	620.69	-0.1
5	U1 Auxiliary Building	1AX 405	68499	620.506	0.32	619.76	620.69	-0.2
6	U1 Turbine Building	1TB 307	59754	620.500	0.31	619.74	620.67	-0.2
7	U1 Turbine Building	1TB 308	58385	620.500	0.08	619.26	620.19	0.3
8	U1 Turbine Building	1TB 306	61482	624.500	0.38	618.98	619.91	4.6
9	U1 Turbine Building	1TB 316	71777	620.392	0.30	619.62	620.55	-0.2
10	U1 Heater Bay	1HB 306	64199	620.410	0.05	619.18	620.11	0.3
11	U1 Heater Bay	1HB 310	68856	620.385	0.26	619.52	620.45	-0.1
12	U1 Heater Bay	1HB 314	67868	620.870	0.10	619.79	620.72	0.2
13	U1 Heater Bay	1HB 311	67870	620.416	0.14	619.45	620.38	0.0
14	U1 Heater Bay	1HB 309	70808	620.610	0.15	619.58	620.51	0.1
15	U1 Heater Bay	1HB 308	71778	620.261	0.49	619.62	620.55	-0.3
16	U1 Turbine Power Complex	1TP 306	73051	620.420	0.52	619.66	620.59	-0.2
17	U1 Turbine Power Complex	1TP 303	67841	620.516	0.26	619.76	620.69	-0.2
18	U1 Off Gas Building	1OG 303	63836	620.467	0.37	619.69	620.62	-0.2
19	U1 Off Gas Building	1OG 302	63499	620.431	0.34	619.69	620.62	-0.2
20	U1 Off Gas Building	1OG 304	61123	620.467	0.24	619.73	620.66	-0.2
21	U1 Water Treatment Building	0WT 101	55241	620.401	0.45	619.50	620.43	0.0
22	Water Treatment Building	0WT 103	55936	620.399	0.58	619.71	620.64	-0.2
23	Water Treatment Building	0WT 102	58372	620.429	0.79	619.74	620.67	-0.2

Beyond Design Basis LIP Flood Hazard Results at Door Locations for End LIP Temporal Distribution

POI ID Number	Building	Door Number	Grid Number	Door Elev. ft PNPP Datum	Max Flow Depth Above Ground Elevation ft	Max WSE ft NAVD88	Max WSE ft PNPP Datum	Margin Δ = Door Elev - Max WSE (ft)
24	Diesel Generator Building	0DG 106	64479	620.512	0.44	619.89	620.82	-0.3
25	Diesel Generator Building	0DG 105	64146	620.547	0.31	619.88	620.81	-0.3
26	Diesel Generator Building	0DG 104	63813	620.680	0.11	619.86	620.79	-0.10
27	Diesel Generator Building	0DG 103	63478	620.650	0.13	619.85	620.78	-0.1
28	Diesel Generator Building	0DG 102	63143	620.503	0.27	619.83	620.76	-0.3
29	Diesel Generator Building	0DG 101	62808	620.487	0.30	619.79	620.72	-0.2
30	Radwaste Building	0RW 422	62470	620.459	0.39	619.80	620.73	-0.3
31	Radwaste Building	0RW 407	62132	620.393	0.44	619.80	620.73	-0.3
32	Radwaste Building	0RW 411	61457	620.435	0.40	619.74	620.67	-0.2
33	Radwaste Building	0RW 101	62819	620.473	0.66	619.69	620.62	-0.1
34	Intermediate Building	0IB 315	76859	620.594	1.07	619.79	620.72	-0.1
35	Intermediate Building	0IB 316	78439	620.570	0.84	619.97	620.90	-0.3
36	Intermediate Building	0IB 317	78120	620.416	0.79	619.99	620.92	-0.5
37	Service Building	0SB 113	71398	620.430	0.91	619.95	620.88	-0.4
38	Service Building	0SB 114	71721	620.434	0.78	619.95	620.88	-0.4
39	Service Building	0SB 107	70749	620.466	0.95	619.96	620.89	-0.4
40	Service Building	0SB 104	70098	620.436	0.92	619.97	620.90	-0.5
41	Service Building	0SB 105	68131	620.493	0.93	620.00	620.93	-0.4
42	Service Building	0SB 108	66808	620.477	0.89	620.03	620.96	-0.5
43	Service Building	0SB 103	66476	620.482	0.86	620.03	620.96	-0.5
44	Service Building	0SB 102	64810	620.461	0.83	619.98	620.91	-0.5
45	Service Building	0SB 101	64477	620.467	0.58	619.91	620.84	-0.4

Beyond Design Basis LIP Flood Hazard Results at Door Locations for End LIP Temporal Distribution

POI ID Number	Building	Door Number	Grid Number	Door Elev. ft PNPP Datum	Max Flow Depth Above Ground Elevation ft	Max WSE ft NAVD88	Max WSE ft PNPP Datum	Margin Δ = Door Elev - Max WSE (ft)
46	U2 Auxiliary Building	2AX 403	78748	620.419	0.78	620.05	620.98	-0.6
47	U2 Auxiliary Building	2AX 404	78745	620.415	0.65	620.08	621.01	-0.6
48	U2 Auxiliary Building	2AX 406	79062	620.439	0.67	620.09	621.02	-0.6
49	U2 Auxiliary Building	2AX 405	73000	620.546	0.69	619.94	620.87	-0.3
50	U2 Auxiliary Building	2AX 407	72041	620.469	0.85	619.95	620.88	-0.4
51	U2 Turbine Building	2TB-E	77146	620.499	0.71	620.14	621.07	-0.6
52	U2 Turbine Building	2TB-D	77780	620.426	0.80	620.14	621.07	-0.6
53	U2 Turbine Building	2TB 316	80003	620.440	1.20	620.37	621.30	-0.9
54	U2 Turbine Building	2TB-C	80320	620.311	1.30	620.40	621.33	-1.0
55	U2 Turbine Building	2TB 306	70722	624.500	0.90	620.18	621.11	3.4
56	U2 Turbine Building	2TB-B	66782	620.373	1.09	620.16	621.09	-0.7
57	U2 Turbine Building	2TB 308	67445	620.318	0.99	620.16	621.09	-0.8
58	U2 Turbine Building	2TB-A	65462	620.325	0.89	620.11	621.04	-0.7
59	U2 Turbine Building	2TB 307	66130	620.311	0.84	620.10	621.03	-0.7
60	U2 Heater Bay	2HB 303	78716	620.476	1.25	620.41	621.34	-0.9
61	U2 Heater Bay	2HB 306	74889	620.430	1.38	620.25	621.18	-0.7
62	U2 Turbine Power Complex	2TP 303	72999	620.377	0.67	619.94	620.87	-0.5
63	U2 Off Gas Building	2OG 304	66467	620.461	0.93	620.08	621.01	-0.6
64	U2 Off Gas Building	2OG 302	68454	620.422	0.89	620.01	620.94	-0.5
65	U2 Off Gas Building	2OG 303	68782	620.477	0.89	620.01	620.94	-0.5
66	Service Water Pump	SW 102	58081	620.000	0.27	618.67	619.60	0.4
67	Service Water Pump	SW105	61183	620.000	0.57	619.03	619.96	0.0
68	Service Water Pump	SW 103	57039	620.000	0.27	618.36	619.29	0.7
69	Service Water Pump	SW 104	56698	620.000	0.47	618.61	619.54	0.5
70	Service Water Pump	SW 101	61190	620.000	0.87	619.13	620.06	-0.1
71	Emergency Water Pump	EW 202	65576	619.500	0.84	619.30	620.23	-0.7

Beyond Design Basis LIP Flood Hazard Results at Door Locations for End LIP Temporal Distribution

POI ID Number	Building	Door Number	Grid Number	Door Elev.	Max Flow Depth Above Ground Elevation	Max WSE		Margin Δ = Door Elev - Max WSE
				ft PNPP Datum	ft	ft NAVD88	ft PNPP Datum	(ft)
72	Emergency Water Pump	EW 201	64916	619.500	1.00	619.55	620.48	-1.0
73	Emergency Water Pump	EW East	66916	619.500	1.10	619.55	620.48	-1.0
74	Circulating Water Pump	U2 CW 101	96539	621.500	0.82	621.02	621.95	-0.5
75	Circulating Water Pump	U2 CW 102	96901	621.500	1.23	621.03	621.96	-0.5
76	Circulating Water Pump	U2 CW 103	97636	626.500	0.28	621.13	622.06	4.4
77	Circulating Water Pump	U2 CW 104	95463	626.500	0.08	624.43	625.36	1.1
78	Circulating Water Pump	U2 CW South	97994	624.500	0.84	621.05	621.98	2.5
79	Circulating Water Pump	U1 CW 101	83930	621.500	0.20	620.02	620.95	0.6
80	Circulating Water Pump	U1 CW 102	83929	621.500	0.06	619.98	620.91	0.6
81	Circulating Water Pump	U1 CW 103	85532	626.500	0.22	621.04	621.97	4.5
82	Circulating Water Pump	U1 CW 104	84900	626.500	0.03	624.55	625.48	1.0
83	Circulating Water Pump	U1 CW South	84887	624.500	0.45	620.40	621.33	3.2

Beyond Design Basis LIP Flood Hazard Results
Flood Duration above Door Elevation for End Temporal Distribution

POI ID Number	Building	Door Number	Flooding Duration	Maximum Water Surface Elevation
			Hours	ft PNPP Datum
1	U1 Auxiliary Building	1AX 406	0.30	620.6
2	U1 Auxiliary Building	1AX 404	0.40	620.6
3	U1 Auxiliary Building	1AX 403	0.40	620.7
4	U1 Auxiliary Building	1AX 407	0.10	620.7
5	U1 Auxiliary Building	1AX 405	0.20	620.7
6	U1 Turbine Building	1TB 307	0.40	620.7
7	U1 Turbine Building	1TB 308	Door Not Flooded	620.2
8	U1 Turbine Building	1TB 306	Door Not Flooded	619.9
9	U1 Turbine Building	1TB 316	0.40	620.6
10	U1 Heater Bay	1HB 306	Door Not Flooded	620.1
11	U1 Heater Bay	1HB 310	0.20	620.5
12	U1 Heater Bay	1HB 314	Door Not Flooded	620.7
13	U1 Heater Bay	1HB 311	Door Not Flooded	620.4
14	U1 Heater Bay	1HB 309	Door Not Flooded	620.5
15	U1 Heater Bay	1HB 308	0.50	620.6
16	U1 Turbine Power Complex	1TP 306	0.40	620.6
17	U1 Turbine Power Complex	1TP 303	0.20	620.7
18	U1 Off Gas Building	1OG 303	0.20	620.6
19	U1 Off Gas Building	1OG 302	0.30	620.6
20	U1 Off Gas Building	1OG 304	0.90	620.7
21	U1 Water Treatment Building	0WT 101	Door Not Flooded	620.4
22	U1 Water Treatment Building	0WT 103	0.50	620.6
23	U1 Water Treatment Building	0WT 102	0.50	620.7
24	Diesel Generator Building	0DG 106	0.80	620.8
25	Diesel Generator Building	0DG 105	0.60	620.8
26	Diesel Generator Building	0DG 104	0.40	620.8
27	Diesel Generator Building	0DG 103	0.50	620.8
28	Diesel Generator Building	0DG 102	0.70	620.8
29	Diesel Generator Building	0DG 101	0.40	620.7
30	Radwaste Building	0RW 422	0.60	620.7
31	Radwaste Building	0RW 407	0.70	620.7
32	Radwaste Building	0RW 411	0.50	620.7

Beyond Design Basis LIP Flood Hazard Results
Flood Duration above Door Elevation for End Temporal Distribution

POI ID Number	Building	Door Number	Flooding Duration Hours	Maximum Water Surface Elevation ft PNPP Datum
33	Radwaste Building	0RW 101	0.20	620.6
34	Intermediate Building	0IB 315	0.30	620.7
35	Intermediate Building	0IB 316	0.50	620.9
36	Intermediate Building	0IB 317	0.80	620.9
37	Service Building	0SB 113	0.80	620.9
38	Service Building	0SB 114	0.80	620.9
39	Service Building	0SB 107	0.80	620.9
40	Service Building	0SB 104	0.90	620.9
41	Service Building	0SB 105	0.80	620.9
42	Service Building	0SB 108	0.90	621.0
43	Service Building	0SB 103	0.90	621.0
44	Service Building	0SB 102	1.00	620.9
45	Service Building	0SB 101	0.80	620.8
46	U2 Auxiliary Building	2AX 403	1.20	621.0
47	U2 Auxiliary Building	2AX 404	1.60	621.0
48	U2 Auxiliary Building	2AX 406	1.90	621.0
49	U2 Auxiliary Building	2AX 405	0.40	620.9
50	U2 Auxiliary Building	2AX 407	0.60	620.9
51	U2 Turbine Building	2TB-E	1.60	621.1
52	U2 Turbine Building	2TB-D	6.10(use maximum of 6 hour)	621.1
53	U2 Turbine Building	2TB 316	1.30	621.3
54	U2 Turbine Building	2TB-C	1.70	621.3
55	U2 Turbine Building	2TB 306	Door Not Flooded	621.1
56	U2 Turbine Building	2TB-B	4.40	621.1
57	U2 Turbine Building	2TB 308	6.50(use maximum of 6 hour)	621.1
58	U2 Turbine Building	2TB-A	1.90	621.0
59	U2 Turbine Building	2TB 307	1.80	621.0
60	U2 Heater Bay	2HB 303	1.30	621.3
61	U2 Heater Bay	2HB 306	2.70	621.2
62	U2 Turbine Power Complex	2TP 303	1.00	620.9
63	U2 Off Gas Building	2OG 304	1.20	621.0
64	U2 Off Gas Building	2OG 302	0.90	620.9

Beyond Design Basis LIP Flood Hazard Results
Flood Duration above Door Elevation for End Temporal Distribution

POI ID Number	Building	Door Number	Flooding Duration Hours	Maximum Water Surface Elevation ft PNPP Datum
65	U2 Off Gas Building	2OG 303	0.80	620.9
66	Service Water Pump	SW 102	Door Not Flooded	619.6
67	Service Water Pump	SW105	Door Not Flooded	620.0
68	Service Water Pump	SW 103	Door Not Flooded	619.3
69	Service Water Pump	SW 104	Door Not Flooded	619.5
70	Service Water Pump	SW 101	0.10	620.1
71	Emergency Water Pump	EW 202	5.80	620.2
72	Emergency Water Pump	EW 201	7.50 (use maximum of 6 hour)	620.5
73	Emergency Water Pump	EW East	7.10 (use maximum of 6 hour)	620.5
74	Circulating Water Pump	U2 CW 101	0.70	622.0
75	Circulating Water Pump	U2 CW 102	0.70	622.0
76	Circulating Water Pump	U2 CW 103	Door Not Flooded	622.1
77	Circulating Water Pump	U2 CW 104	Door Not Flooded	625.4
78	Circulating Water Pump	U2 CW South	Door Not Flooded	622.0
79	Circulating Water Pump	U1 CW 101	Door Not Flooded	620.9
80	Circulating Water Pump	U1 CW 102	Door Not Flooded	620.9
81	Circulating Water Pump	U1 CW 103	Door Not Flooded	622.0
82	Circulating Water Pump	U1 CW 104	Door Not Flooded	625.5
83	Circulating Water Pump	U1 CW South	Door Not Flooded	621.3