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Engineering Information Record

Document No.: 51 - 9226987 - 000

Palisades Nuclear Plant
Flooding Hazard Re-Evaluation Report

AREVA

Palisades Nuclear Plant
Flooding Hazard Re-Evaluation Report

Safety Related? [X] YES [ ] NO

Does this document establish design or technical requirements? [ ] YES [X] NO

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Signature Block

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DOCUMENT AMENDMENT

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3/9/15	FLS

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Contract No.:	F.502242 - C-02 (SGT 3/9/15) Safety Related: X Yes ☐ No	Document Amendment No:	159 - 9236786 - 000
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Amended Document Title:	Palisades Nuclear Plant Flooding Hazard Re-Evaluation Report		

Description of Change(s):

Referring to Document Amendment No. 159-9236737-000 which was issued to change several figures in Document No. 51-9224283-000 – Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Screening for Tsunami, the subject Document Amendment changes Figures 3-18 and 3-19 in Document No. 51-9226987-000 – Palisades Nuclear Plant Flooding Hazard Re-Evaluation Report, since Figures 3-18 and 3-19 are the same figures as those in the Tsunami Screening. As such, the 'star' designation for the Palisades Nuclear Plant on Figures 3-18 and 3-19 of the Flooding Hazard Re-Evaluation Report needs to be shifted slightly to the north along the southeastern shore of Lake Michigan. Note that these figures were provided for illustration purposes and were not pertinent to the technical content of Document No. 51-9226987-000.

The attached versions of Figures 3-18 and 3-19, (i.e., page 77 of 142 and page 78 of 142, respectively) from Document No. 51-9226987-000 show the correct 'star' designation for the location of the Palisades Nuclear Plant on the southeastern shore of Lake Michigan.

SIGNATURES AND DISTRIBUTION

Amendment Originating Organization

Organization: Engineering and Licensing

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Record of Revision

Revision No.	Pages/Sections/ Paragraphs Changed	Brief Description / Change Authorization
000	All	Initial release.

Executive Summary

Following the Fukushima Dai-ichi accident on March 11, 2011, which resulted from an earthquake and subsequent tsunami, the U.S. Nuclear Regulatory Commission (NRC) established the Near-Term Task Force (NTTF) to review the accident. The NTTF subsequently prepared a report with a comprehensive set of recommendations. Recommendation 2.1 Flooding Enclosure 2 of Title 10 Code of Federal Regulations (CFR) Section 50.54(f) contains a "Requested Information" section which requires a "Hazard Reevaluation Report". This report provides the requested information pursuant to flooding hazards for the Palisades Nuclear Plant (PLP).

The following flood-causing mechanisms were considered in the flood hazard re-evaluation for PLP:

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

In addition, a combined effect flood (i.e., a combination of the probable maximum surge and seiche with wind-wave activity and the appropriate antecedent water level (i.e., 100-year lake water surface elevation)) was also evaluated. Flooding due to local intense precipitation and the combined effect flood are the only flood mechanisms that result in inundation above Current License Basis flood heights; however, plant walkdowns have confirmed that no plant areas in the vicinity of systems, structures or components important to safety would be inundated by these two flood events. Except for sealing of conduits within Manhole #4 (i.e., situated east of Containment), no additional actions are planned.

Overview

This report describes the approach, methods, and results from the re-evaluation of flood hazards at the Palisades Nuclear Plant (PLP). It provides the information, in part, requested by the U.S. Nuclear Regulatory Commission (NRC) to support the evaluation of the NRC staff recommendations for the Near-Term Task Force (NTTF) review of the accident at the Fukushima Dai-ichi nuclear facility.

Section 1.0 provides introductory information related to the flood hazard. The section includes background regulatory information, scope, general method used for the re-evaluation, assumptions and the elevation data used in the report.

Section 2.0 describes detailed PLP site information, including present-day site layout, topography, and current licensing basis flood protection and mitigation features. The section also identifies relevant changes since license issuance to the local area and watershed as well as flood protections.

Section 3.0 presents the results of the flood hazard re-evaluation. It addresses each of the eight flood-causing mechanisms required by the NRC as well as a combined effect flood. In cases where a mechanism does not apply to the PLP site, a justification is included. The section also provides a basis for inputs and assumptions, methods, and models used.

Section 4.0 compares the current and re-evaluated flood-causing mechanisms. It provides an assessment of the current licensing and design basis flood elevation to the re-evaluated flood elevation for each applicable flood-causing mechanism evaluated in Section 3.0.

Section 5.0 presents an interim evaluation and actions taken, or planned, to address those higher flooding hazards identified in Section 4.0 relative to the current licensing and design basis.

Section 6.0 describes the additional actions taken to support the interim actions described in Section 5.0. Note that no additional actions were identified as necessary.

The report also contains one appendix. Appendix A provides large scale drawings of the Local Intense Precipitation model setup and results, as well as relevant input/output files for review of the simulation.

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Acronyms and Abbreviations

Acronym/Abbreviation	Description
ADCIRC	Advanced Circulation
ANSI	American National Standards Institute
ARC	Antecedent Rainfall Condition
ASCE	American Society of Civil Engineers
ASPRS	American Society for Photogrammetry and Remote Sensing
CAP	Corrective Action Program
CEM	Coastal Engineering Manual
CFR	Code of Federal Regulations
CLB	Current License Basis
CN	Curve Number
DTM	Digital Terrain Model
DUT	Delft University of Technology
FEMA	Federal Emergency Management Agency
FSAR	Final Safety Analysis Report
GIS	Geographic Information Systems
GMT	Greenwich Mean Time
HHA	Hierarchical Hazard Assessment
HMR	Hydrometeorological Report
HURDAT	National Hurricane Center Data
IDNR	Indiana Department of Natural Resources
IGLD85	International Great Lakes Datum of 1985
IJC	International Joint Commission
ISG	Interim Staff Guidance (NRC)
LiDAR	Light Detection and Ranging
LIP	Local Intense Precipitation
MSL	Mean Sea Level
NAD83	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NCDC	National Climatic Data Center
NEH	National Engineering Handbook
NGDC	National Geophysical Data Center
NGS	National Geodetic Survey
NGVD29	National Geodetic Vertical Datum of 1929
NID	National Inventory of Dams
NOAA	National Oceanic and Atmospheric Administration

Acronym/Abbreviation	Description
NRC	U.S. Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
NTTF	Near-Term Task Force
PLP	Palisades Nuclear Plant
PM	Preventive Maintenance
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PMS	Probable Maximum Seiche
PMSS	Probable Maximum Storm Surge
PMWS	Probable Maximum Wind Storm
POT	Peaks over Threshold
RMSE	Root Mean Square Error
SCS	Soil Conservation Service
SEP	Systematic Evaluation Program
SSCs	Structures, Systems and Components
SWAN	Simulating Waves Nearshore
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WIS	Wave Information Studies
cfs or ft^3/s	cubic feet per second
fps	feet per second
mi^2	square mile
mph	miles per hour

1.0 INTRODUCTION

Following the Fukushima Dai-ichi accident on March 11, 2011, which resulted from an earthquake and subsequent tsunami, the U.S. Nuclear Regulatory Commission (NRC) established the Near-Term Task Force (NTTF) to review the accident. The NTTF subsequently prepared a report with a comprehensive set of recommendations.

In response to the NTTF recommendations, and pursuant to Title 10 of the Code of Federal Regulations (CFR), Section 50.54(f), the NRC has requested information from all operating power licensees (NRC 2012). The purpose of the request is to gather information to re-evaluate seismic and flooding hazards at U.S. operating reactor sites.

The Palisades Nuclear Plant (PLP), located on the eastern shore of Lake Michigan in Covert Township, Michigan, approximately four and one-half miles south of South Haven, Michigan, is one of the sites required to submit information.

The NRC information request to flooding hazards requires licensees to re-evaluate their sites using updated flooding hazard information and present-day regulatory guidance and methodologies and then compare the results against the site's current licensing basis (CLB) for protection and mitigation from external flood events.

1.1 Purpose

This report satisfies the "Hazard Reevaluation Report" Request for Information pursuant to 10 CFR 50.54(f) by the NRC dated November 12, 2012, NTTF Recommendation 2.1 Flooding Enclosure 2.

The report describes the approach, methods and results from the re-evaluation of flood hazards at PLP.

1.2 Scope

This report addresses the eight flood-causing mechanisms and a combined effect flood, identified in Attachment 1 to Enclosure 2 of the NRC information request (NRC 2012). No additional flood causing mechanisms were identified for PLP.

Each of the re-evaluated flood causing mechanisms and the potential effects on the PLP site are described in Sections 3.0 and 4.0 of this report.

1.3 Method

This report follows the Hierarchical Hazard Assessment (HHA) approach, as described in NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America" (NRC 2011), NRC Interim Staff Guidance (ISG), as appropriate, and their supporting reference documents.

A HHA consists of a series of stepwise, progressively more refined analyses to evaluate the hazard resulting from phenomena at a given nuclear power plant site to structures, systems and components (SSCs) important to safety with the most conservative plausible assumptions consistent with the available data. The HHA starts with the most conservative, simplifying assumptions that maximize the hazards from the maximum probable event. If the assessed hazards result in an adverse effect or exposure to any SSCs important to safety, a more site-specific hazard assessment is performed for the probable maximum event.

The HHA approach was carried out for each flood-causing mechanism, with the controlling flood being the event that resulted in the most severe hazard to the SSCs important to safety at PLP. The steps involved to estimate the design-basis flood typically included the following:

1. Identify flood-causing phenomena or mechanisms by reviewing historical data and assessing the geohydrological and structural failure phenomena in the vicinity of the site and region.
2. For each flood-causing phenomena, develop a conservative estimate of the flood from the corresponding probable maximum event using conservative simplifying assumptions.
3. If any SSCs important to safety are adversely affected by flood hazards, use site-specific data and/or more refined analyses to provide more realistic conditions and flood analysis, while ensuring that these conditions are consistent with those used by Federal agencies in similar design considerations.
4. Repeat Step 2 until all SSCs important to safety are unaffected by the estimated flood, or if all site-specific data and model refinement options have been used.

Section 3.0 of this report provides additional HHA detail for each of the flood-causing mechanisms evaluated.

Due to use of the HHA approach, the results (water elevation) for any given flood hazard mechanism may be significantly higher than results that could be obtained using more refined approaches. Where initial, overly conservative assumptions and inputs result in water elevations bounded by the CLB, no subsequent refined analyses are required to develop flood elevations that are more realistic or reflect a certain level of probability.

1.4 Assumptions

Assumptions used to support the flood re-evaluation are described in Section 3.0 and its subsections, and depend on the mechanism being evaluated. Details relating to assumption justifications are discussed further in referenced, supporting documentation. None of the assumptions require verification, i.e., need to be confirmed prior to use of the results.

1.5 Elevation Values

The PLP Final Safety Analysis Report (FSAR) states that Great Lakes levels are currently reported using the International Great Lakes Datum of 1985 (IGLD85), which is converted to mean sea level (MSL) at the PLP site by adding a correction factor of 0.88 feet (PLP 2012, Sections 2.2.2). The correction factor of 0.88 feet is based on the National Geodetic Vertical Datum of 1929 (NGVD29) reference point at Calumet Harbor, Illinois (PLP 2012, Section 2.2.2 and PLP 2014, Input Source Document #3), and as applicable, used in the screening assessments of flood causing mechanisms (i.e., Dam Breaches and Failures, Tsunami and Ice Induced Flooding).

To convert elevations in IGLD85 to NGVD29 near PLP, 0.78 feet is added as determined using the National Geodetic Survey (NGS) IGLD85 'Height Conversion' and the 'NGS VERTCON' tool (NGS 2014a and NGS 2014b). As applicable, a conversion factor of 0.78 feet was used in the analyses of flood causing mechanisms (i.e., Local Intense Precipitation, Storm Surge and the Combined Effect Flood). NGVD29 is equivalent to the PLP plant datum in the FSAR (see Section 3.1); elevations listed as MSL in this report refer to elevations provided in plant documentation such as the FSAR (i.e., MSL = NGVD29 = IGLD85 + 0.78 feet (AREVA 2015)).

Updated topographic data for the site was developed using aerial light detection and ranging (LiDAR) and supporting ground control surveying performed in 2014 (AREVA 2014). This topographic survey provided results in Michigan State Plane North American Datum of 1983(NAD83), international feet (horizontal) datum, and elevations in North American Vertical Datum of 1988 (NAVD88), (vertical) datum. The unit of the survey is U.S. feet. As noted in Section 3.1, the datum shift from NAVD88 to NGVD29 of 0.48 feet was calculated using the web-based program VERTCON (NGS 2014b) (i.e., MSL = NGVD29 = NAVD88 + 0.48 feet (AREVA 2015)).

1.6 References

AREVA 2014. AREVA Document No. 38-9226943-000, PLP Topographic Survey, 2014.

AREVA 2015. AREVA Document No. 32-9226981-000, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Combined Events, 2015.

NGS 2014a. IGLD 85 Height Conversion, National Geodetic Survey; Available at: http://www.ngs.noaa.gov/TOOLS/IGLD85/igld85.shtml; Accessed June 2, 2014; Date modified November 5, 2012. (See AREVA 32-9226981-000, Appendix D)

NGS 2014b. VERTCON – North American Vertical Datum Conversion, National Geodetic Survey; Available at: http://www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html; Accessed June 2, 2014; Date modified January 24, 2013. (See AREVA Document No. 32-9226981-000, Appendix D)

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

NRC 2012. 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, U.S. Nuclear Regulatory Commission, March 2012.

PLP 2012. Palisades Nuclear Plant Final Safety Analysis Report (FSAR), Revision 30, September 4, 2012. (AREVA Document No. 38-9223712-000)

PLP 2014. Palisades Design Input Record, Document Number PLP RFI #2014-002, dated May 5, 2014. (AREVA Document No. 38-9223712-000)

2.0 INFORMATION RELATED TO THE FLOOD HAZARD

2.1 Site Information

The PLP site consists of approximately 432 acres along the eastern shore of Lake Michigan in Covert Township, on the western side of Van Buren County, about four and one-half miles south of the Town of South Haven, Michigan (PLP 2012a, Sections 1.2.1 and 2.1). See Figure 2-1, Site Location Map. In the vicinity of the site, sand dunes rise from the beach, which is at an approximate elevation of 582 feet above MSL, to 780 feet MSL (PLP 2012a, Section 2.1.1). The sand dunes extend inland about 5,000 feet for approximately two miles north and five miles south of the PLP site (PLP 2012a, Section 2.3).

There are no perennial streams at or adjacent to PLP; the nearest stream to PLP is Brandywine Creek (USGS 1981 and USGS 2011). Due to sand dunes north, east and south of the PLP site, PLP site drainage is independent of the Brandywine Creek drainage basin (PLP 2012a, Section 2.2).

2.1.1 Site Layout

Figure 2-2, Site Topography and Layout, shows the PLP site layout and topography, including important features related to flood hazards.

2.1.2 Site Topography

The PLP site slopes down to the Lake Michigan shoreline at elevation 582 feet MSL from plant grade of 589 feet along the lakeshore to 632.5 feet above MSL in the parking lot which is situated east of the plant's main footprint. General grading around the Auxiliary Building and Turbine Building is at elevation 590 feet (PLP 2012b, Section 2.1.1).

It is unlikely that the PLP beach front will experience changes due to shoreline erosion since PLP's shoreline is stabilized against erosion from wind, currents and water fluctuations with stone riprap which provides protection against erosive forces and protects shoreline structures by preventing the erosion or failure of the underlying soils (AREVA 2014a, Section 5.2.1).

During PLP site investigations, groundwater elevations varied from 580 feet to 590 feet, with an average elevation of 580 feet MSL beneath the building site, corresponding to the water surface elevation of Lake Michigan (PLP 2012a, Sections 2.2.1 and 2.4.1). All below grade portions of structures were designed for hydrostatic pressures based upon a groundwater elevation of 585 feet (PLP 2012a, Section 5.9.1.4). During the 2012 flooding walkdowns that were performed for resolution of Fukushima Near-Term Task Force Recommendation 2.3 (PLP 2012b), required as part of the response to the 10 CFR 50.54(f) letter, hydrostatic loads on structures were evaluated for ground water levels of 589 feet (i.e., lower level of the PLP site) and 625 feet (i.e., upper level of the PLP site), and were found to be acceptable (PLP 2012b, Section 2.1.7).

2.2 Current Design Basis Flood Elevation

The current design basis and related flood elevation from natural sources is described in the PLP FSAR (PLP 2012a, Section 5.4.1).

The PLP design basis flood level is 594.1 feet MSL, which is postulated to occur coincident with the highest recorded lake level and a maximum on shore surge height of 10.9 feet. PLP SSCs important to safety are protected to elevation 594.4 feet MSL from lake flooding.

2.2.1 Elevation of Safety Structures, Systems and Components

The lower bearing lube oil reservoirs associated with the service water pump motors are at an approximate elevation of 594 feet 5-1/2 inches (PLP 2012a, Section 5.4.1.2), and comprise the plant's equipment important to safety at the lowest elevation.

2.3 Current Licensing Basis Flood Protection

The CLB for flooding protection at PLP is described in the FSAR (PLP 2012a, Section 5.4).

2.3.1 CLB Flood Causing Mechanisms

The following is a summary of the flood causing mechanisms that are part of the CLB.

2.3.1.1 Probable Maximum Flood

The local probable maximum flood (PMF), based on the probable maximum precipitation (PMP) of 25.5 inches of rain within six hours, assumed that rainfall occurs in the immediate vicinity of the plant and that related surface runoff would flow towards Lake Michigan (PLP 2012a, Section 5.4.1.1). One half of the peak runoff (555 $ft.^3$/sec) was assumed to pond on the east side of the Service Building to a depth of five feet, i.e., elevation 601 feet; the depth of ponded water elsewhere on site was determined to be less than six inches (PLP 2012a, Section 5.4.1.1).

2.3.1.2 Storm Surge and Seiche

As part of the Systematic Evaluation Program (SEP Topic II-3.B), the maximum probable storm surge was determined to produce an onshore surge height (seiche) of 10.9 feet (PLP 2012a, Section 2.2.2).

2.3.2 CLB Flood Protection and Mitigation Features

The design approach used by PLP for Class 1 structures was determined by the NRC to be more than adequate for resisting the effects of storm surge (i.e., the design basis flood), and the Intake Structure was determined to be able to withstand the dynamic effects of up to approximately eight feet of wave run-up (PLP 2012a, Section 5.4.1.2). The PLP Containment Building is watertight (PLP 2012a, Section 4.3.10). Various barriers (e.g., marine-type watertight doors) protect equipment important to safety against a flood level up to elevation 594.4 feet (PLP 2012a, Section 5.4.1.2). Similarly, Class 1 structures are adequately designed for the PMF; however, ponding due to the PMF on the east side of the Service Building was not evaluated since the Service Building is not a Class 1 structure (PLP 2012a, Section 5.4.1.2).

No temporary or active flood protection measures are required to be installed for the protection of SSCs important to safety during flooding conditions (PLP 2012b). Mitigation features for the protection against external flooding include both incorporated passive and incorporated active features. In addition to the site's topography and drainage systems, flood protection features credited in the CLB include the following (PLP 2012b):

- The walls and floors of the Auxiliary Building, Turbine Building and Screen House/Intake Structure, and penetration seals through exterior walls.
- The concrete top for T-10A, and tank penetration caps.
- Watertight doors in the Auxiliary Building and Turbine Building.
- Check valves in the Auxiliary Building.

SSCs important to safety are protected from flooding due to the location above the postulated maximum flood level or because they are enclosed in reinforced concrete, Category I structures (PLP 2012b). A "hardened" flood protection approach, incorporating structural provisions that protect SSCs important to safety from the static and dynamic effects of a flood, is implemented for PLP Category I structures that may be impacted by a design basis flood (PLP 2012b). The hardened approach includes the installation of watertight doors, equipment hatches, piping and electrical penetrations below the postulated maximum design flood level (PLP 2012b). Seismic Category I structures are essentially equivalent to Class 1 structures (PLP 2012a, Section 5.2.2.1).

Actions to be taken during high lake level conditions, sustained heavy rains and other acts of nature are specified in PLP's Abnormal Operating Procedure (PLP 2014b). Annual inspections of watertight barriers protecting equipment important to safety required to function during accident conditions (i.e., door and hatch gaskets, rubber and fabric boot expansion joints, boot seals, ball check valves, penetration sealant) are performed under the plant's Structural Monitoring Program (PLP 2011, PLP 2012a, Section 1.9.1.19 and PLP 2014c). Based on inspection findings, the maintenance of watertight doors is performed as needed (PLP 2006).

2.4 Licensing Basis Flood-Related and Flood Protection Changes

Originally, PLP was evaluated for protection against flooding and the ability to safely shut down for a flood level up to 594 feet 8 inches based on the service water pump motor windings being the limiting components important to safety (PLP 2012a, Section 5.4.1.2). Subsequently, the plant's flood protection level was changed to elevation 594.4 feet based on the motor lower bearing lube oil reservoir at elevation 594 feet 5.5 inches (PLP 2012a, Section 5.4.1.2).

2.5 Watershed and Local Area Changes

2.5.1 General PLP Site Hydrological Description

There are no perennial streams adjacent to the PLP site. The nearest stream to PLP is Brandywine Creek; however, due to drainage divides on sand dunes situated north, east and south of PLP, PLP site drainage is independent of the Brandywine Creek drainage basin (AREVA 2014a, USGS 1981 and USGS 2011). PLP's drainage basin incorporates a 13.9 acre area. Storm sewers and drainage ditches surround the PLP power block and are designed to largely carry runoff associated with the CLB PMP. There are no storm drain pathways that lead to watertight buildings. Runoff from the east and northeast sides of the site flows into a concrete ditch that discharges into Lake Michigan. Flow in excess of storm water design capacity also drains directly into Lake Michigan via overland flow (PLP 2012a, Section 2.2 and PLP 2012b).

2.5.2 Watershed Changes

The description of Brandywine Creek in the PLP FSAR, circa the mid-1960s, appears similar to its depiction on the 1981 and 2011 site area topographic maps (PLP 2012a, Sections 2.2 and 2.3, USGS 1981 and USGS 2011). Hence, there have been no significant watershed changes to the site vicinity.

2.5.3 Local Area Changes

Additions and relocation of security barriers have a potential impact on localized surface water drainage. The impact of the PLP concrete security barriers is evaluated as part of the Local Intense Precipitation evaluation documented in Section 3.1.

2.6 Additional Site Details – Walkdown Results

A total of 36 walkdown flood protection features that included 36 attributes were reviewed during the walkdown completed at PLP in 2012 (PLP 2012b, Section 7.0).

Of the total flood protection features, 28 were defined as passive - incorporated, none as passive - temporary, eight as active - incorporated and none as active - temporary. The walkdown scope included evaluation of external doors with respect to design specifications and capacity to withstand CLB flood elevations, as well as inspection of concrete walls and floors identified as external flood barriers. None of the flood protection features reviewed was determined to be non-operable or deficient (PLP 2012b, Section 7.1). However, several of the features were not inspected at the time of the 2012 walkdown due to restricted access (PLP 2012b, Section 7.4). Observations that did not meet the walkdown acceptance criteria were entered into the plant's corrective action program (CAP) and tracked accordingly (PLP 2012b, Section 7.3). All related corrective actions and condition reports were subsequently closed.

2.7 References

AREVA 2014a. Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Screening for Channel Diversion, AREVA Document No. 51-9226164-000, dated August 2014.

AREVA 2014b. Palisades Nuclear Plant Topographic Survey, 2014. (AREVA Document No. 38-9226943-000)

ESRI 2014. USA Topo Maps, ESRI Map Service, Date accessed: June 19, 2014, Date modified: March 4, 2014. (AREVA Document No. 32-9226944-002)

PLP 2006. Palisades Nuclear Plant Permanent Maintenance Procedure No. MSM-M-67, Watertight Door Maintenance, Revision 1, dated December 20, 2006. (AREVA Document No. 38-9223712-000)

PLP 2011. Palisades Nuclear Plant Permanent Maintenance Procedure No. MSM-M-16, Inspection of Watertight Barriers, Revision 17, dated January 18, 2011. (AREVA Document No. 38-9223712-000)

PLP 2012a. Palisades Nuclear Plant Final Safety Analysis Report (FSAR), Revision 30, September 4, 2012. (AREVA Document No. 38-9223712-000)

PLP 2012b. Palisades Nuclear Plant Flooding Walkdown Report – Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding the Flooding Aspects of Recommendation 2.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, November 27, 2012. (ADAMS Accession Number ML12332A377)

PLP 2014a. Not Used.

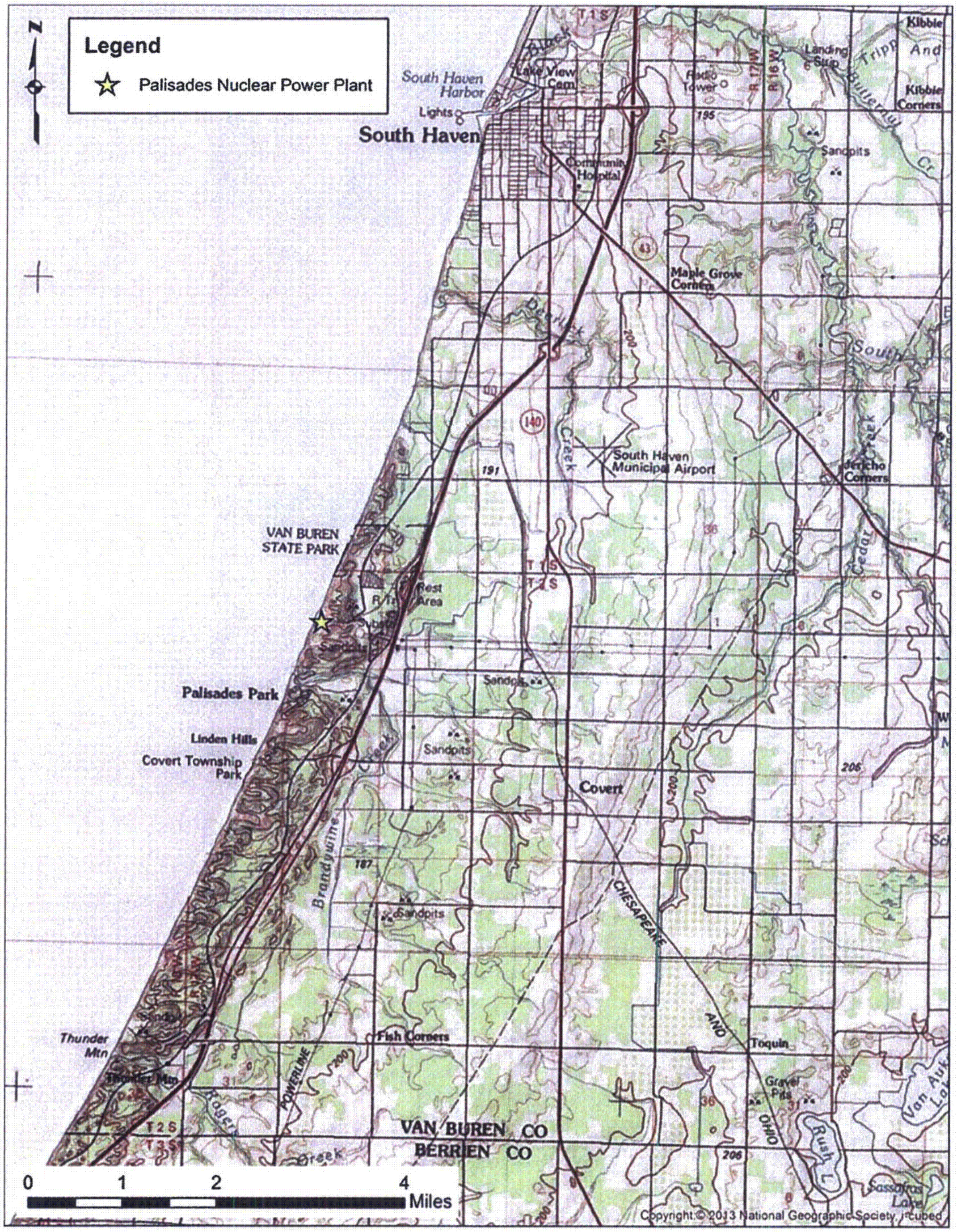
PLP 2014b. Palisades Nuclear Plant Abnormal Operating Procedure, AOP-38 Revision 1, dated April 1, 2014. (AREVA Document No. 38-9223712-000)

PLP 2014c. Palisades Design Input Record, Document Number PLP RFI #2014-002, dated May 5, 2014. (AREVA Document No. 38-9223712-000)

USGS 1981. Covert, Michigan Topographic Quadrangle Map, SE/4 South Haven 15' Quadrangle 1981, Scale 1: 24 000, U.S. Geological Survey. (See AREVA Document No. 51-9226164-000)

USGS 2011. Covert Topographic Quadrangle Map, Michigan 2011 – Van Buren County, 7.5 Minute Series, Scale 1: 24 000, U.S. Geological Survey. (See AREVA Document No. 51-9226164-000)

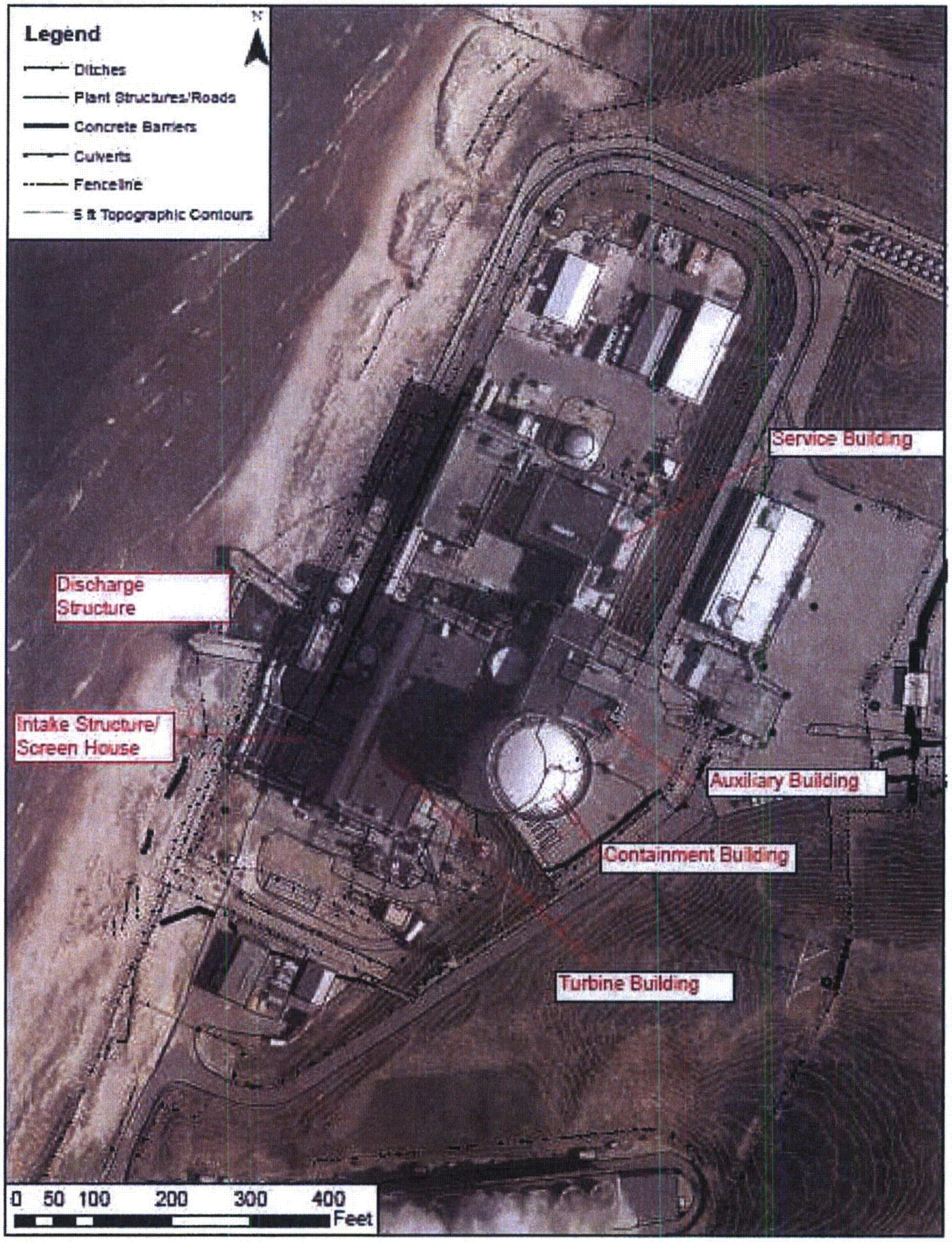
Figure 2-1: Site Location Map



Basemap Source: (ESRI 2014)

Note: Illegible text or features in this figure are not pertinent to the technical purposes of this document.

Figure 2-2: Site Topography and Layout



Note: Any illegible text or features are not pertinent to the technical purposes of this document. Site topography, orthoimagery, and plant structure delineation are from AREVA 2014b.

3.0 FLOOD HAZARD RE-EVALUATION

This section details the evaluation of the eight flood causing mechanisms and combined effect flood for PLP as described in Attachment 1 to Enclosure 2 of the NRC information request. No additional flood causing mechanisms were identified for PLP.

3.1 Local Intense Precipitation

This section addresses the potential for flooding at PLP due to the local intense precipitation (LIP) event. The LIP event is a distinct flooding mechanism that consists of a short-duration, locally heavy rainfall centered upon the plant site itself.

This section summarizes the LIP evaluation performed in AREVA Calculation No. 32-9226944-002 (AREVA 2015).

3.1.1 Method

3.1.1.1 Local Intense Precipitation

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC 2011, Section 2) was used for the evaluation of the LIP and resultant water surface elevation at PLP.

With respect to LIP, the HHA used the following steps:

1. Develop LIP/probable maximum precipitation (PMP) inputs.
2. Develop the FLO-2D computer model with site features.
3. Perform flood simulations in FLO-2D and estimate maximum water surface elevations throughout the PLP site.

3.1.2 Results

Maximum LIP flood elevations calculated by FLO-2D (Table 3-1) near critical locations as displayed in Figure 3-1 indicate flooding at or below the protected flood level elevation of 594.4 feet (PLP 2012, Section 5.4.1) for the portion of the site directly adjacent to Lake Michigan.

3.1.2.1 Local Intense Precipitation

3.1.2.1.1 FLO-2D Model Limits for LIP Analysis

Due to anticipated unconfined flow characteristics, a two-dimensional hydrodynamic computer model, FLO-2D, was used for the LIP analysis. FLO-2D (AREVA 2014b) is a physical process model that routes flood hydrographs and rainfall-runoff over unconfined flow surfaces or in channels using the dynamic wave approximation to the momentum equation (FLO-2D 2013). The FLO-2D model computational boundary is shown in Figure 3-2. The computational domain of the FLO-2D model encompasses the PLP plant and its peripheral site features/structures. The model computational boundary also includes the PLP site drainage basin; therefore, the LIP analysis bounds a probable maximum flood (PMF) evaluation that considers only the PLP drainage area, which is less than the LIP model boundary. The computational domain of the FLO-2D model is bounded by Lake Michigan and topographic ridges to the north, south and east. The total FLO-2D model extent is approximately 230 acres (0.36 square miles) and the PLP site drainage area is approximately 60 acres (0.09 square miles).

The FLO-2D model includes topography, site location and building structures. Grid elements along the model computational boundary were selected as outflow grid elements.

3.1.2.1.2 FLO-2D Computer Model with Site Features

The FLO-2D model developed for the LIP analysis was based on PLP site features including: topography, site location, concrete barrier layout and structures. The selected grid element size for the project was 10 feet by 10 feet. The elevation data used to develop the FLO-2D model consist of 2014 light detection and ranging (LiDAR) site survey data (AREVA 2014c) for PLP. Flow obstructions due to buildings were also included in the model. The main input parameters for the PLP FLO-2D model include:

Elevation: The elevation data used to develop the FLO-2D model were prepared by LiDAR methods using aerial acquisition. The surveyed topographic data of the site were provided as digital terrain model (DTM) contained in an AutoCAD type file (AREVA 2014c) and is in Michigan State Plane NAD83, international feet (horizontal) datum and elevations are in NAVD88 (vertical) datum. The unit of the survey is U.S. feet. The elevations in the FLO-2D model were converted to NGVD29; which is equivalent to the PLP plant datum in the FSAR (PLP 2012). The datum shift from NAVD88 to NGVD29 of 0.48 feet was calculated using the web-based program VERTCON (NGS 2014b).

The topographic data for PLP were developed based on a site-specific aerial survey using methodology consistent with the need for first-order level of accuracy (i.e. +/- 0.1 feet). The topographic survey performed in 2014 at PLP was required to meet the American Society for Photogrammetry and Remote Sensing (ASPRS) Class I Accuracy Standard for 1" = 100' planimetrics and 1-foot contour intervals, with +/- 1 feet horizontal accuracy, +/- 0.33 feet Root Mean Square Error (RMSE) vertical accuracy for 1 foot contours and +/- 0.17 feet RMSE vertical accuracy for spot elevations, at well-defined points. Additional designated critical structures and locations with respect to site flooding impacts were identified and surveyed (AREVA 2014d) with a vertical accuracy of +/- 0.1 feet. The methodology of the topographic survey was aerial LiDAR mapping of the site with sufficient control points for calibration meeting the mapping standard, and conventional ground survey loops for the critical structures and locations (AREVA 2014c).

Model grid elevations cannot be more accurate than the survey they are based upon. Therefore, model grid elevations have a minimum level of uncertainty of +/- 0.1 feet. A minimum of two closest DTM points within the vicinity of a grid element was used in computing grid elevations. The density of LiDAR points on the DTM provided for adequate coverage for each grid element. FLO-2D interpolated elevations for grid elements were spot checked for accuracy and modified as necessary based on recent site topographic survey (AREVA 2014c). Some of the grid cells had their elevations interpolated based on the two distinct grades (i.e., points at one side of the retaining wall are significantly different from the other side) and the resultant interpolated elevations were inaccurate in these cases. Specifically, some FLO-2D ground interpolated elevations near locations with abrupt changes in elevations near the retaining wall located southwest of the Containment Building were manually adjusted to conform to the site survey (AREVA 2014c).

The Discharge Channel (i.e., mixing basin) was conservatively modeled as having "ground" elevations equal to elevation 588.0 feet NGVD29 to account for backwater from the "bridge" located west of the Discharge Channel. The elevation 588.0 feet NGVD29 is the top elevation of the "bridge" based on a site drawing (Bechtel Company 1995). The discharge pipes that connect the Discharge Channel to Lake Michigan were conservatively assumed to be blocked.

The walkway bridge located east of the Service Building that leads to the Auxiliary Building Addition was not included into the FLO-2D model due to its deck configuration that is mostly grated and therefore would not significantly block or reroute flow. It was assumed that the grass slope adjacent to the Service Building extends

underneath the bridge. Therefore, the grid elevations underneath the walkway bridge were modified to preserve the adjacent grass slope. Note that this model simplification/assumption does not impact critical locations.

Generally, the FLO-2D interpolated values for the grid elevations were accurate and therefore, model interpolation errors in the majority of the grid elements are believed to be minimal. Uncertainty regarding onsite flood elevations is generally limited to the level of accuracy of the site survey. The nature of the two dimensional flow model is such that the impact of potential inaccuracy in the elevation of any single grid element is generally mitigated by the surrounding grid elements.

Buildings and Roof Tops: Buildings at PLP were incorporated into the FLO-2D model based on the surveyed topographic site plan (AREVA 2014c) by manually adjusting (increasing) grid element elevations. Buildings were represented by grid elements with a ground elevation at least five feet higher than surrounding areas to ensure that runoff from the roofs freely flows to adjacent ground grid elements and flows around the building footprint (i.e. not through the building). Buildings that appeared to have flat roofs based on the surveyed DTM (AREVA 2014c) were assigned uniform elevations to the grid elements representing a single building so as to ensure that runoff from rooftops is uniformly distributed to the surrounding areas. For buildings with different rooftop elevations adjacent to each other based on the surveyed DTM (AREVA 2014c), the relative changes in rooftop elevations were represented as a minimum 2-feet relative difference in building grid element elevations. The peak 1-hour duration LIP depth of 17.3 inches is less than the relative change in elevation of at least two feet. Therefore, water is not expected to build-up high enough to drive flow from rooftops with lower elevations to adjacent rooftops with higher elevations. This ensures that general flow directions of runoff from rooftops are considered.

Levee Elements: The concrete barrier at the fence west of the Turbine and Feedwater Purity Buildings and the sheeting top along the north and south walls of the Discharge Channel at PLP were modeled using the levee structure component in FLO-2D, shown in Figure 3-3.

The location of the concrete barrier was based on the site topographic survey (AREVA 2014c) and the height was based on a vendor drawing (Kontek 2014a). The concrete barrier height was modeled as 4 feet higher than the FLO-2D model grid element elevations. The top elevations of the levees representing the concrete barrier were based on the interpolated FLO-2D grid elevations and the height of the concrete barrier extracted from the vendor drawing. An approximately 24-foot wide spacing (i.e., opening) within the concrete barrier (Kontek 2014b) where a grated fence gate is in place was conservatively modeled as a 10-foot-wide lower section of the levee that represents the concrete barrier. The significantly shorter opening width modeled as 10 feet within the concrete barrier compared to the 24-foot-wide gate is to conservatively compensate for the potential blockage caused by the gate grates (Kontek 2014b). Also, the modeled 10-foot-wide levee lower section top elevation was modeled as one foot above the FLO-2D grid element elevation to account for a plate located at the bottom of the gate that potentially blocks flows. This plate was observed during a site visit.

The sheeting tops along the north and south walls of the Discharge Channel that are higher than surrounding ground were modeled in FLO-2D using the levee structure component. Note that an approximately 20-foot-wide notch (i.e., section with lower top) is present on both sides of the Discharge Channel. There is a notch in the north side of the south wall and in the south side of the north wall of the Discharge Channel. The top elevations of these notches are generally level with surrounding ground surface. Therefore, the 20-foot-wide notches were modeled as openings at the levees. The locations of the walls and the width of the notch were based on the visual assessment of high resolution orthoimagery included with the survey package (AREVA 2014c). The top elevation of 591.5 feet NGVD29 of the levees representing the sheeting at the north and south walls of the Discharge Channel were based on site topographic information (AREVA 2014d).

Water Surface Elevation at Lake Michigan: As a conservative approach, the highest recorded monthly mean elevation of Lake Michigan was used along the model extent of Lake Michigan as a boundary condition in the FLO-2D model. Lake Michigan is modeled in the LIP analysis as a constant (i.e., in time) conservative elevation along the west boundary of the FLO-2D model. The highest recorded monthly mean elevation of Lake Michigan of 582.37 feet IGLD85, recorded on October 1986, was taken from the National Oceanic and Atmospheric Administration (NOAA) Holland tide station (NOAA 2014). To convert elevations in IGLD85 to NGVD29, add 0.78 feet (NGS 2014a and NGS 2014b). Therefore, the maximum monthly mean elevation converted to NGVD29 is 583.15 feet, which was rounded to 583.2 feet NGVD29.

Calculate Manning's Roughness Coefficients: Manning's n-values used in FLO-2D are composite values that represent flow resistance. An "apparent land cover" Geographic Information Systems (GIS) shape file was created based on visual assessment of high resolution orthoimagery (AREVA 2014c). Grid element Manning's n-values were conservatively assigned based on the land cover at the site, and the recommended upper end of the range of Manning's roughness coefficients contained in Table 1 of FLO-2D Reference Manual (FLO-2D 2013). Table 3-2 shows the relationship between Manning's n-values and selected land cover categories. The Manning's roughness coefficient values for the grid elements generally range from 0.05 for concrete or paved areas to 0.4 for wooded areas. Figure 3-4 shows the Manning's coefficients selection for each land cover.

Calculate Curve Number to Model Infiltration: The Curve Number (CN) Method developed by the Soil Conservation Service (SCS, now known as Natural Resources Conservation Service or NRCS) was used to model infiltration for the LIP analysis. The SCS infiltration method in FLO-2D is computed by subtracting the calculated infiltration loss based on the CN from the total precipitation before the flood routing starts at each grid element, which is similar to using a lower precipitation. CN used in FLO-2D are composite values that represent potential infiltration. The land cover GIS shape file created for the Manning's n-values calculation described above, was used for the CN calculation. The SCS hydrologic soil group classification (A, B, C or D from lowest runoff potential to highest runoff potential) was determined from the Web Soil Survey by NRCS (NRCS 2013) as soil "A" (i.e., well drained) for the entire LIP model computational area.

A GIS shape file was created with CN values assigned by correlating land cover types with soil types ("A" in all areas of PLP drainage basin) based on tables provided in Chapter 9 of the NRCS National Engineering Handbook (NEH) Part 630 Hydrology (NRCS 2004a) that assume normal Antecedent Rainfall Conditions (ARC) II. The selected CNs were then conservatively replaced to assume ARC III (i.e., wet conditions) based on NRCS guidance (Table 10-1 in Chapter 10 "Estimation of Direct Runoff from Storm Rainfall of NEH Part 630 Hydrology"; NRCS 2004b). ARC I, II, and III represent dry, normal and wet conditions, respectively. The GIS shape file with the ARC III CN values was used to compute the grid elements CN in the FLO-2D model. Note that for a land cover category with multiple hydrologic conditions (good, fair and poor), the most conservative option (i.e., poor) was selected.

Table 3-3 shows the relationship between CN values and selected land cover categories. The CN values for the grid elements generally range from 68 for wooded and brush areas to 98 for concrete or paved areas. Note that the CN for the wooded areas were conservatively selected as the same as the "brush-brush-forbs-grass" areas (Table 9-1 of NRCS 2004a). Figure 3-5 shows graphically and the FLO-2D output file "INFIL.DAT" lists the grid element CN values used by the model. According to the FLO-2D output file "SUMMARY.OUT", approximately 3.2 inches or 13-percent of the total precipitation (25.5 inches) was infiltrated or decreased from the total precipitation.

3.1.2.1.3 LIP/PMP Inputs

The LIP parameters were defined using Hydrometeorological Report No. 51 (HMR-51) and HMR-52; (NOAA 1978 and NOAA 1982, respectively) as prescribed in NUREG/CR-7046 (NRC 2011, Section 3.2). The total

rainfall depth for the 1-hour, 1-mi^2 PMP is 17.3 inches, with peak intensity of 5.85 inches during the first 5 minutes of the event. The total rainfall depth for the 6-hour PMP is 25.5 inches. The rainfall hyetograph distribution used as input into the model for the LIP simulation is based on Figure B-5 of NUREG/CR-7046 (NRC 2011). The 6-hour PMP hyetograph was constructed using the 1-hour PMP for the first hour and equal rainfall increments for the next 5 hours (Figure 3-6).

3.1.2.1.4 LIP Simulation Results

Results of the PLP FLO-2D LIP model are summarized in Table 3-1. Large plots showing the LIP grid element numbers, interpolated ground surface elevations, water surface elevations, depths, velocity and direction are provided in Appendix A. Based on the LIP model simulation, the maximum LIP flood elevations near the critical locations range from 592.5 feet (NGVD29) near the north entrance to the Screen House/Intake Structure (Door #33) and three locations in the courtyard, to 594.4 feet (NGVD29) near the Containment Post Tension Tunnel Hatch Door 10A. The maximum LIP flood elevation near the four critical locations on the upper level of the site (approximate site grade of 625 feet NGVD29), was calculated at 626.1 feet NGVD29. Calculated maximum flood depths near the critical locations range from 1.8 feet near the Diesel Generator Fuel Oil Tank T-10A Vent to 5.3 feet near the Containment Post Tension Tunnel Hatch Door 10A. The maximum flood depths near the critical locations in the courtyard generally range from 2.6 feet to 2.8 feet and up to approximately 9.8 feet at the east side of the Service Building. Maximum flow velocities within the model computational area are up to 12.3 feet per second (fps) on top of the "bridge" west of the Discharge Channel (i.e., mixing basin). Time-series plots showing water surface elevation vs. time near the critical locations are shown in Appendix A.

Surface runoff generally discharges to the south, west, and north due to the general topography of the site sloping towards Lake Michigan. Significant upgradient runoff coming from the cooling towers area and runoff from adjacent wooded areas that discharges into the parking lot significantly affect the south side of the Turbine Building and Containment Building area. These flows are constricted between the Intake Structure and a berm near the Intake Structure before flow discharges through the Discharge Channel (i.e., mixing basin) area. High runoff coming from the parking lot area is conveyed between the Service Building Expansion and the Feedwater Purity Building and then flows through the courtyard and is constricted by the concrete barrier before discharging north of the Discharge Channel into Lake Michigan.

Generally, the maximum flood depths occur at the beginning of the simulation when the precipitation intensity is at maximum (see Appendix A). After six hours, the PMP ends and flood depths typically begin to recede toward dry conditions (no flood). Some critical locations (e.g., No. 28 [Manhole No. 4]) maintain considerable flood depths (about one foot) until the end of the simulation (24 hours) possibly due to localized grading that causes local ponding and the conservative assumption of the absence of an underground drainage system included in the LIP model.

The FLO-2D Reference Manual (FLO-2D 2013) provides three keys to a successful project application. These include volume conservation, area of inundation, and maximum velocities and numerical surging.

- Volume Conservation: Reviews of the "SUMMARY.OUT" files (included in Appendix A) indicate volume conservation errors of 0.000003 percent for the FLO-2D runs. This value is well below the threshold of 0.001 percent specified in the FLO-2D Data Input Manual (FLO-2D 2013) for a successful project application.
- Area of Inundation: Reviews of the "SUMMARY.OUT" files (included in Appendix A) indicate maximum inundated areas of 229.3 acres. The FLO-2D model is made up of 99,864 grid elements (see "FPLAIN.DAT" file), each 10 feet by 10 feet in dimension. The LIP was simulated within the entire computational domain of the model. The maximum inundation area should therefore be equal to the area of the computational domain of 229.3 acres ((10 x 10 x 99,864) x (1 acre / 43,560 feet)). The FLO-2D calculated maximum inundation

area matches the computational area. Visual inspection of flood depth results also is consistent with expected results; areas of high flood depth were noted and discussed above. This information indicates a successful project application.

- Maximum Velocities and Numerical Surging: Numerical surging, if it exists, would be evident in unreasonably high velocities in the "VELTIMEFP" (floodplain) file (FLO-2D 2013). A review of the "VELTIMEFP.OUT" file (included in Appendix A) does not indicate unreasonably high velocities in the model runs and indicates a successful project application. The maximum velocity is up to 12.3 feet per second reported in the "VELTIMEFP.OUT" file occurred at the "bridge" west of the Discharge Channel.

3.1.2.1.5 Review Areas of Supercritical Flow

FLO-2D does not simulate supercritical flow conditions (FLO-2D 2013). FLO-2D provides the option to output a text file of grid elements, SUPER.OUT, to identify which grid cells have supercritical flow. This option was selected to identify these areas. Grid elements are determined to be supercritical if the calculated Froude number is greater than 1.0 (FLO-2D 2013). Froude number measures the effect of gravity upon the state of flow, which is represented by a ratio of inertial forces to gravity forces and is defined as follows (Chow 1959):

$$F = \frac{V}{\sqrt{gL}}$$

F = Froude number

V = Mean velocity (feet/second)

g = acceleration of gravity (feet/second^2)

L = characteristic length (feet)

Supercritical flow may occur at several locations within PLP as displayed in Figure 3-7 and described as follows:

- Generally along the intersection of building grid elements and grid elements representing the adjacent grade possibly due to the artificially high hydraulic gradient created by elevating grid elements to represent buildings. According to FLO-2D results (Appendix A), runoff coming from building roofs discharges to areas with generally high depths on adjacent grade where erosion is unlikely.
- Along the retaining wall located southeast of the Containment Building. The retaining wall is expected to act hydraulically as a weir with critical flow conditions (i.e., Froude Number equal 1.0). Therefore, supercritical flow conditions are not expected along the retaining wall.
- Along the "bridge" at the Discharge Structure west of the mixing basin. Erosion is not expected to occur at this location based on the structure configuration and surface type (i.e., concrete).
- Two locations along the steeply sloped cooling tower road. Area 1 is located near the middle of the cooling tower road that runs between the power block and the northern Cooling Tower and Area 2 is located near the toe of the cooling tower road that leads to the Cooling Towers (Figure 3-7).

The FLO-2D model results in conservative estimates for flow depth, because supercritical flow is shallower, and the program limits supercritical flow by reducing the velocity which increases the flow depth. The velocities at the two referenced locations on the cooling tower road were hand calculated using the flow rates calculated at the flood plain cross sections and Manning's equation. The CROSSMAX.OUT file calculated the maximum discharge for the two cross sections as follows:

- Cross section 1 = 26.3 cubic feet per second (cfs).

- Cross section 2 = 187 cfs.

The calculated velocities at each floodplain cross section are as follows:

- Cross section 1 = 5.2 fps.
- Cross section 2 = 15.6 fps.

The permissible velocity when conservatively using rough asphalt is 12 feet per second (USACE 1984). The calculated maximum flow velocity at cross section 2 of 15.6 fps exceeds the referenced permissible velocity for rough asphalt. However, significant erosion of the roadway at this location is not anticipated due to the short duration of high flow rates based on inspection of the hydrograph from the floodplain cross sections (see Appendix A). Erosion that might occur is anticipated to be localized and unlikely to affect critical structures.

3.1.3 Conclusions

The maximum water surface elevations at the site due to the LIP at PLP result from a PMP depth of 17.3 inches in 1 hour and 25.5 inches within 6 hours. The maximum flood depths range from 1.8 feet to locally as high as approximately 5.3 feet above grade near the critical locations on the plant's lower level as shown in Table 3-1.

3.1.4 References

AREVA 2014a. Not Used.

AREVA 2014b. AREVA Document No. 38-9225054-000, Computer Software Certification – FLO-2D Professional Version, Build No. 14.03.07, GZA GeoEnvironmental, Inc., 2014.

AREVA 2014c. Palisades Nuclear Plant Topographic Survey, 2014. (AREVA Document No. 38-9226943-000)

AREVA 2014d. Palisades Nuclear Plant Additional Topographic Survey Information, 2014. (AREVA Document No. 38-9228910-000)

AREVA 2015. AREVA Document No. 32-9226944-002, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Local Intense Precipitation, 2015.

Bechtel Company 1995. Discharge Channel Modification Plan & Sections, Drawing No. C-440, Revision 6. (AREVA Document No. 38-9223712-000)

Chow 1959. Open-Channel Hydraulics, Ven Te Chow, 1959. (See AREVA Document No. 32-9226944-002)

FLO-2D 2013. FLO-2D® Pro Reference Manual, FLO-2D Software, Inc., Nutrioso, Arizona (www.flo-2d.com), 2013. (See AREVA Document No. 32-9226944-002)

Kontek 2014a. Block Details, Drawing No. K14012, Revision 1. (AREVA Document No. 38-9228910-000)

Kontek 2014b. Delay Fence Elevations, Drawing No. K14011, Revision 0. (AREVA Document No. 38-9228910-000)

NGS 2014a. IGLD 85 Height Conversion, National Geodetic Survey; Available at: http://www.ngs.noaa.gov/TOOLS/IGLD85/igld85.shtml; Accessed June 2, 2014; Date modified November 5, 2012. (See AREVA Document No. 32-9226944-002, Appendix A)

NGS 2014b. VERTCON – North American Vertical Datum Conversion, National Geodetic Survey; Available at: http://www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html; Accessed June 2, 2014; Date modified January 24, 2013. (See AREVA Document No. 32-9226944-002, Appendix A)

NOAA 1978. Probable Maximum Precipitation Estimates - United States East of the 105th Meridian, Hydrometeorological Report No. 51 (HMR-51) by US Department of Commerce & USACE, June 1978. (See AREVA Document No. 32-9226944-002)

NOAA 1982. Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, NOAA Hydrometeorological Report No.52 (HMR-52) by US Department of Commerce & USACE, August 1982. (See AREVA Document No. 32-9226944-002)

NOAA 2014. NOAA/NOS/CO-OPS Verified Monthly Means at 9087031, Holland MI, Tides & Currents, National Oceanic and Atmospheric Administration; Available at: http://tidesandcurrents.noaa.gov/stationhome.html?id=9087031; Accessed June 10, 2014; Date modified June 10, 2014. (See AREVA Document No. 32-9226944-002, Appendix H)

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

NRCS 2004a. Chapter 9 Hydrologic Soil-Cover Complexes, Part 630 Hydrology, National Engineering Handbook, U.S. Department of Agriculture Natural Resource Conservation Service, July 2004. (See AREVA Document No. 32-9226944-002)

NRCS 2004b. Chapter 10 Estimation of Direct Runoff from Storm Rainfall, Part 630 Hydrology, National Engineering Handbook, U.S. Department of Agriculture Natural Resource Conservation Service, July 2004. (See AREVA Document No. 32-9226944-002)

NRCS 2013. Web Soil Survey, U.S. Department of Agriculture Natural Resource Conservation Service; Available at: http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm; Accessed September 26, 2014; Date modified December 6, 2013. (See AREVA Document No. 32-9226944-002, Appendix E)

PLP 2012. PLP Final Safety Analysis Report, Revision 30, 2012. (AREVA Document No. 38-9223712-000)

USACE 1984. Drainage and Erosion Control Mobilization Construction, U.S. Army Corps of Engineers, EM 1110-3-136, April 1984. (See AREVA Document No. 32-9226944-002)

Table 3-1: LIP Model Results

Identification Number[A]	Description	Representative Grid Element	Grid Element Ground Surface Elevation (feet, NGVD29)	Maximum Flood Elevation (feet, NGVD29)	Maximum Flood Depth[B] (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
19	Screen House/Intake Structure Roll-Up (Door #14)	44061	589.9	593.1	3.3	0.5	1.1
20	North Entrance to Screen House/Intake Structure (Door #33)	41333	589.7	592.5	2.7	0.5	0.6
21	Turbine Building Laydown Area Access (Door #12)	45229	589.7	593.4	3.8	0.5	1.1
22	Turbine Building Southwest Roll-Up Door (Door #13)	45229	589.7	593.4	3.8	0.5	1.1
23	Diesel Generator Fuel Oil Tank T-10A Vent	48689	591.9	593.6	1.8	0.5	1.9
25	Door to Transformer Yard from Feedwater Pumps (Door #11)	43688	589.8	594.2	4.4	0.5	1.6
26	Turbine Building Southside Roll-Up Door to Transformer Yard (Door #10)	44078	589.7	594.3	4.6	0.5	1.5
27	Containment Post Tension Tunnel Hatch (Door #10A)	45246	589.1	594.4	5.3	0.5	1.8
28	Manhole #4 (East of Containment Building)	41762	623.9	626.1	2.2	0.2	1
29	North Chained Double Door to Diesel Generators (Door #170)	35828	589.7	592.5	2.8	0.5	0.4
33	Turbine Building North Entrance Door	36221	589.9	592.5	2.6	0.5	0.4

Identification Number[A]	Description	Representative Grid Element	Grid Element Ground Surface Elevation (feet, NGVD29)	Maximum Flood Elevation (feet, NGVD29)	Maximum Flood Depth[B] (feet)	Time to Maximum Flood Elevation (hours)	Maximum Velocity (feet per second)
34	Track Alley Rollup Door	37820	624.9	626.0	1.1	0.2	0.3
35	Admin Building Hallway East Entrance (Door #28)	40585	624.7	626.1	1.4	0.2	0.7
36	North Penetration Room (Door #106)	40969	625.0	626.1	1.0	0.2	0.3
230	South Stairwell (Service Bldg Addition) Across from Elevator (Door #123)	35038	589.8	592.5	2.7	0.5	0.9

Notes:

A. The identification number locations are displayed in Figure 3-1. Other locations used as referenced positions in the site topographic survey additional request (Location Nos. 1 to 18) and Location Nos. 24, 30, 31 and 32 were not identified as critical locations, and were not included in Table 3-1.

B. LIP results are rounded to one decimal place. Therefore, the maximum flood depth was approximately calculated by subtracting the grid element ground surface elevation from the maximum flood elevation.

Table 3-2: Manning's n-values for Selected Land Cover Categories

Land Cover Category	Manning's n-value
Paved / Concrete	0.05
Building Roofs	0.05
Dune Grass	0.4
Short Grass	0.4
Sandy Areas with No Cover	0.1
Trees	0.4
Brush	0.4
Open Ground	0.1
Water	0.025

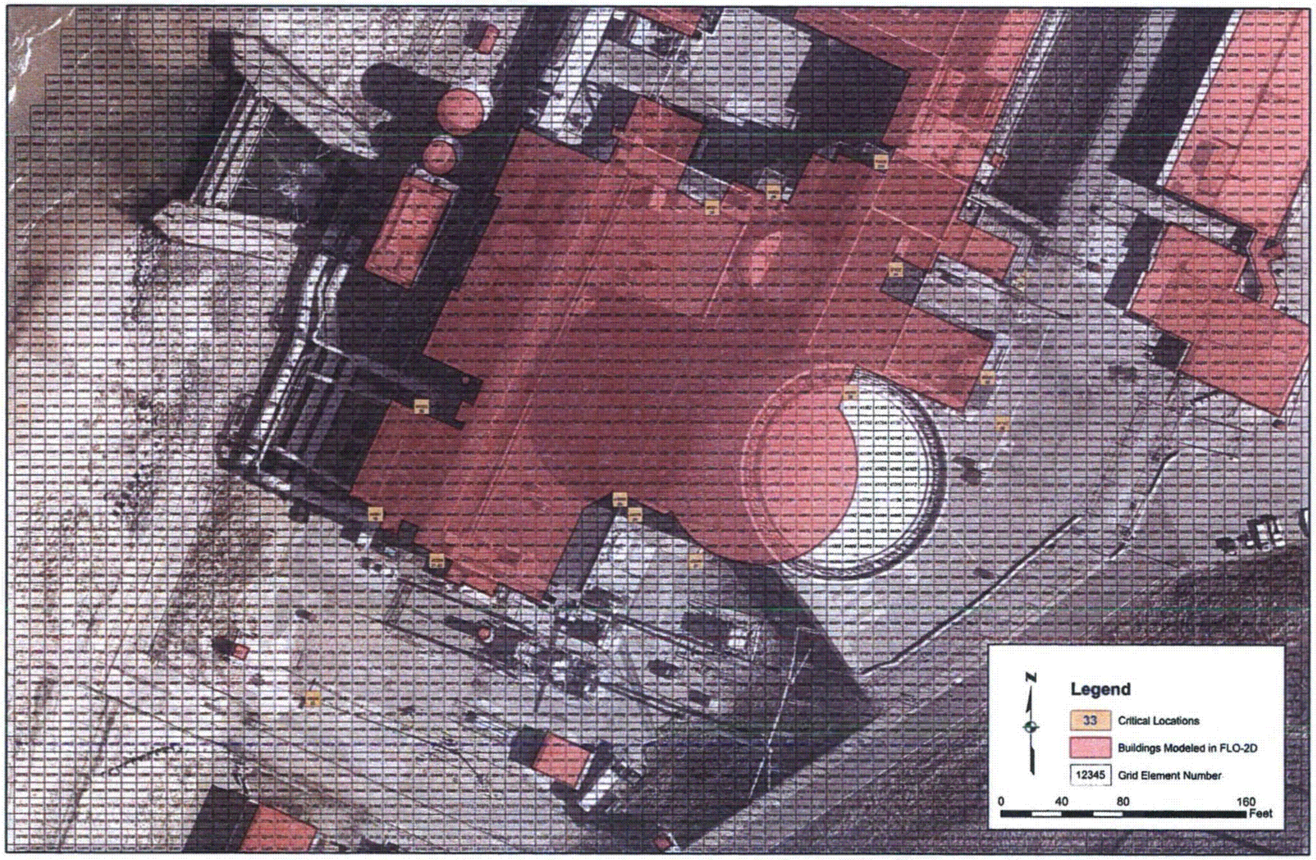
Note: The Manning's n-value for 'Open Ground, no Debris' was used for the 'Sandy Area' (e.g., dunes) as the reference used for Manning's n-value (FLO-2D 2013) does not include a land use category related to sand.

Table 3-3: Curve Number (CN) Values for Selected Land Cover Categories

Land Cover Category	NRCS Cover Type	CN
Paved / Concrete	Impervious	98
Building Roofs	Impervious	98
Dune Grass	Open space – Poor condition (grass cover<50%)	84
Short Grass	Open space – Poor condition (grass cover<50%)	84
Sandy Areas with No Cover	Impervious	98
Trees	Brush-brush-forbs-grass (poor)	68
Brush	Brush-brush-forbs-grass (poor)	68
Open Ground	Impervious	98
Water	Impervious	98

Note: Initial values for ARC II were taken from Tables 9-1 and 9-5 of NRCS 2004a. The CN values in the table above for ARC III were taken from Table 10-1 of NRCS 2004b.

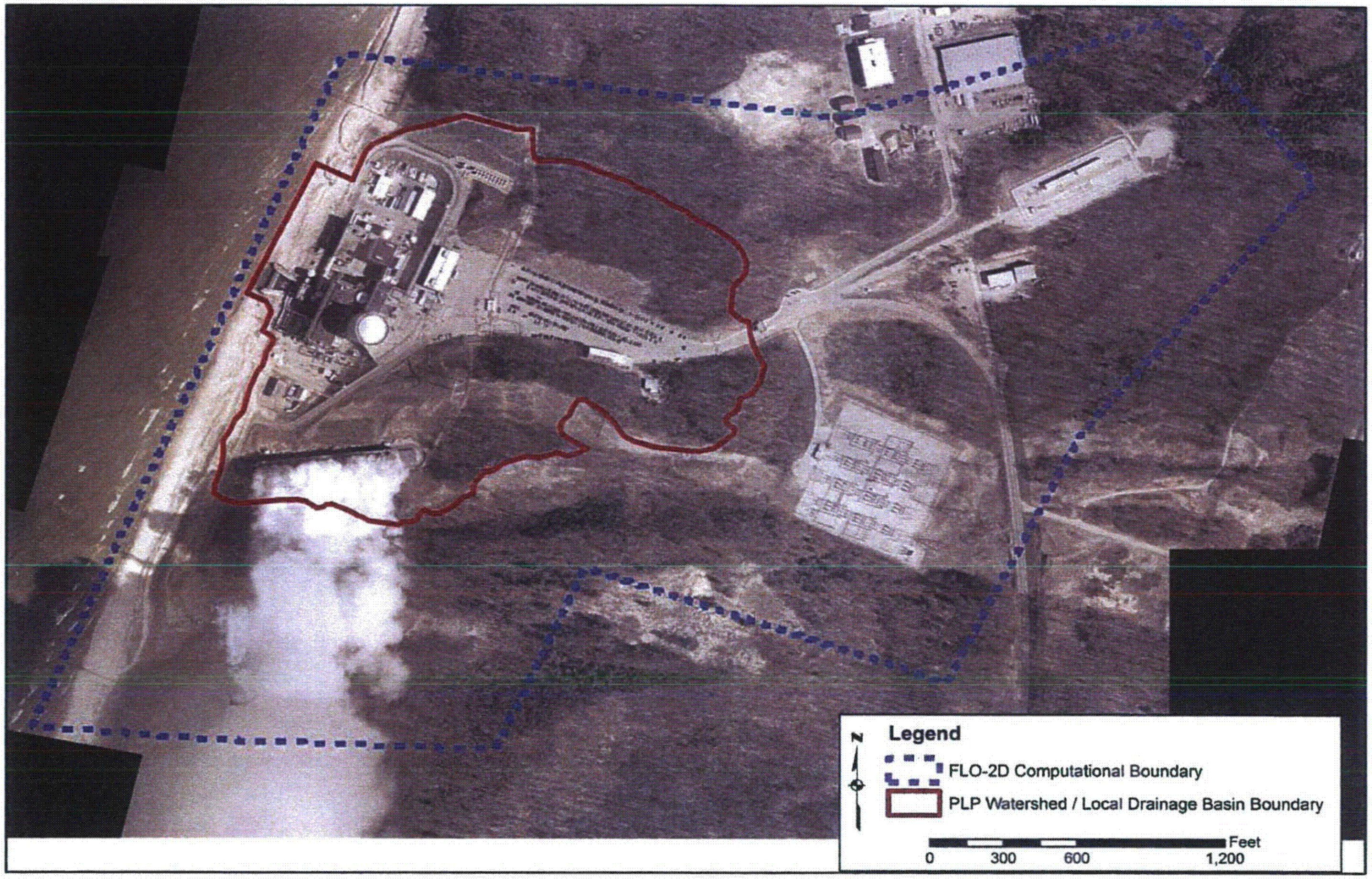
Figure 3-1: PLP Critical Locations



Basemap Source: High resolution orthoimagery (AREVA 2014c).

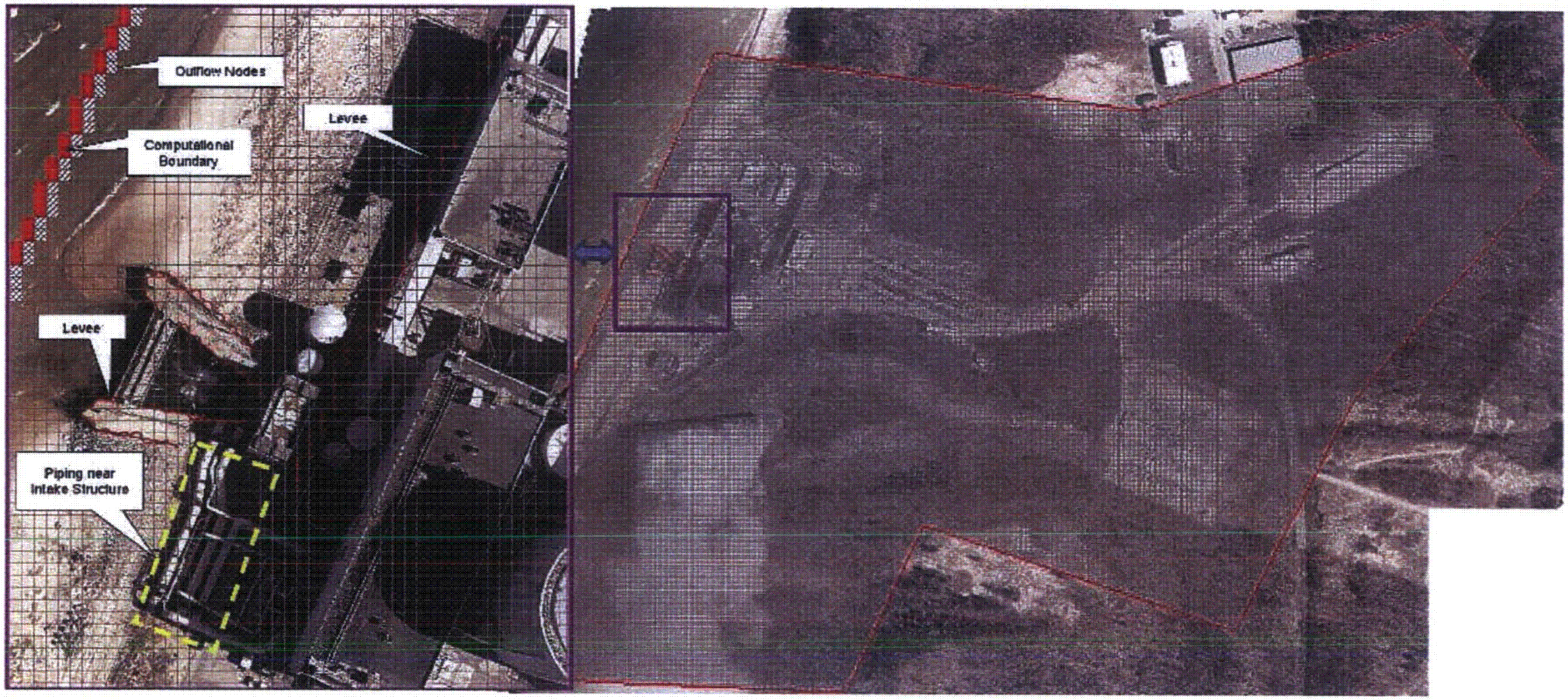
Note: A larger version of this figure is available in Appendix A. Other locations identified for the purpose of obtaining additional site topographic survey information (Location Nos. 1 to 18) were not identified as critical locations and therefore, were not included. Additionally, Location Nos. 24, 30, 31, and 32 were also not identified as critical locations and were not included in Figure 3-1. Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

Figure 3-2: FLO-2D Computational Boundary



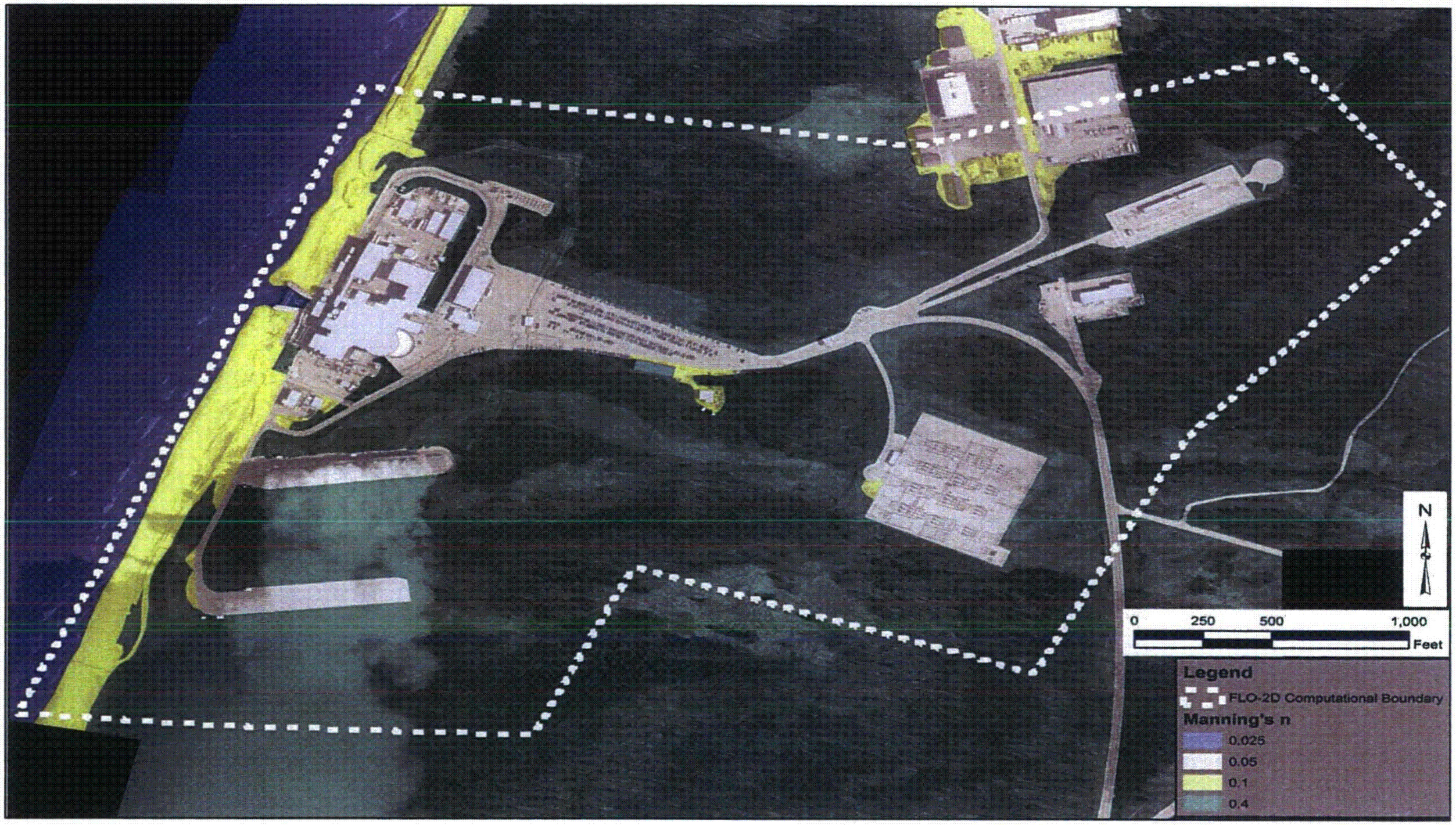
Basemap Source: High resolution orthoimagery (AREVA 2014c)

Figure 3-3: FLO-2D Modeled Site Features



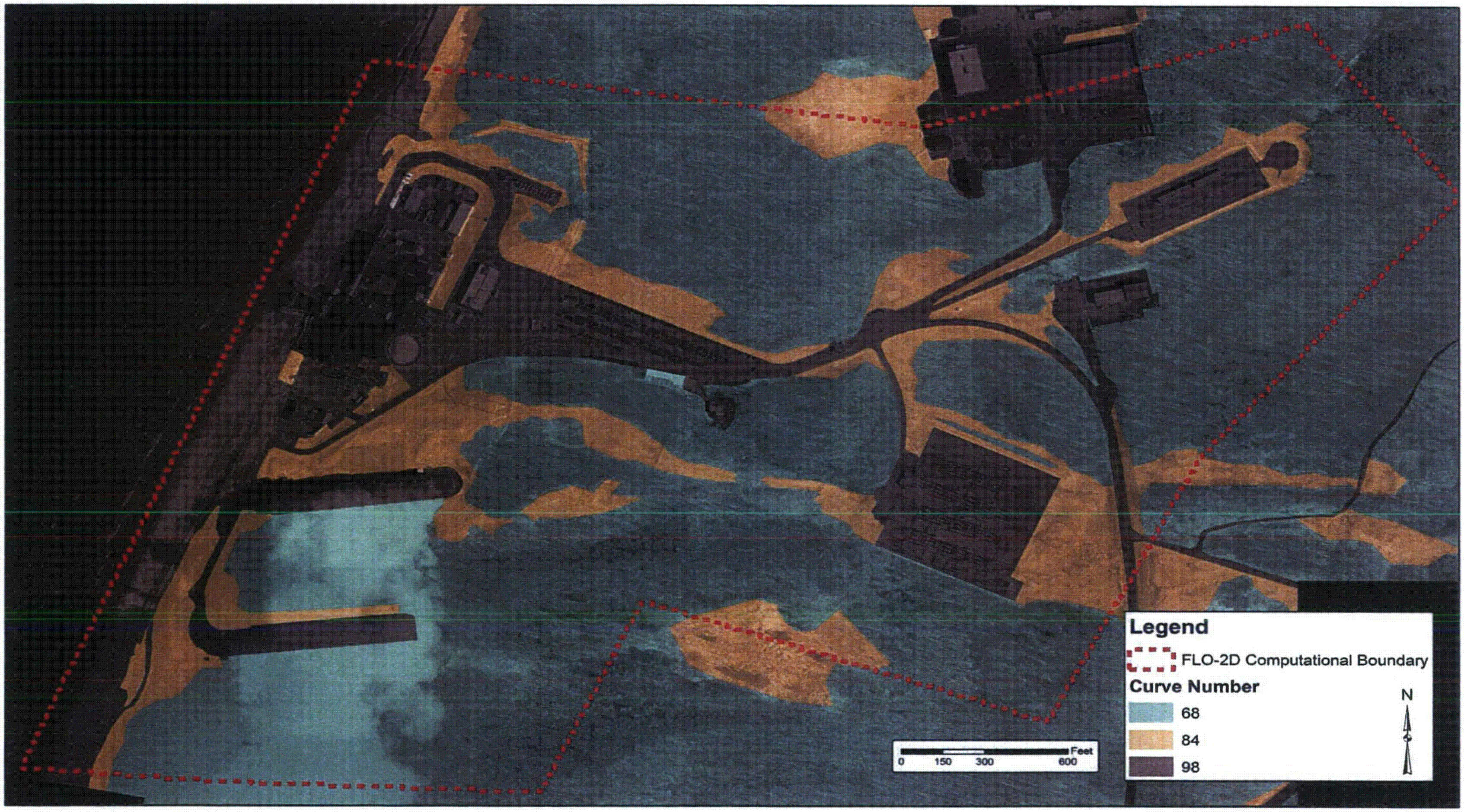
Basemap Source: High resolution orthoimagery (AREVA 2014c)

Figure 3-4: FLO-2D Manning's Coefficient Assignment



Basemap Source: High resolution orthoimagery (AREVA 2014c)

Figure 3-5: FLO-2D Curve Number Selections



Basemap Source: High resolution orthoimagery (AREVA 2014c)

Figure 3-6: LIP Hyetograph

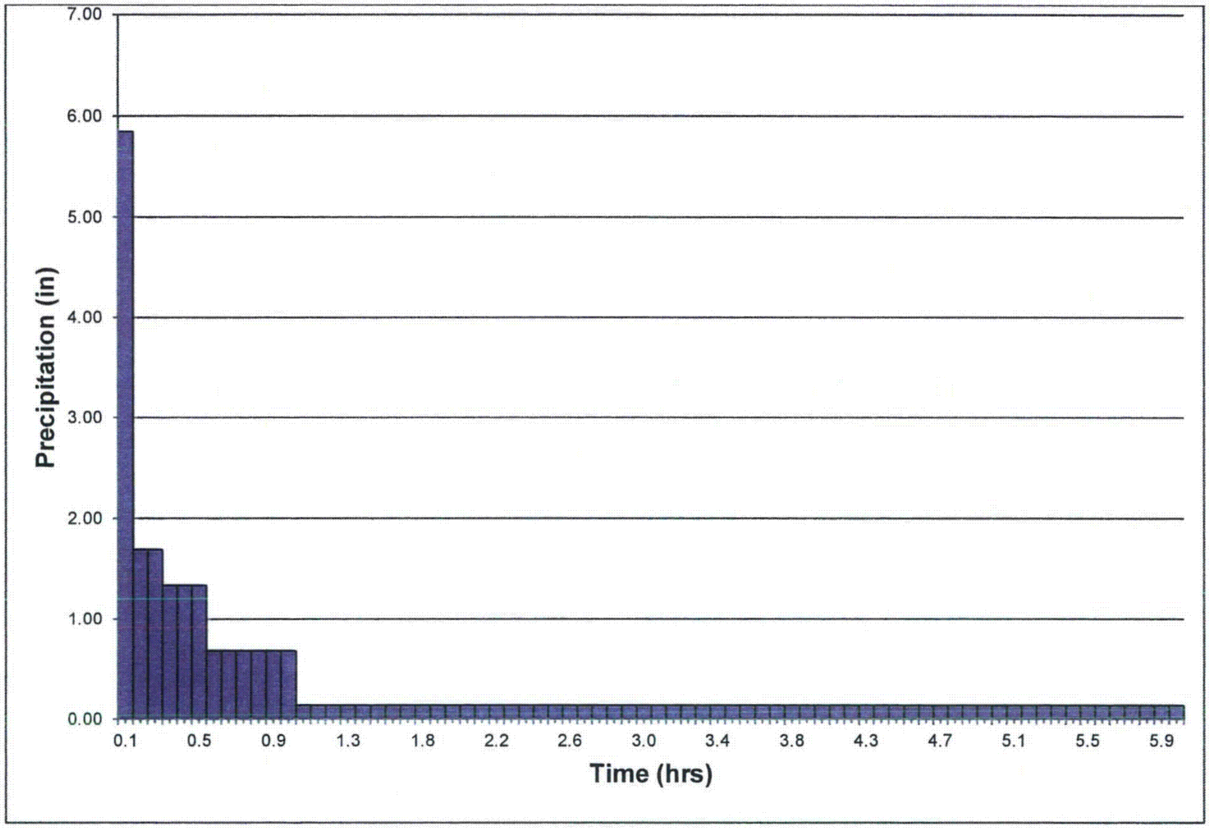


Figure 3-7: Supercritical Flow Regime

3.2 Flooding in Streams and Rivers

This section addresses the potential for flooding at PLP due to the Probable Maximum Flood (PMF) mechanism on rivers and streams. The PMF is the hypothetical flood (described by the peak discharge, total volume, and hydrograph shape) along a stream that is considered to be the most severe reasonably possible. The PMF is developed based on a comprehensive hydrometeorological application of the Probable Maximum Precipitation (PMP) and other hydrologic factors favorable for maximum flood runoff such as sequential storms and snowmelt (NRC 2011).

There are no perennial streams within the PLP watershed based on visual assessment of the drainage basin and information from the digital hydrography layer (Center for Share Solutions and Technology Partnerships 2014) displayed in Figure 3-8. Because there are no perennial streams or rivers within the watershed surrounding PLP, the facility is not subject to direct river flooding from a PMF-type flood. Flooding of local drainage courses at PLP was evaluated during the examination of the Local Intense Precipitation flood mechanism.

Brandywine Creek is the nearest significant perennial stream, but it is located in a separate watershed. The Brandywine Creek watershed is separated from PLP by a topographic divide at a minimum elevation of about 640 feet NAVD88 (Figure 3-8). The Brandywine Creek watershed drainage area is approximately 16 square-miles. The approximate vertical distance between Brandywine Creek channel and the low spot in the divide (Brandywine's watershed boundary adjacent to PLP) is 50 feet (640 feet at divide – 590 feet at stream channel), as shown in Figure 3-8. Thus, a PMF on Brandywine Creek will not affect PLP.

Therefore, the PMF on rivers and streams is judged to be not applicable at PLP.

3.2.1 References

Center for Share Solutions and Technology Partnerships 2014. Hydrography digital layer included in the Michigan Geographic Framework: State of Michigan, originated by the Center for Share Solutions and Technology Partnerships, Date accessed: January 14, 2015; Publication date: June 2014. (AREVA Document No. 38-9235194-000)

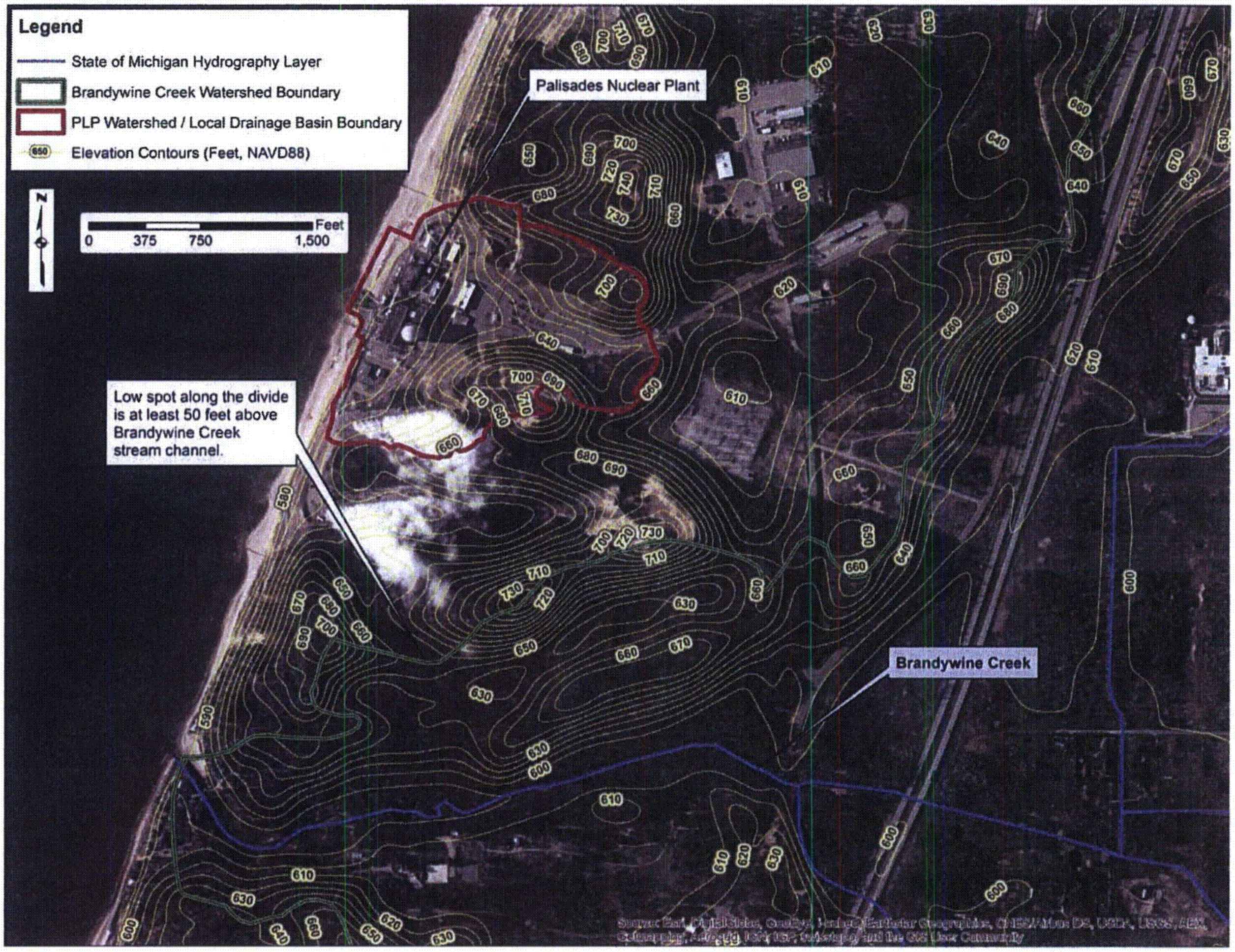
ESRI 2014. World Imagery, ESRI Map Service, Date accessed: January 15, 2015; Date modified: December 19, 2014. (AREVA Document No. 38-9235194-000)

NRC 2011. NUREG/CR-7046: Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, U.S. Nuclear Regulatory Commission, Springfield, VA, National Technical Information Service, 2011.

NRCS 2005. Digital hydrologic unit boundary layer Subwatershed (12-digit) 6th level for the State of Michigan, Michigan Natural Resources Conservation Service (NRCS), Date accessed: May 7, 2014; Publication date: 2005. (AREVA Document No. 38-9235194-000)

USGS 2012. Digital elevation contours for Racine E, Michigan, U.S. Geological Survey, Date accessed: May 7, 2014; Publication date: 2012. (AREVA Document No. 38-9235194-000)

Figure 3-8: PLP and Brandywine Creek Watersheds Location



Notes:

1) Brandywine Creek watershed boundary was extracted from a digital hydrologic unit boundary layer (NRCS 2005).
2) State of Michigan hydrograph layer is based on digital hydrography layer (Center for Share Solutions and Technology Partnerships 2014).
3) Elevation contours source is the U.S. Geological Survey (USGS 2012).
4) Base map is satellite imagery for the world map service from ESRI (ESRI 2014).

3.3 Dam Breaches and Failures

This section summarizes the assessment performed for flooding at PLP due to potential dam breaches and failures. For further details of the assessment, refer to AREVA Document No. 51-9226268-000 (AREVA 2014).

3.3.1 Methodology

As part of the HHA approach described in NUREG/CR-7046 (NRC 2011), assessments for potential dam-breach induced floods should consider the following steps, as applicable:

1. Investigate the failure of a subset of all upstream dams, while assuming that peak discharges of individual dam-failure induced floods reach the site at the same time.
2. Investigate the most severe cascading failure combination.

For the PLP site, Step 1 was performed. Based on the findings for Step 1, it was not necessary to perform Step 2 as discussed below.

3.3.2 Results

3.3.2.1 Water Controls for the Great Lakes

Water is continually flowing from the headwaters of Lake Superior via the St. Marys River, to Lake Huron and the remainder of the Great Lakes (Michigan, Erie and Ontario). Lake Huron and Lake Michigan are connected by the deep Straits of Mackinac and are considered to be one lake hydraulically, having a common water level (USACE 1999).

Outflow from Lake Superior is controlled near the twin cities of Sault Ste. Marie, Ontario and Michigan, by three hydropower plants, five navigation locks and a 16-gated controlled structure, called the Compensating Works. Prior to man-made controls, a rock ledge at the head of the St. Marys Rapids provided natural control for Lake Superior outflows. Evidence suggests that water levels on Lake Michigan and Lake Huron were five feet higher within the last 1,000 years, than they have been since the recording of lake levels in 1865 (USACE 1999).

There are five locations on the Great Lakes where water is diverted into, out of, or between lake basins. The Long Lac and Ogoki diversions divert water into Lake Superior. These diversions take water from the Hudson Bay watershed and augment natural flows for hydropower plants in the northern part of Lake Superior. The nearest man-made water diversion to PLP is the Lake Michigan diversion at Chicago, known as the Chicago Sanitary and Ship Canal. The Chicago Sanitary and Ship Canal links Lake Michigan to the Mississippi River. Lake water has been diverted at Chicago since 1848 for various reasons, including water supply, sewage disposal and navigation. The Welland Canal is a navigational waterway between Lakes Erie and Ontario that allows ships to bypass the Niagara River's falls and rapids. This diversion also provides water for hydropower generation. The smallest of the five diversions is the New York State Barge Canal which draws water from the Niagara River and returns diverted water to Lake Ontario. The impact on the Great Lakes from the five man-made diversions has been determined to be insignificant compared to natural forces. For example, it has been determined that there is no measurable effect on the water level of Lakes Michigan and Huron from the cumulative impacts of all five diversions (USACE 1999).

3.3.2.2 Failure of Locks and Diversions

Considering that there are no dams, per se, on the Great Lakes and that the water level in Lake Michigan depends in part on discharge from Lake Superior through a lock system, it was conservatively assumed that the entire lock system between Lake Superior and Lakes Michigan and Huron fails along with the five man-made water diversion structures, simultaneously contributing to a rise in the level of Lake Michigan at the PLP site. Based on the presumed lake levels within the past 1,000 years (i.e., pre man-made lake controls), if the Sault Ste. Marie Locks on Lake Superior fail, along with the Chicago Sanitary and Ship Canal and the other four diversions on the Great Lakes, it was assumed that the level of Lake Michigan would rise by five feet. An increase of five feet would also be indicative of the level of Lake Michigan before dams were built on rivers feeding into the Great Lakes. However, since PLP SSCs important to safety are protected to elevation 594.4 feet MSL from lake flooding (PLP 2012, Section 5.4.1.2), adding five feet to the maximum recorded lake level of 583.2 feet MSL yields 588.2 feet MSL, which is 6.2 feet below the PLP flood protected elevation. Therefore, the hypothetical failure of upstream locks and man-made diversions on the Great Lakes would not impact components important to safety at PLP. Hence, Step 2 of the HHA, a cascading failure combination, was not performed.

3.3.3 Conclusions

Based on conservative assumptions, potential breaches of the upstream locks and water diversion structures on the Great Lakes would not impact SSCs important to safety at PLP considering the following:

- The failure of the five, man-made water diversion structures on the Great Lakes have an insignificant impact on the water level of Lake Michigan.
- If the entire Sault Ste. Marie lock system fails and the water level of Lake Michigan instantly increases by five feet to its presumed natural level within the last 1,000 years (i.e., pre man-made alterations on the Great Lakes and lake tributaries), and the five feet is added to the highest lake level recorded within the past 54 years (i.e., based on years 1960 through 2014 (IDNR 2014)), a margin of 6.2 feet would exist between the lake's water level and the plant's flood protection elevation.

3.3.4 References

AREVA 2014. AREVA Document No. 51-9226268-000, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Screening for Dam Failures, August 2014.

INDR 2014. Indiana Department of Natural Resources, Lake Level Fluctuations, Lake Michigan Lake Level Graph 1960-2014; Available at: http://www.in.gov/dnr/water/3661.htm; Accessed May 27, 2014. (See AREVA Document No. 51-9226268-000)

PLP 2012. Palisades Nuclear Plant Final Safety Analysis Report (FSAR), Revision 30, September 4, 2012. (AREVA Document No. 38-9223712-000)

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

USACE 1999. Living with the Lakes: Understanding and Adapting to Great Lakes Water Level Changes, U.S. Army Corps of Engineers Detroit District and Great Lakes Commission, 1999. (See AREVA Document No. 51-9226268-000)

3.4 Storm Surge

Storm surges are defined as rises in offshore water elevations caused principally by the shear force of winds acting on the water surfaces (NRC 2011, Section 3.5). Storm surges can be caused by a variety of meteorological events, including tropical cyclones, extra-tropical storms, or moving squall lines.

This section summarizes the Probable Maximum Storm Surge (PMSS) evaluation. This evaluation was performed in two calculations. AREVA Calculation No. 32-9226959-000 (AREVA 2014a) assessed the Probable Maximum Wind Storm (PMWS) which was used as an input into the storm surge analysis. AREVA Calculation No. 32-9226962-000 (AREVA 2014b) developed the stillwater surface elevation at the site resulting from the PMSS.

3.4.1 Methodology

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC 2011, Section 2) was used to calculate the PMSS at PLP. The ADvanced CIRCulation model version 50.99.10 (ADCIRC) developed by the U.S. Army Corps of Engineers (USACE) (Luettich et al., 1992) was used to conservatively calculate the PMSS at PLP using the PMWS wind velocity and direction as inputs. The PMWS is defined as the hypothetical extra-tropical cyclone that may result from the most severe combination of meteorological storm parameters that is considered reasonably possible in the region involved (ANS 1992). Tropical cyclones and moving squall lines were also assessed as potential controlling meteorological events for input into the PMSS analysis.

The methodology to determine the design storm surge at PLP includes the following steps, consistent with guidance in ANSI/ANS-2.8-1992, American National Standard for Determining Design Basis Flooding at Nuclear Reactor Sites (ANS 1992) and the HHA approach:

- Selection of the design storm type (e.g., moving squall line, extra-tropical storm, etc.) to identify the past events that have resulted in the largest storm surges near PLP. Past events were identified by review of the statistical analysis of available one-hour and six-minute water level data conducted by the USACE (USACE 2012a) and other direct data sources to: a) eliminate long term water level fluctuations; b) identify the short term water level fluctuations; and c) identify the historical storm and storm type resulting in the highest recorded wind speeds and storm surges. The USACE analysis used National Oceanic and Atmospheric Administration (NOAA) hourly and 6-minute water level data (NOAA 2014e and NOAA 2014d) available through 2010. The USACE study was supplemented during the current calculation process with the most recent available 6-minute water level data (up to 2014).
- Development of the PMWS meteorological parameters by modification of the historical storm parameters of the selected design storm in accordance with ANSI/ANS-2.8-1992.
- Development of the antecedent water level by comparing the maximum controlled water level elevation of Lake Michigan to the 100-year return period water level, and selecting the lesser water elevation per ANSI/ANS-2.8-1992.
- Calculation of the PMSS still water elevation using ADCIRC.

3.4.2 Results

3.4.2.1 Selection of Design Storm

Consistent with NUREG/CR-7046 and ANSI/ANS-2.8-1992, identification of the controlling historic storm for development of the PMWS parameters was based on review and analysis of the USACE Great Lakes Study (USACE 2012a), NOAA National Climatic Data Center (NCDC) water level data from the closest NOAA Tides and Currents station (NOAA 2014d and NOAA 2014e) and the NOAA National Hurricane Center data (HURDAT) (NOAA 2014b). The storm types evaluated included tropical cyclones (hurricanes), extra-tropical cyclones, and moving squall lines.

Lake Michigan is about 300 miles long, 50 to 80 miles wide, has an average depth of 300 feet , and is oriented with its long dimension trending in an approximately north to south direction (USACE 2012a). PLP is located along the southeastern lake shoreline as shown in Figure 3-9.

Water-level fluctuations due to storm surges on the Great Lakes are generally caused by one of several types of strong storms, including:

- Tropical systems that move north from the Gulf Region and Mid-Atlantic;
- Non-convective storms (extra-tropical cyclones) that originate in Canada and move to the east through the lakes region or originate in the southern and central Rockies and move east through the lakes region; and
- Convective storms or thunderstorm frontal passages, including moving squall lines (FEMA 2014).

3.4.2.1.1 Tropical Cyclones (Hurricanes, Tropical Storms, Tropical Depressions)

The HURDAT record (NOAA 2014b) and best track data (NOAA 2014c) were evaluated relative to tropical cyclones (hurricanes, tropical storms) originating in the southern latitudes and moving into the Great Lakes region. No storms which remained at hurricane strength have tracked within 150 miles of PLP (NOAA 2014c) and according to NOAA, "No actual hurricane has ever been observed in Michigan under the true definition of a hurricane" (NOAA 2004). Nineteen tropical cyclones that decayed to below hurricane strength tracked within 150 miles of PLP (NOAA 2014c). These storms are summarized on Table 3-4, which presents the storm name, date, maximum sustained wind speed (1-minute, 10 meter) and central pressure at the storm track latitudes and longitudes near the lake.

Storm tracks were typically in an approximate southwest to northeast direction. Ten of the nineteen storms had transitioned into an extra-tropical state before reaching Lake Michigan. The other nine remained tropical depressions or tropical storms. Maximum sustained wind speeds ranged up to 50 mph, with the highest wind speed recorded for remnants of the September 1900 hurricane and Hurricane Ike in 2008. The lowest reported central pressure was 985 mbars for the remnants of the September 1941 hurricane.

Based on the above analysis it was found that tropical cyclones experience significant decay before reaching the area and therefore tropical cyclones were not considered in the development of the PMWS.

3.4.2.1.2 Extra-Tropical Cyclones

Most of the strong storms in the Great Lakes are extra-tropical, low-pressure non-convective systems. These non-convective, extra-tropical storms typically originate in Canada or the southern or central Rocky Mountains (Angel 1996). Winds in low-pressure systems spin counter-clockwise, while high-pressure systems spin in a clockwise direction. Winds on the eastern side or leading edge of a low-pressure system are typically in the northerly direction, while winds on the eastern side of a high-pressure system are in the southerly direction (FEMA 2014). High winds and large atmospheric pressure variations are commonly associated with these storm events, and they

can cause elevated water levels, or storm surge, along the lake shoreline (FEMA 2014). The winter season is characterized by the most severe, extra-tropical cyclones that occur from November to April (USACE 2012a). The principal, most frequent storm track of winter storms in the Lake Michigan-Huron region is from the southwest to the northeast moving from Colorado through Michigan (Angel 1996).

In accordance with the procedures presented in ANSI/ANS-2.8-1992, an analysis of available data for historic synoptic cyclonic wind storms was performed. The top surges in the vicinity of PLP (specifically, the Holland, Michigan water level gage), during the period of 1959 to 2010, were identified in the USACE study (USACE 2012a) using 6-minute and hourly water level data. To identify the top surge events, short-term water level variations (i.e., storm surges) were analyzed. The extreme surge values were identified by the USACE using the Peaks over Threshold (POT) method (USACE 2012a). POT values were obtained by selecting water level deviations from a 30-day moving average of the measured water level time series (USACE 2012a). The USACE used a Gaussian smoothing algorithm coupled with the POT method.

The Great Lakes Study conducted by the USACE used gage data up to January 2010 (USACE 2012a). This period of record was supplemented with 6-minute water level data at the Holland, Michigan gage (NOAA 2014d) in order to identify surges that occurred in the period of 2010 to July 2014 (NOAA 2014d). Consistent with the USACE methodology, a 30-day moving average was used as a base water level and the difference between each 6-minute measurement and the base water level was ranked for the recent period of record.

In addition to the Holland, Michigan gage (which is located closest to PLP), data from other gages was also reviewed. Lake Michigan has nine long-term NOAA water level gages. However, there is little correlation between peak water levels among these gages and no correlations among gages that are separated by greater than half the lake length (USACE 2012a). Therefore, the water level data at the Holland, Michigan gage is considered the most applicable to assessing storm surge in the vicinity of PLP, shown in Figure 3-9.

The top 20 storm surge events (for the period of 1959 through 2014), based on water level data at the Holland, Michigan gage, are presented in Table 3-5. The 20 storms identified in Table 3-5 generally occurred between November to April. These storm surges do not correlate with the tropical systems in Table 3-4 and are inferred to be primarily associated with non-convective, extra-tropical storms (USACE 2012a). The most frequent storm track of these storms is from southwest to northeast as shown in Figure 3-10. As shown on Table 3-5, the highest surge identified at the Holland, Michigan gage was 2.01 feet on December 4, 1990 (USACE 2012a). This was a cyclonic storm that moved from Oklahoma northeast over Lake Michigan toward Quebec (USACE 2012a). On December 4th, sustained winds of 30 to 40 mph began blowing across the widest part of Lake Michigan from the northwest (USACE 2012a). The storm surge peaked at 6:00 GMT as shown in the hourly water level hydrograph at Holland, Michigan (NOAA 2014e) for the December 4, 1990 storm, shown in Figure 3-11.

The review of the water level, wind and wave data indicate that the largest observed storm surge and wave events are primarily associated with extra-tropical storm events. The December 4, 1990 storm resulted in the highest storm surge, the second highest recorded sustained wind speed and the third highest wave event. This storm was used as a representative historic storm for the development of the PMWS parameters for extra-tropical cyclones.

3.4.2.1.3 Moving Squall Lines

Squall lines are also a consideration in the Great Lakes region, particularly along the shores of Lake Michigan. ANSI/ANS-2.8-1992 states that "A moving squall line should be considered for the locations along Lake Michigan where significant surges have been observed because of such a meteorological event" and notes the possible occurrence of squall lines within the other Great Lakes (ANS 1992). The current design basis storm at PLP is a moving squall line as referenced in the PLP FSAR (PLP 2012).

Moving squall lines are defined as "a line or narrow band of active thunderstorms having a pressure jump with the cold front providing the initial piston-like impetus, and mature instability line that is located as the warm sector of a wave cyclone about 50 to 200 miles in advance of the cold front, usually oriented roughly parallel to the cold front and moving in about the same direction and speed as the cold front" (ANS 1992). Thunderstorms (i.e., convective systems) are common in the summer months in the vicinity of PLP. Three severe, historic storm surge events on Lake Michigan caused by moving squall lines are referenced in literature: June 26, 1954; August 3, 1960 and May 30-31, 1998 (Platzman et al., 1965, PLP 1982a and NOAA 2014g). The 1954 and 1960 events affected the Chicago area and resulted in elevated water levels from a surge wave that reflected off the southeastern shore of Lake Michigan (Platzman et al., 1965). The 1998 event moved across Lake Michigan with intense westerly winds which produced a reported storm surge near Muskegon, Michigan, about 60 miles north of PLP (NOAA 2014g). Therefore, storm surge caused by moving squall lines was evaluated and judged to be an additional potential design storm.

The parameters of moving squall lines were developed with reference to previous site-specific studies for PLP and general guidance by the American Nuclear Society studies, including:

1. The PLP design basis squall was modeled using a procedure described in "The Prediction of Surges in the Southern Basis of Lake Michigan" (PLP 1982b). This study used an idealized model of the pressure and wind distributions of a moving squall line as shown in Figure 3-12 (Platzman et al., 1965). The idealized squall line was modeled to move across the southern basin of Lake Michigan at a uniform speed and direction (Platzman et al., 1965).
2. Design parameters for moving squall lines at PLP were also developed in the "High Water Level Study, Palisades Plant" in 1982 (PLP 1982a). These parameters included a pressure gradient of 7.1 mbars (0.21 inches mercury) over an 11.5 mile (10 nautical miles) width (PLP 1982a). Maximum wind speeds of 52.9, 64.4, and 74.8 mph (46, 56 and 65 knots) and squall line propagation directions of 105, 120, 135 and 150 degrees were tested (PLP 1982a).
3. ANS subsequently developed probable maximum squall line parameters for the Great Lakes in 1976, which were adopted and included in ANSI/ANS-2.8-1992 (ANS 1992). These parameters included a pressure gradient of 8 mbars over 11.5 miles and a maximum wind of 75 mph (ANS 1992).

3.4.2.2 Development of Design Storm Parameters

3.4.2.2.1 Extra-Tropical Cyclone

In accordance with the procedures outlined in ANSI/ANS-2.8-1992, the December 4, 1990 storm, listed in Table 3-5, was selected as a representative extra-tropical storm event for use in developing the properties of the PMWS. Pressure maps for the storm were obtained from the NCDC (NOAA 2014f) and used as input into the PMWS model. The time of the maximum surge was 6:00 GMT on December 4, 1990 and was found using the hourly water level hydrograph at the Holland, Michigan gage (NOAA 2014e), shown in Figure 3-11. The pressure maps were used to calculate the isobars, distance between isobars, and wind angles affecting the lake and to establish a relationship between the pressure maps and a geographic coordinate system in order to determine storm speed.

PMWS Storm Track and Speed

The December 4, 1990 storm used for development of the PMWS approached the Great Lakes from the southwest and travelled in an approximately southwest to northeast direction. The storm track of the PMWS follows the primary tracks constructed from the historic extra-tropical storm tracks shown in Figure 3-13. The storm tracks for the December 2, 1985; January 4, 1982; February 15, 1967 and November 15, 1961 storms compared well with the PMWS storm track. The PMWS was translated along several straight-line tracks to represent the range

of historic storm tracks shown in Figure 3-13. The track was adjusted such that the storm center moved along a critical path until the maximum wind speed vectors were approximately in the direction of the major axis of the water body. The storm tracks were selected based on descriptive statistics of the storm bearings for the top 12 extra-tropical storm surge events at Holland, Michigan, shown in Figure 3-13 and Table 3-6. The critical storm track was determined by the track that resulted in the maximum surge height near PLP.

The translation speed calculated for the December 4, 1990 storm ranged from 17 to 50 mph with an average speed of 26 mph. The PMWS translational speed was maintained at a constant, steady state speed within the range of the historic December 4, 1990 storm speeds. The storm translation speeds were selected based on descriptive statistics of the translation speeds for the top 12 surge events at Holland, Michigan, shown in Table 3-6. The critical translation speed was determined by the translational speed that resulted in the maximum surge near PLP.

PMWS Wind Field

A spatially varying pressure and wind field was developed by dividing the PMWS into ten radials, see Figure 3-14. Additional radials in the western half of the storm were added to more accurately spatially resolve the winds that cause the surge at PLP. The PMWS isobar pattern was used to calculate the pressure, wind speed, and wind direction at points between each isobar for each radial.

The PMWS pressure, wind speed, and wind direction at points between each isobar for each radial was calculated using the methods presented in the USACE Coastal Engineering Manual (CEM) (Resio et al., 2008). Wind direction for each point was estimated from the orientation of the isobars. Wind directions were calculated to be at an angle of 10 degrees convergent across the isobars as specified by ANSI/ANS-2.8-1992 for the Great Lakes region (ANS 1992).

In order for the maximum wind speed to reach 100 mph, the wind field speeds were scaled up by a factor equal to the ratio of 100 mph to the maximum wind speed of the entire storm (ANS 1992). Based on a maximum calculated wind speed of 77 mph, the wind field speeds were scaled up by a constant value of 1.30. In order for the minimum pressure to reach 950 mbars, the minimum pressure of the storm, 998.9 mbars, was scaled down to 950 mbars using a reduction of 48.9 mbars for each pressure value. Figure 3-14 presents the results of the PMWS model analysis, including adjusted pressure, adjusted wind speed, and wind direction.

3.4.2.2.2 Moving Squall Line

Storm surge caused by moving squall lines was evaluated and judged to be an additional potential design storm. The parameters of moving squall lines have been developed in ANSI/ANS-2.8-1992 based on the probable maximum positive excess pressure that is reasonably possible (ANS 1992). The parameters for a synoptic model of moving squall line are presented in ANSI/ANS-2.8-1992 (ANS 1992):

- A maximum pressure jump of 8 mbars over a width of 11.5 miles (10 nautical miles).
- A maximum wind of 75 mph (65 knots) (ANS 1992).
- The squall line speed should be considered equal to the resonant speed of the surge (ANS 1992).
- A range of squall line speeds, critical line tracks and angles of the squall line should be tested to determine the controlling parameters.

The wind field for the moving squall line was based on the rectilinear configuration developed by Platzman and shown in Figure 3-14 (Platzman et al., 1965). The maximum pressure jump was set to 8 mbars over 11.5 miles with a maximum wind speed of 75 mph. The squall line was assumed to move with a constant speed and direction. A range of squall line speeds and angles were selected to determine the "worst case" scenario. The

storm tracks were selected based on the "High Water Level Study" (PLP 1982a) and aligning the wind field at a critical angle approaching PLP.

3.4.2.3 Development of the Antecedent Water Level

As defined in Appendix H of NUREG/CR-7046 (NRC 2011) for enclosed bodies of water, the lesser of the 100-year level or the maximum controlled water level should be used for the evaluation of flood levels from storm surges.

The 100-year water level was calculated to be 582.6 feet IGLD85 (583.4 feet NGVD29) using the log-Pearson Type III distribution of monthly mean water level data at Holland, Michigan (NOAA 2014a). The maximum controlled water level was determined to be naturally regulated by the St. Clair river based on review of USACE and International Joint Commission reports (USACE 2000 and IJC 2013). The maximum observed water level of Lake Michigan for the years from 1918 to 2012 was 582.4 feet IGLD85 (583.1 feet NGVD29) (USACE 2012c). The calculated 100-year water level is greater than the maximum observed water level in Lake Michigan for the years from 1918 to 2012. Because the two values are so close, the 100-year water level of 582.6 feet IGLD85 (583.4 feet NGVD29) was conservatively used as the antecedent water level for PMSS calculations.

3.4.2.4 ADCIRC Storm Surge Model

Storm surge simulations were performed for the candidate extra-tropical storm and moving squall lines identified in the PMWS calculation (AREVA 2014a) using the ADCIRC model.

ADCIRC uses a non-structured, triangulated model mesh with variable mesh resolution. The ADCIRC mesh and boundary information used in this analysis were developed by the USACE and Federal Emergency Management Agency (FEMA) for the current (on-going) FEMA Region V Flood Coastal Analysis and Mapping project. The USACE ADCIRC mesh is a verified mesh that has been used to simulate storm surge on Lake Michigan for FEMA flood mapping purposes. Therefore, it has sufficient resolution to simulate storm surge near PLP.

The USACE initially validated the ADCIRC mesh by simulating the following severe storm events: December 1990, December 2009, May 1998, October 1993, November 1992, September 1989 and March 1985. The USACE compared simulated time series to measurements at NOAA gages around Lake Michigan. The USACE concluded that the model was able to capture the trend in surge response magnitude and duration at these gage locations. The USACE also statistically compared ADCIRC water level time series results to NOAA water levels at 10 gages. Statistical comparisons performed were calculation of bias and root mean square error (RMSE) at the 10 gages for 124 storms. In general, the bias was within ±0.05 meters (0.2 feet). The Holland gage in particular had no bias more than 0.12 meters (0.4 feet). In general, the RMSE was within 0.10 meters (0.3 feet) for the gages used in the analysis (USACE 2012b). The model performed well at simulating water levels over a large spatial extent with varying storm intensity, size and location. Therefore USACE concluded that "the model's ability to predict water level at many locations under various conditions provides a strong degree of confidence in the model to predict water levels at other locations around Lake Michigan as well." (USACE 2012b)

The ADCIRC model uses inputs related to forcing that represent the intensity of external influences (e.g., storm intensity, tides, etc.). Tides on the Great Lakes are on the order of 0.4 to 1.6 inches as reported by the Canadian Hydrologic/Hydrographic Service (USACE 2012a). Tidal fluctuations on Lake Michigan are therefore, not significant in comparison to water level variations and were not used as input to ADCIRC. For this calculation, ADCIRC was forced with the following inputs:

- Meteorological forcing from the extra-tropical storm wind, pressure and ice fields (Extra-Tropical Case)
- Meteorological forcing from the moving squall line wind and pressure fields (Moving Squall Line Case)

3.4.2.4.1 Extra Tropical Cyclone

A range of storm tracks and translational speeds selected in Section 3.4.2.2.1 were simulated using the ADCIRC model to determine the sensitivity of storm surge to track and speed and to identify the critical combination of track and speed resulting in the largest surge height near PLP. The ice concentration for the entire lake was conservatively set at 0.5, the concentration that results in the greatest wind drag. Seven extra-tropical storm surge simulations were performed. The most conservative PMWS track and translational speed were determined using ADCIRC to simulate storm surge at PLP. Therefore, the PMWS track represents a conservative track for the generation of winds on Lake Michigan which is deemed to have a reasonable probability of occurrence.

Based on the sensitivity analysis of storm surge to storm track and translational speed, the largest extra-tropical storm surge height near PLP was 9.7 feet, which was the result of storm traveling at 17 mph and crossing southern Lake Michigan at a bearing of 45° east from north. The surge hydrograph near PLP for the critical extra-tropical storm is shown in Figure 3-15. The surge stillwater elevation (surge on top of the antecedent water level) for the critical extra-tropical storm was 592.3 feet IGLD85 (593.1 feet NGVD29).

3.4.2.4.2 Moving Squall Line

The critical storm track was determined by simulating a range of four storm track bearings and three translational speeds using ADCIRC to determine the storm track and translation speed that resulted in the maximum surge near PLP. Seven moving squall line storm surge simulations were performed. Based on the sensitivity analysis of storm surge to storm track bearing and translational speed, the largest moving squall line surge was 3.6 feet, which was the result of storm traveling at 65 mph and crossing Lake Michigan from a bearing of 45° west from north (from the northwest). The surge stillwater elevation (surge on top of the antecedent water level) for the critical moving squall line was 586.2 feet IGLD85 (587.0 feet NGVD29).

3.4.2.4.3 Identification of Controlling PMWS and Resulting PMSS

The largest storm surge stillwater heights for the candidate extra-tropical storm and moving squall line are shown in Table 3-7. The extra-tropical storm type resulted in the largest storm surge stillwater height of 9.7 feet near PLP and is therefore, the controlling storm type. This storm surge elevation is representative of the PMWS defined as a slow-moving (17 mph) extra-tropical cyclone bearing in a northeast direction such that the highest velocity winds are coincident with the long axis of Lake Michigan. The PMSS stillwater elevation is 592.3 feet IGLD85 (593.1 feet NGVD29).

3.4.3 Conclusions

Based on the analyses of the PMWS and PMSS for PLP, the following conclusions are reached:

- The controlling storm type is an extra-tropical storm.
- Per ANSI/ANS-2.8-1992, the antecedent water level is the 100-year water elevation, which is 583.4 feet (NGVD29).
- The largest surge near PLP resulted from the PMWS moving at 17 mph along a track bearing 45° east of north. The peak sustained wind speed near PLP was 87 mph coincident with the critical wind direction coming from 45° west of north.
- The predicted PMSS height is 9.7 feet.
- The predicted PMSS stillwater elevation is 593.1 feet (NGVD29).

Uncertainty and conservatism were considered in the calculation as per Section 5.4 of NUREG/CR-7046, as follows. The PMWS parameters, which are the basis for the ADCIRC model used to calculate the PMSS, were adjusted to provide the most adverse conditions. The adjustments included:

- The PMWS storm track was simulated as straight-line and the translation speed of the storm was set to a constant 17 mph (which was the minimum recorded storm speed) to increase the effect of the pressure gradients and resulting wind speeds.
- The predicted peak wind speed for the PMWS was increased to reflect a maximum over-water wind speed of 100 mph (as defined in ANSI/ANS-2.8-1992). This resulted in a 21% increase in the calculated peak wind speed determined for the synthetic storm used to evaluate the PMWS.

In analyzing the PMSS, a range of storm tracks and translational speeds were selected and simulated using the ADCIRC model to determine the sensitivity of storm surge to track and speed and to identify the critical combination of track and speed resulting in the largest surge height near PLP.

3.4.4 References

Angel 1996. Cyclone Climatology of the Great Lakes, J. Angel, Midwestern Climate Center, Miscellaneous Publication 172, February 1996. (See AREVA Document No. 32-9226959-000)

ANS 1992. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8 – 1992).

AREVA 2014a. AREVA Document No. 32-9226959-000, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Probable Maximum Wind Storm, 2014.

AREVA 2014b. AREVA Document No. 32-9226962-000, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Probable Maximum Storm Surge and Seiche, 2014.

ESRI 2014a. USA Topo Maps, ESRI Map Service, Date modified: March 4, 2014, Date accessed: August 1, 2014. (See AREVA Document No. 32-9226959-000)

ESRI 2014b. World Topographic Map, ESRI Map Service, Date modified: June 26, 2014, Date accessed: August 1, 2014. (See AREVA Document No. 32-9226959-000)

FEMA 2014. Guidelines and Standards for Flood Hazard Mapping Partners: FEMA Great Lakes Coastal Guidelines, Appendix D.3 Update, Federal Emergency Management Agency, January, 2014.

IJC 2013. International Joint commission's Advice to Governments on the Recommendations of the International Upper Great Lakes Study, International Joint Commission, April 15, 2013. (See AREVA Document No. 32-9226959-000)

Luettich et al., 1992. ADCIRC: An Advanced Three-Dimensional Circulation Model for Shelves Coasts and Estuaries: Report 1 Theory and Methodology of ADCIRC-2DDI and ADCIRC-3DL, Dredging Research Program Technical Report, DRP-92-6, R.A. Luettich Jr., J.J. Westerink and N.W. Scheffner, U.S., Army Engineers Waterways Experiment Station, November 1992. (See AREVA Document No. 32-9226962-000)

NOAA 2004. "Hurricanes in Michigan???", William R. Deedler, National Oceanic and Atmospheric Administration, National Weather Service Detroit/Pontiac, MI, http://www.crh.noaa.gov/dtx/stories/dtxcane.php, August 18, 2004. (AREVA Document No. 32-9226959-000)

NOAA 2014a. NOAA/NOS/CO-OPS Verified Monthly Means at 9087031, Holland MI, Tides & Currents, National Oceanic and Atmospheric Administration, http://tidesandcurrents.noaa.gov/stationhome.html?id=9087031, Date accessed: June 10, 2014, Date modified: June 10, 2014. (AREVA Document No. 32-9226962-000)

NOAA 2014b. Revised Atlantic Hurricane Database (HURDAT2), Chris Landsea, James Franklin and Jack Beven, Atlantic Oceanographic and Meteorological Laboratory, National Oceanic Atmospheric Administration, date modified: April 2014, date accessed: June 9, 2014. (See AREVA Document No. 32-9226959-000)

NOAA 2014c. International Best Track Archive for Climate Stewardship, Point and Line Shapefiles, National Climatic Data Center, National Oceanic and Atmospheric Administration, date accessed: June 9, 2014. (AREVA Document No. 32-9226959-000)

NOAA 2014d. NOAA/NOS/CO-OPS Verified 6-Minute Water Level at 9087031, Holland MI, Tides & Currents, National Oceanic and Atmospheric Administration, http://tidesandcurrents.noaa.gov/stationhome.html?id=9087031, Date accessed: August 5, 2014, Date modified: August 5, 2014. (AREVA Document No. 32-9226959-000, Appendix A and Appendix G)

NOAA 2014e. NOAA/NOS/CO-OPS Verified Hourly Height Water Level at 9087031, Holland MI", Tides & Currents, National Oceanic and Atmospheric Administration, http://tidesandcurrents.noaa.gov/stationhome.html?id=9087031, Date accessed: July 24, 2014, Date modified: June 24, 2014. (AREVA Document No. 32-9226959-000, Appendix A)

NOAA 2014f. U.S. Daily Weather Maps", National Oceanic and Atmospheric Administration, National Weather Service, various dates, http://www.lib.noaa.gov/collections/imgdocmaps/daily_weather_maps.html, date accessed: July 22, 2014. (AREVA Document No. 32-9226959-000, Appendix B)

NOAA 2014g. The Southern Great Lakes Derecho of 1998, National Oceanic and Atmospheric Administration, http://www.spc.noaa.gov/misc/AbtDerechos/casepages/may30-311998page.htm; Date accessed: July 25, 2014. (AREVA Document No. 32-9226959-000)

NRC 2011. NUREG/CR-7046: Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, U.S. Nuclear Regulatory Commission, Springfield, VA, National Technical Information Service, 2011.

Platzman et al., 1965. The Prediction of Surges in the Southern Basin of Lake Michigan, Monthly Weather Review, Volume 93, Number 5, George W. Platzman, Shirley M. Irish and Lawrence A. Hughes, May 1965. (See AREVA Document No. 32-9226959-000)

PLP 1982a. High Water Level Study, Palisades Plant, For Consumers Power Company, R. M. Noble & Associates, March 12, 1982. (AREVA Document No. 38-9223712-000)

PLP 1982b. Palisades – SEP Topic II-3.A, Hydrologic Description, II-3.B, Flooding Potential and Protection Requirements, Letter from U.S. Nuclear Regulatory Commission to Consumers Power Company, Docket No. 50-255, October 11, 1982, Palisades Nuclear Plant Updated Final Safety Analysis Report, Table 1-3, Page 1 of 12. (AREVA Document No. 38-9223712-000)

PLP 2012. PLP Final Safety Analysis Report, Revision 30, 2012. (AREVA Document No. 38-9223712-000)

Resio et al., 2008. Coastal Engineering Manual, Part II, Hydrodynamics, Chapter II-2 Meteorology and Wave Climate, Engineering Manual 1110-2-1100, U.S. Army Corps of Engineers, D. Resio, S. Bratos and E. Vincent, 2008.

USACE 2000. Living with the Lakes: Understanding and Adapting to Great Lakes Water Level Changes, U.S. Army Corps of Engineers and Great Lakes Commission, February, 2000. (See AREVA Document No. 32-9226959-000)

USACE 2012a. Wave Height and Water Level Variability on Lakes Michigan and St Clair, ERDC/CHL TR-12-23, Great Lakes Coastal Flood Study, 2012 Federal Inter-Agency Initiative, U.S. Army Corps of Engineers, Engineer Research and Development Center Coastal and Hydraulic Laboratory, October 2012. (See AREVA Document No. 32-9226959-000, Appendix E)

USACE 2012b. Lake Michigan Storm: Wave and Water Level Modeling, ERDC/CHL TR-12-26, Great Lakes Coastal Flood Study, 2012 Federal Inter-Agency Initiative, U.S. Army Corps of Engineers, Engineer Research and Development Center Coastal and Hydraulic Laboratory, November 2012. (See AREVA Document No. 32-9226962-000, Appendix A)

USACE 2012c. Final 2012 and Long-Term (1918-2012) Mean, Max & Min Water Levels (Based on Gage Networks), U.S. Army Corps of Engineers, Detroit District, June 17, 2013. (See AREVA Document No. 32-9226962-000)

Table 3-4: HURDAT Hurricane Data for Tropical Systems in the Vicinity of PLP

Name	Date	Maximum Sustained Wind Speed (miles/hour)	Central Pressure (millibars)	Storm Type
Unnamed	October 5, 1898	25	Not Available	Tropical Storm
Unnamed	September 11, 1900	50	Not Available	Extra-Tropical Storm
Unnamed	June 29, 1902	35	Not Available	Extra-Tropical Storm
Unnamed	June 14, 1906	30	Not Available	Extra-Tropical Storm
Unnamed	September 6, 1915	25	Not Available	Tropical Storm
Unnamed	October 19, 1923	35	Not Available	Extra-Tropical Storm
Unnamed	September 4, 1932	40	Not Available	Extra-Tropical Storm
Unnamed	September 25, 1941	40	985	Extra-Tropical Storm
Unnamed	September 7, 1948	20	Not Available	Tropical Storm
Unnamed	October 6, 1949	25	Not Available	Tropical Storm
Unnamed	June 29, 1960	15	Not Available	Tropical Storm
Carla	September 14, 1961	30	Not Available	Extra-Tropical Storm
Candy	June 25, 1968	25	Not Available	Tropical Storm
Gilbert	September 20, 1988	25	995	Extra-Tropical Storm
Arlene	June 13, 2005	20	1005	Tropical Depression
Rita	September 26, 2005	20	1006	Tropical Depression
Dennis	July 16, 2005	10	1014	Tropical Depression
Ike	September 14, 2008	50	988	Extra-Tropical Storm
Gustav	September 5, 2008	20	1002	Extra-Tropical Storm

Note: Values from HURDAT (NOAA 2014b). Wind values are the maximum sustained wind in the storm system.

Table 3-5: Top 20 Surges at Holland, Michigan from 1959 to 2014

Rank	Date	Surge Height above Ambient Lake Level (feet)
1	12/4/1990	2.01
2	12/2/1985	1.75
3	12/30/1971	1.58
4	4/4/1982	1.55
5	4/18/2013	1.49[1]
6	1/4/1982	1.47
7	12/21/2012	1.46[1]
8	1/26/1971	1.45
9	11/15/1961	1.43
10	2/15/1967	1.42
11	12/15/1968	1.36
12	12/9/2009	1.36
13	4/15/1963	1.35
14	9/30/2011	1.32[1]
15	12/12/2010	1.30[1]
16	3/10/2002	1.25
17	1/25/1990	1.25
18	12/16/1987	1.25
19	12/4/1970	1.25
20	4/15/1966	1.25

Source: (USACE 2012a)

Note: 1) Additional values added based on analysis of Holland, Michigan data from 2010 to July 2014.

Table 3-6: Top 12 Storm Surge Events at Holland, Michigan up to 2014

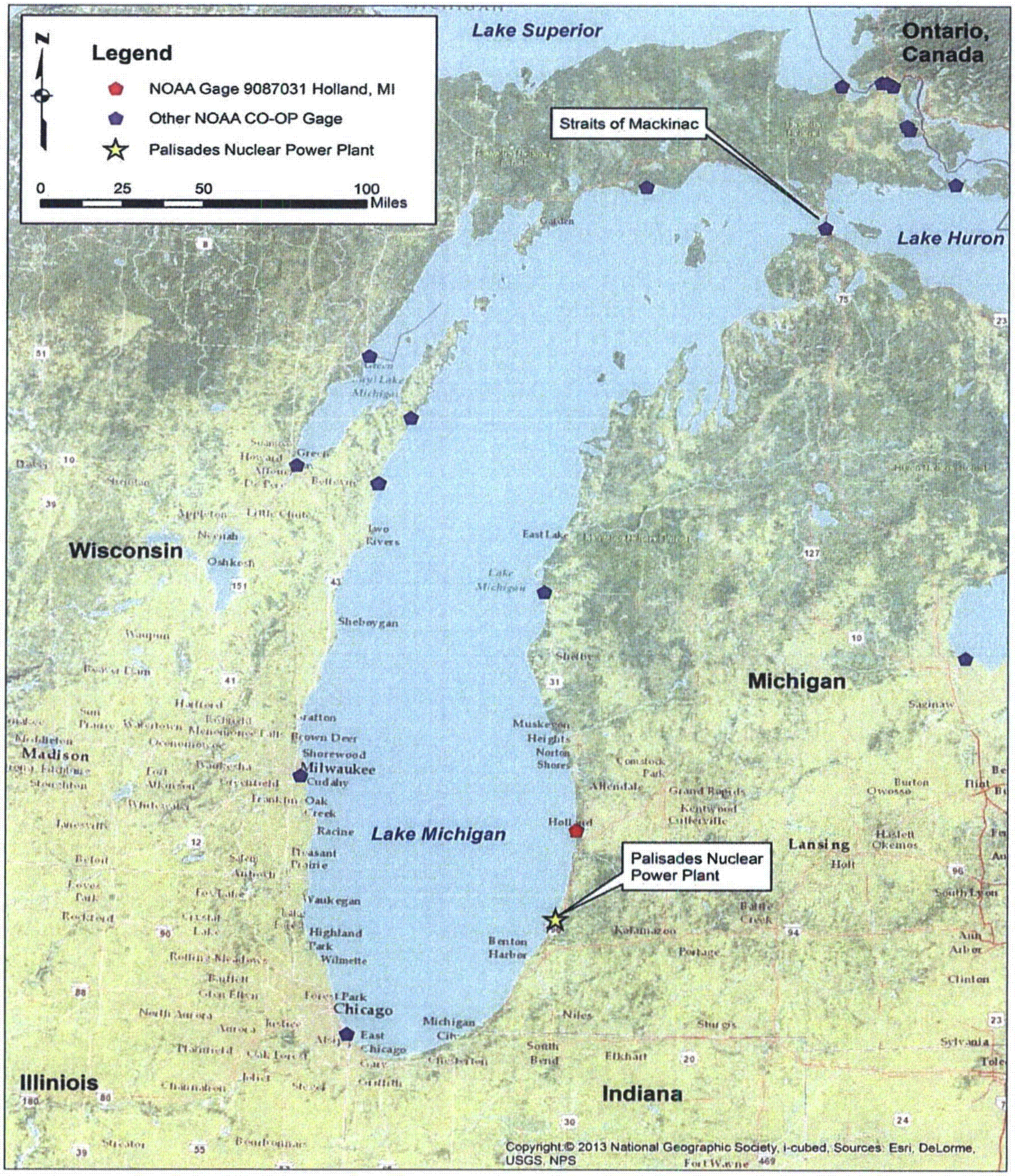
Rank	Date	Surge Height above Ambient Lake Level (feet)	Translation Speed (miles/hour)	Storm Track Bearing (degrees clockwise from north)
1	12/4/1990	2.01	17	50
2	12/2/1985	1.75	48	49
3	12/30/1971	1.58	47	95
4	4/4/1982	1.55	30	79
5	4/18/2013*	1.49	62	N/A
6	1/4/1982	1.47	59	45
7	12/21/2012	1.46	22	70
8	1/26/1971	1.45	30	60
9	11/15/1961	1.43	39	57
10	2/15/1967	1.42	66	77
11	12/15/1968	1.36	46	68
12	12/9/2009	1.36	33	55

Note: (*) Storm on 4/18/2013 (Surge Rank #5) was most likely a convective storm and was not considered as an extra-tropical storm. Surge Height above Ambient Lake Level is based on values calculated by the USACE (USACE 2012a). Translational speed was calculated in "Palisades Nuclear Plant Flooding Hazard Re-Evaluation - Probable Maximum Wind Storm Calculation" (AREVA 2014a). Storm Track Bearing was calculated in "Palisades Nuclear Plant Flooding Hazard Re-Evaluation - Probable Maximum Storm Surge and Seiche Calculation" (AREVA 2014b).

Table 3-7: Maximum Surge Stillwater Height near PLP for each Storm Type

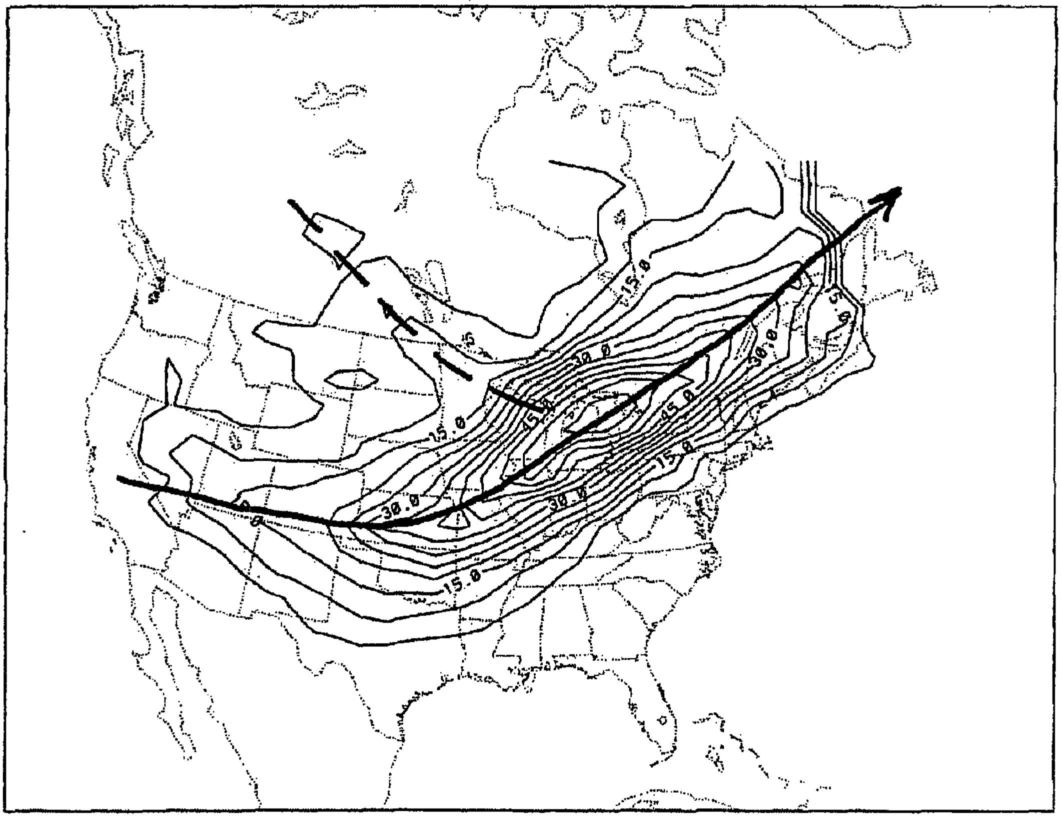
Storm Type	Surge Height near PLP (feet)
Extra-Tropical Storm	9.7
Moving Squall Line	3.6

Figure 3-9: Site Setting



Basemap Source: (ESRI 2014a)
Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

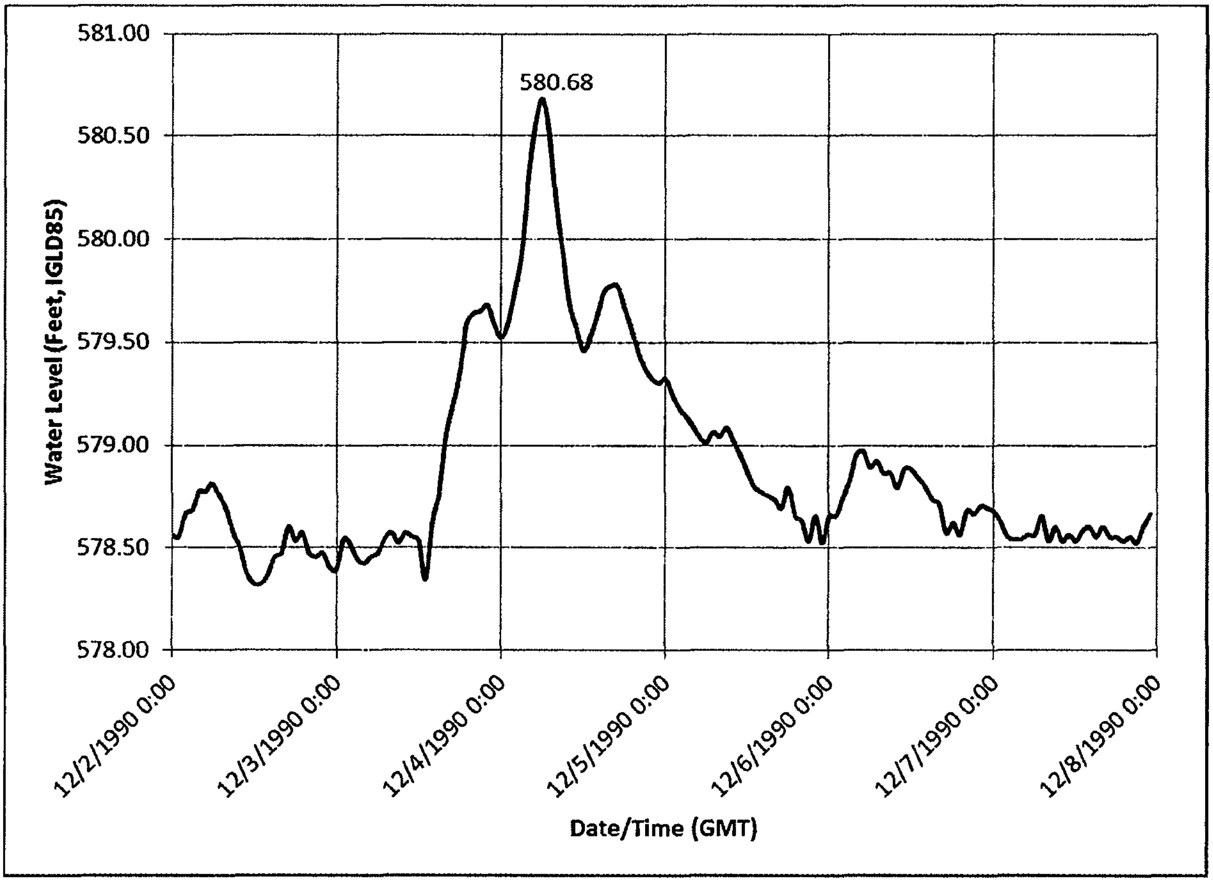
Figure 3-10: Frequency of Strong Winter Cyclones for the Michigan-Huron-St. Clair Region



Source: (Angel 1996)

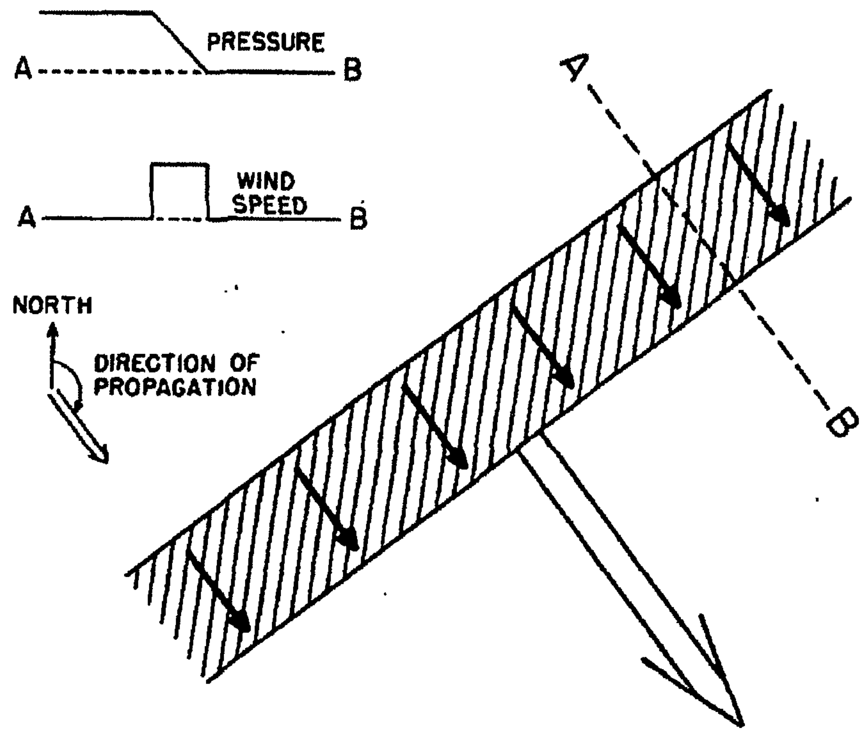
Note: Solid lines indicate the primary tracks along the axes of high frequency. Secondary tracks (dashed lines) are along the axes of lesser frequency.

Figure 3-11: December 4, 1990 Storm Lake Michigan Water Level Hydrograph at Holland, Michigan



Source: (NOAA 2014e)

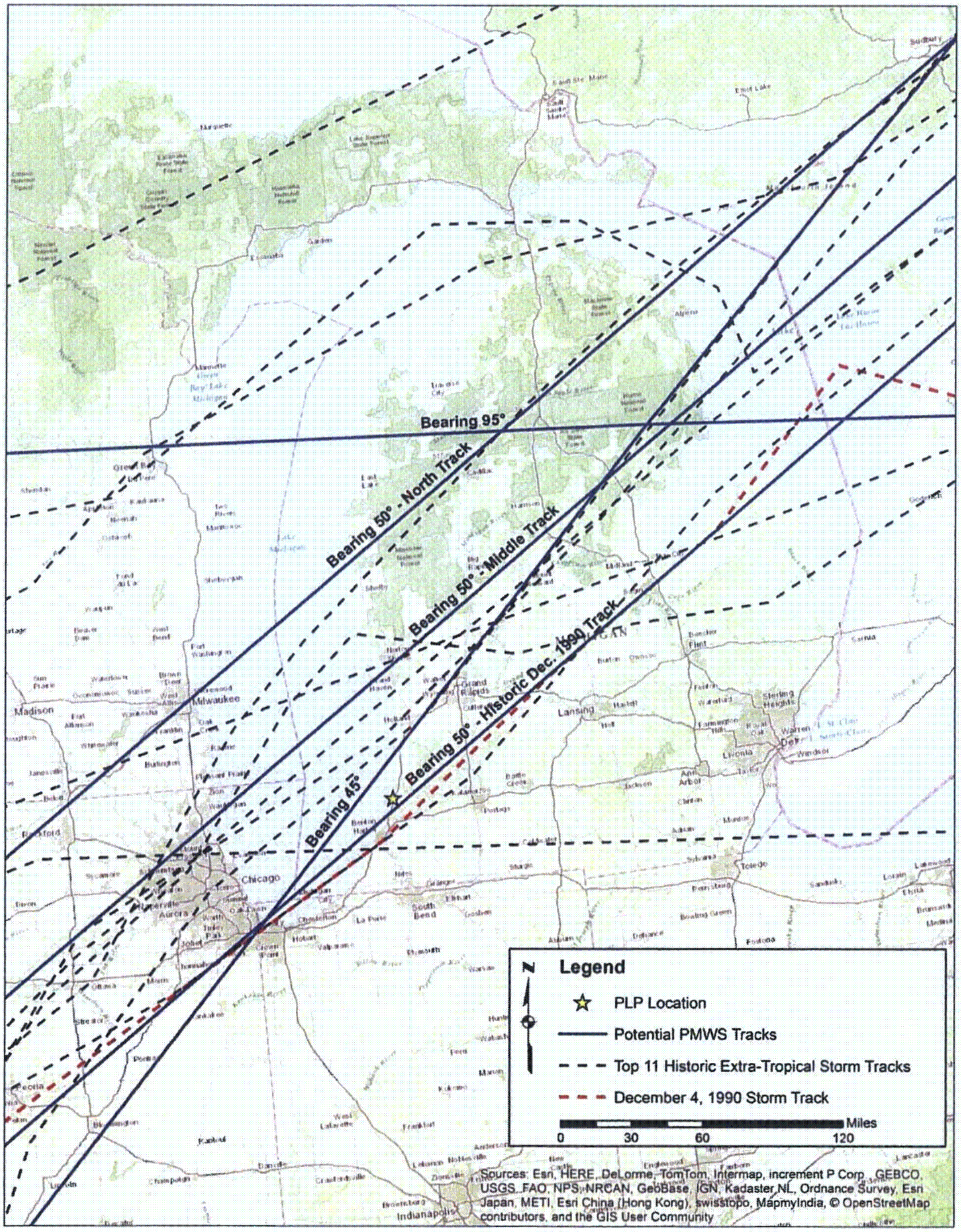
Figure 3-12: Idealized Squall Line Configuration



Source: (Platzman et al., 1965)

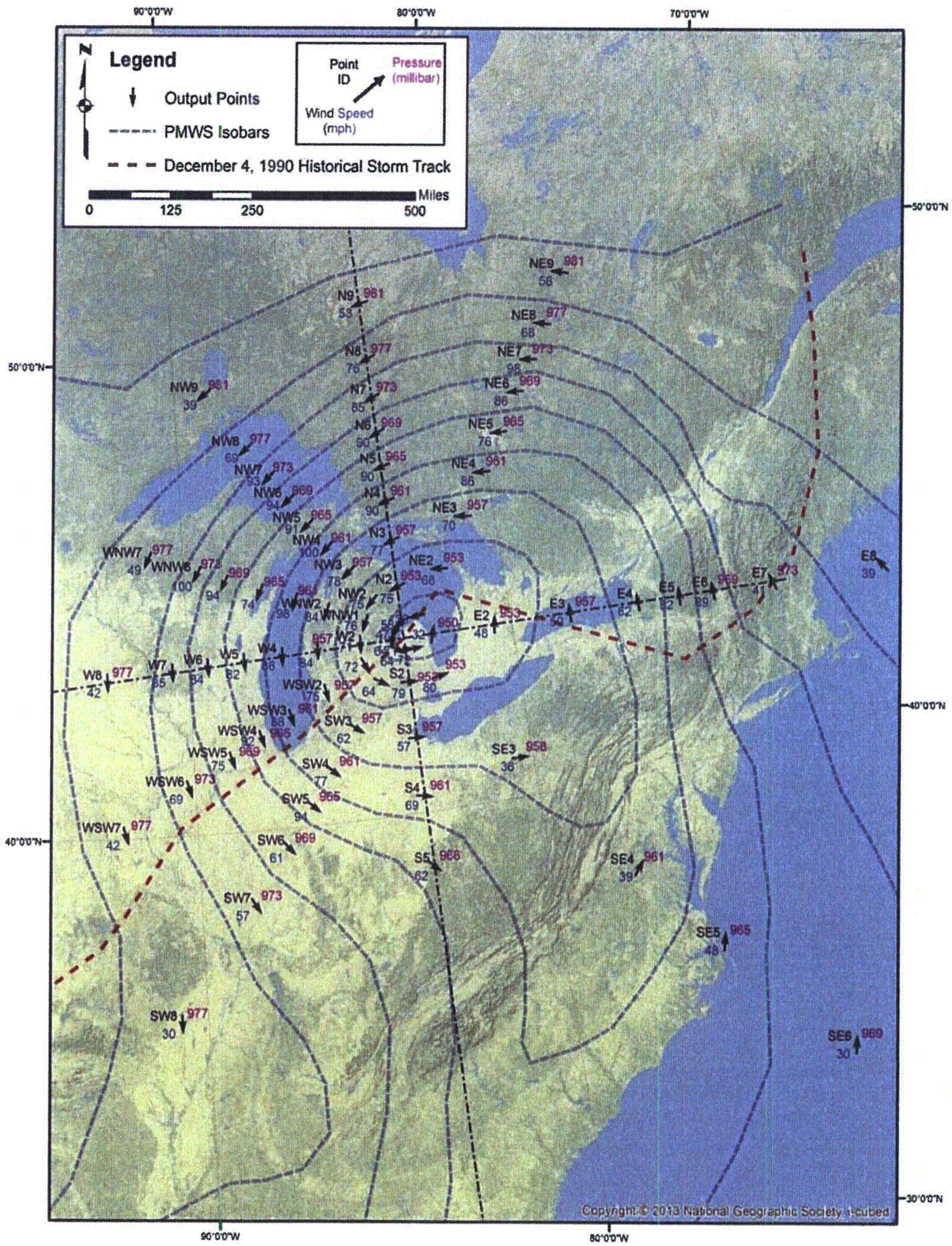
Inserts show pressure and wind speed distributions in a section perpendicular to the squall line and definition of angle used to specify direction of propagation vector.

Figure 3-13: Potential Probable Maximum Wind Storm Tracks



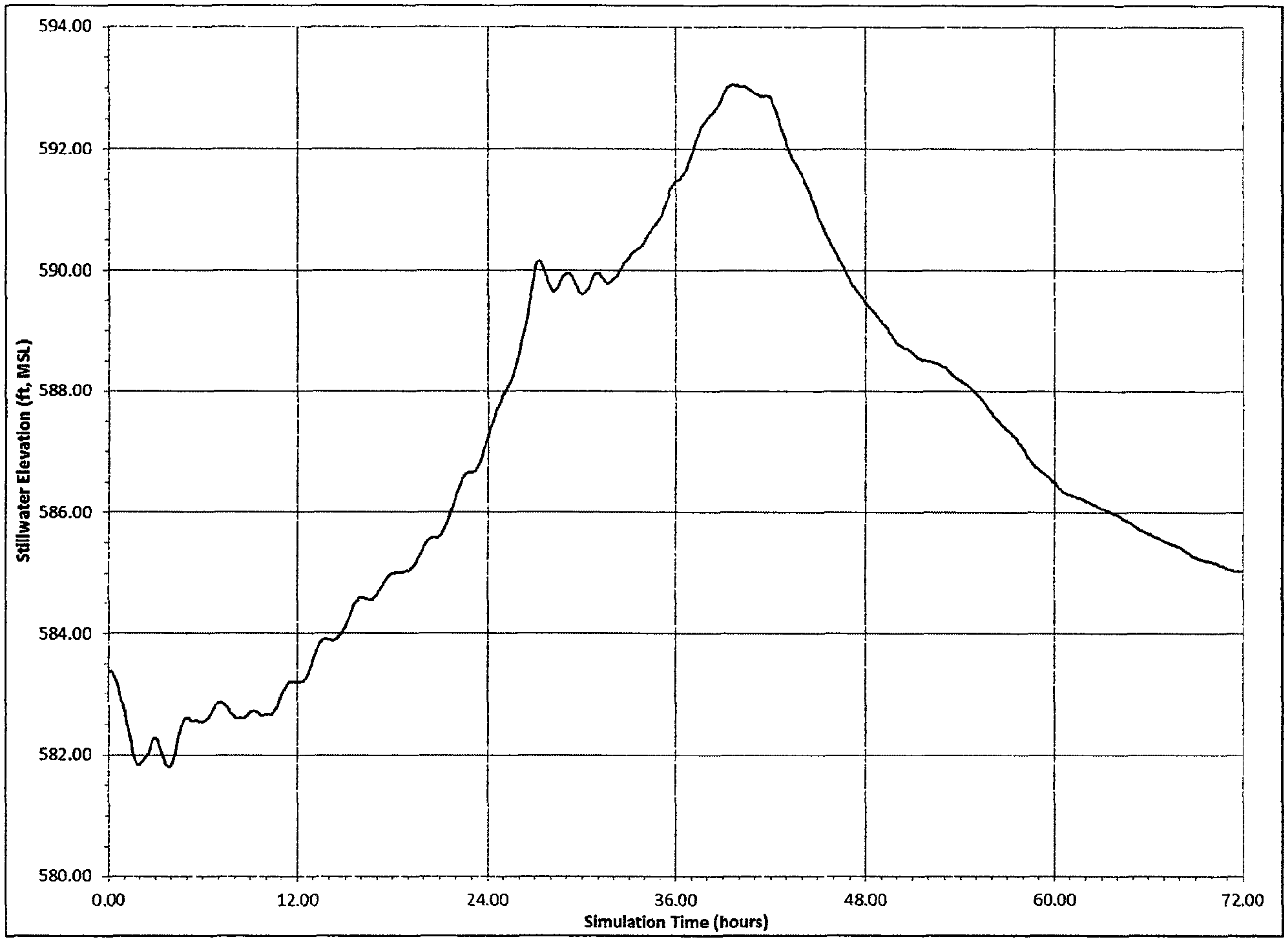
Basemap Source: (ESRI 2014b)
Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

Figure 3-14: Probable Maximum Wind Storm Parameters



Basemap Source: (ESRI 2014a)
Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

Figure 3-15: Controlling Extra-Tropical Storm Surge Stillwater Hydrograph near PLP



3.5 Seiche

This section addresses the potential for flooding at PLP due to the Probable Maximum Seiche (PMS). Seiches are long period standing waves in enclosed or partially enclosed bodies of water (USACE 2012 and FEMA 2014). Seiches are initiated by external forcing, generally of an atmospheric, tsunami, or seismic nature. Cessation of the external force causes periodic water level fluctuations as the standing wave reflects from the ends of the lake (USACE 2012). Seiches can occur in Lake Michigan, resulting in elevated still-water elevations in the vicinity of PLP.

This section summarizes the PMS evaluation performed in the calculation titled Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Probable Maximum Storm Surge and Seiche (AREVA 2014a).

3.5.1 Methodology

A seiche is an oscillation of the water surface in an enclosed or semi-enclosed body of water initiated by some external force (NRC 2007). Enclosed basins such as cooling reservoirs, ponds or lakes are not present at PLP; therefore, an evaluation of these types of basins is not required. However, due to the coastal setting of PLP on the shore of Lake Michigan, evaluation of seiches occurring on Lake Michigan and their potential impact to PLP was performed. Seiches can also result from lake excitations due to external forcing including meteorological events, earthquakes, and landslides.

The HHA approach described in NUREG/CR-7046 (NRC 2011) was used to determine whether a seiche in Lake Michigan can result in significant flooding at PLP. This approach initially involves the determination of the natural period of the lake, evaluation of the natural oscillation periods of the external forces, such as tropical and extra-tropical storms, comparison of the periods to determine if resonance is possible, and review of water level data to evaluate potential seiche heights.

3.5.2 Results

A seiche from a landslide-induced wave would not exceed the level of the initial wave. The maximum runup onto the site from such initial waves is 592.2 feet based on the tsunami screening assessment (see Section 3.6) (AREVA 2014b). This water surface elevation is lower than the Probable Maximum Storm Surge (PMSS) water level and is therefore, bounded by the PMSS.

The resonant period of Lake Michigan is approximately 4 days (Saylor et al., 1980). High-energy earthquake waves have substantially shorter periods than the resonant period of Lake Michigan and therefore, were not considered.

Seiche events resulting from storm surge are not considered a unique flood hazard because they produce water elevations which are equal to or lower than the surge event that initiated them (FEMA 2014). Additionally, seiches are not accompanied by storm waves which would be coincident with storm surge (USACE 2012). Therefore, seiches on Lake Michigan are bounded by the PMSS and the combined effects of wind-waves with the PMSS.

3.5.3 Conclusions

Seiches on Lake Michigan were determined to be bounded by the PMSS. Based on literature review, the resonant period of Lake Michigan is four days. Earthquake- and landside-induced seiches will not result in resonance and therefore are not expected to be greater than the initial rise in water level generated by the initial events.

No further analysis or modeling is required due to the direct observational evidence that potential seiches on Lake Michigan will not be the controlling flooding event at PLP and will not impact structures, systems and components important to safety.

3.5.4 References

AREVA 2014a. AREVA Document No. 32-9226962-000, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Probable Maximum Storm Surge and Seiche, 2014.

AREVA 2014b. Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Screening for Tsunami, AREVA Document No. 51-9224283-000.

FEMA 2014. Guidelines and Standards for Flood Hazard Mapping Partners: FEMA Great Lakes Coastal Guidelines, Appendix D.3 Update, Federal Emergency Management Agency, January, 2014.

NRC 2007. NUREG-0800, United States Nuclear Regulatory Commission (NRC) Standard Review Plan, revised March 2007.

NRC 2011. NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, U.S. Nuclear Regulatory Commission, Springfield, VA, National Technical Information Service, 2011.

Saylor et al., 1980. Vortex Modes in Southern Lake Michigan, James H. Saylor, Joseph C. K. Huang, Robert O. Reid, Journal of Physical Oceanography, Volume 10, August 1, 1980. (See AREVA Document No. 32-9226962-000)

USACE 2012. Wave Height and Water Level Variability on Lakes Michigan and St Clair, ERDC/CHL TR-12-23, Great Lakes Coastal Flood Study, 2012 Federal Inter-Agency Initiative, U.S. Army Corps of Engineers, Engineer Research and Development Center Coastal and Hydraulic Laboratory, October 2012. (See AREVA Document No. 32-9226962-000)

3.6 Tsunami

A tsunami is a series of water waves generated by a rapid, large-scale disturbance of a water body due to seismic, landslide, or volcanic tsunamigenic sources (NRC 2009, Section 1.3). Although the term tsunami typically relates to oceanic tsunamis, as an inland site, PLP may be susceptible to tsunami-like waves in Lake Michigan (NRC 2009, Section 2.1).

3.6.1 Methodology

The HHA approach described in NUREG/CR-6966 (NRC 2009) and Interim Staff Guidance JLD-ISG-2012-06 (NRC 2013) was used for tsunamis and considered the first two of three steps. The third step was unnecessary based on the results of the first two steps, which answered the questions:

1. Is the site region subject to tsunamis?
2. Is the plant site affected by tsunamis?

Question 1 was answered by performing a regional survey and assessment of tsunamigenic sources. The regional survey was conducted in four parts. The first part of the regional survey was to review the Global Historical Tsunami Database (NOAA 2014a), maintained by the National Oceanic Atmospheric Administration (NOAA)'s National Geophysical Data Center (NGDC), to determine the history of tsunamis. The second, third, and fourth parts of the regional survey included an assessment of the mechanisms likely to cause a tsunami.

Question 2 was answered by using the results from Question 1 to identify the primary effects of a tsunami wave near the PLP site and then performing a site screening to determine the potential effects to the PLP site (AREVA 2014).

3.6.2 Results

3.6.2.1 Regional Survey

Tsunamis are generated by rapid, large-scale disturbance of a body of water. Therefore, only geophysical events that release a large amount of energy in a very short time into a water body generate tsunamis. The most frequent cause of tsunamis is an earthquake. Less frequently, tsunamis are generated by submarine and subaerial landslides and volcanic eruptions (NRC 2009, Section 1.3). Meteorite impacts and ice falls can also generate tsunamis, but were excluded from the regional survey because meteorite impacts are considered infrequent events in comparison to earthquakes (NRC 2009, Section 6.2) and ice falls, which are glacial ice processes (NRC 2009, Section 1.3.2), are comparable to subaerial landslides.

3.6.2.1.1 NGDC Database Review

The NGDC tsunami-source-event database is global in extent with information dating from 2000 B.C. to the present (NOAA 2014a). As an inland site, PLP needs to consider the possibility of a tsunami-like wave in water bodies in the region (NRC 2009, Section 2.2). As a result, the regional survey considered tsunami-like waves in the area around the Great Lakes, extending from 41° to 49° N Latitude and 74° to 92° W Longitude (see Figure 3-16).

Seven events occurred in or near the Great Lakes. All events resulted in a seiche or disturbance in an inland river. Two of the events were caused by earthquakes, two by meteorological conditions, one by a landslide and the remaining two by unknown conditions. The maximum event water height increase was nearly 10 feet in Lake Michigan and was related to a meteorological event. The maximum event water height related to an earthquake was 9 feet in Lake Erie.

3.6.2.1.2 Earthquakes

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 (NRC 2009, Section 1.3.1).

Based on the information presented in the PLP FSAR, seismicity of the PLP region is considered slightly active (PLP 2012, Section 2.4.2). Three recorded earthquakes with epicentral intensities of V or VI on the Modified Mercalli (MM) scale have occurred within a 100-mile radius of the PLP site (PLP 2012, Table 2-13). These earthquakes, which occurred since the early 1800s, are equivalent to a magnitude of 4.9 to 5.0 (USGS 2014a). The largest earthquake within 200 miles of the site had a maximum MM of VII-VIII, and occurred 150 miles from the PLP site. This event is equivalent to a magnitude of 5.9 to approximately 6.5 (USGS 2014a). The largest seismic event that occurred to the south of PLP, with a magnitude greater than 6.5, was situated southwest of Indianapolis, about 200 miles from the PLP site (NRC 2006, Figure 2.5.1-4).

Seismic activity outside the region can also produce tsunami like waves within the region (USGS 2014b). For example, seismic waves from the Alaska earthquake of 1964 caused water bodies to oscillate at many places in North America, including the southern tip of Lake Michigan. Seiches were recorded at hundreds of surface-water gaging stations. The seismic seiche distribution did not have an obvious dependence on distance or azimuth from the epicenter. Instead, the distribution had a regional pattern, which reflected the influence of major geologic features. Favorable conditions for seismic seiche generation include thrust faults (USGS 2014b); Lake Michigan lacks such features (NOAA 2014b). Seiche locations were also controlled by structural uplifts and basins (i.e., the Michigan basin) (USGS 2014b).

3.6.2.1.3 Landslides

There are two broad categories of landslides: (1) subaqueous that are initiated and progress beneath the surface of the water body, and (2) subaerial that are initiated above the water and impact the water body during their progression or fall into the water body. In addition, landslide-generated tsunamis have a very strong directivity in the direction of mass movement. Therefore, the outgoing wave from the landslide source propagates in the direction of the slide. Also, the amplitude of the outgoing wave from a landslide is affected by the terminal velocity of the movement, which in turn is a function of the repose angle, i.e., the slope angle. Subaqueous slopes present in Lake Michigan of less than 10 degrees do not approach sufficient steepness to provide an unstable slope condition that would result in a slope failure having a terminal velocity sufficient to produce a tsunami-like wave (AREVA 2014). The mass of the landslide is also important. The most common landslide mechanism for either landslide category is an earthquake (NRC 2009, Section 1.3.2).

Subaqueous Landslide – Lake Michigan Bathymetry

The general bathymetry of Lake Michigan is divided into three main areas: the Chippewa Basin and the region to the north, the Mid Lake Plateau, and the South Chippewa Basin (see Figure 3-17). The Chippewa Basin is the largest and deepest basin in Lake Michigan. There are ridges throughout the basin with the potential to produce a subaqueous landslide. However, the ridge locations, directional trending, and slopes make it unlikely that observable tsunami-like waves would be generated. For example, the ridge lines in the north region of the basin trend north-south. Thus, if a landslide occurred in the north area it would be east-west, opposite the direction of PLP to the south. The trending of the ridges in the southern end of the basin is similar, with ridges oriented such that given a subaqueous landslide it would be directed away from PLP. An exception is the Door-Leelanau Ridge, which trends east-west. It, therefore, has the potential of generating a tsunami-like wave to the south. The slope of the ridge, however, is less than two degrees (Map ID B5 in Table 3-8 and Figure 3-18). Thus, given a

landslide, its speed would be limited and judged unlikely to generate an observable tsunami-like wave. As a result, the effect to the PLP site would be minimal, if any.

The Mid Lake Plateau is a broad, relatively flat-topped cuesta, which has depths of less than 295 feet and extends upward to minimum depths of 131 to 197 feet. Escarpments form its northwestern, western, and southwestern boundaries. Deeper channels on the west and east, and the South Chippewa Basin on the south, bound the Mid-Lake Plateau and isolate it from other shallow water areas.

The steepest gradients occur on the northwest portion of the plateau abutting the Milwaukee Basin. There are lesser steep gradients abutting the Ludington Basin to the north and Muskegon Basin to the east. The maximum slope in these gradient areas is about two degrees (Map ID B1-B3, Table 3-8 and Figure 3-18). The gradients trend northeast-southwest and given a subaqueous landslide in the area and resultant tsunami-like wave, it would be directed westward or eastward away from PLP. As a result, the effect to the PLP site would be minimal, if any.

The South Chippewa Basin, unlike the Chippewa Basin and Mid Lake Plateau, is characterized by linear bathymetry with uniform gradients. The maximum slope of gradients is in the western portion of the basin (i.e., southwestern part of the lake), where the gradients trend north/northeast-south/southwest. Thus, a potential landside would be directed east/southeast toward PLP. However, the largest slope in the area is less than two degrees and has a relief of only 33 feet. See Map ID B4, Table 3-8 and Figure 3-18 . Thus, given a landslide, its speed and mass would be limited and judged unlikely to generate an observable tsunami-like wave.

Subaerial Landslide – Lake Michigan Topography

The geographical areas where subaerial landslides occur are generally limited to areas of steep shoreline topography (NRC 2009, Section 1.3.2).

There are numerous areas along the lake perimeter that have a moderate to high susceptibility for landslide potential. Most of these areas, however, were judged to have little, if any, effect to the PLP site. Given a landslide, any tsunami-like wave propagation in these areas would be directed away from the PLP site due to the shoreline orientation. See Map IDs T3 – T5, Table 3-9 and Figure 3-19.

The exception is the southwestern area of the lake perimeter, extending from Racine, Wisconsin, to north of Chicago, Illinois. The direction of a landslide in this area, if it occurred, would be toward the opposite lake southeastern shoreline, where PLP is located.

The maximum slope of gradients of the area is in Highland Park, Illinois, with slopes of up to 29 degrees (Map ID T1, Table 3-9 and Figure 3-19). But the gradients in the area trend in a northwest-southeast direction. Thus, a potential landside would be directed northeast, north of PLP (Figure 3-19).

The gradient trend in the area south of Racine, Wisconsin, however, is north/northeast-south/southwest. Thus, a landslide tsunami-like wave would propagate east/southeast, toward PLP (Figure 3-19). The speed of such a wave would, however, be limited because the shoreline gradient is less than ten degrees (Table 3-9). Similarly, the mass would be limited as well because the shoreline has a maximum 50-foot relief. Thus, the amplitude of any tsunami-like wave originating from a subaerial landslide in the south Racine, Wisconsin area is judged to be limited, resulting in little if any effect to PLP.

3.6.2.1.4 Volcanoes

The Global Historical Volcano Database, also maintained by the NGDC, was used to conduct a regional survey to determine if volcanic activity could be a mechanism to produce a tsunami or tsunami-like wave. The database survey area reviewed was the same as for the tsunami regional survey; however, no data were found (NOAA 2014c).

3.6.2.1.5 Regional Survey Findings

Based on the regional survey results,

- PLP is not susceptible to oceanic tsunamis; instead, there is the potential of tsunami-like waves in Lake Michigan.
- Seven tsunami-like events have occurred that have produced tsunami-like waves, namely seiches, in or near the Great Lakes. Two of these events were caused by earthquakes, one by a landslide, two by meteorological conditions and two by unknown conditions.
- Seismicity of the PLP site is low. No recorded earthquakes with magnitudes greater than 6.5 have occurred within a 100-mile radius of the PLP site. The nearest seismic event that has occurred with a magnitude greater than 6.5 is nearly 200 miles south of the PLP site.
    - Subaqueous landslides, if they occurred, are unlikely to generate an observable tsunami-like wave due to gradient orientation, the limited bathymetric relief of ridges (45 m or less) and their respective slopes (less than 3 degrees). See Table 3-8.
    - Subaerial landslides around the west, north, and east perimeter of Lake Michigan are unlikely to affect PLP because topographic trends would direct any resultant tsunami-like wave away from the site. The exception is the southwest lake perimeter. The topography in this area is oriented such that the direction of a landslide and resultant tsunami-like wave, if it occurred, would be toward PLP. However, given a landside, it would cause little, if any, effect to the PLP site due to gradient orientation, limited topographic and bathymetric, or slope angle (less than 10 degrees). See Table 3-9.
- No records were found of volcanic activity being a source mechanism to produce a tsunami or tsunami-like wave in or near the Great Lakes.

3.6.2.2 Site Screening

Based on the regional survey, tsunami-like waves (seiches) have occurred in or near the Great Lakes, including Lake Michigan. At least two of the reported waves were caused by meteorological conditions. Three, however, were due to tsunamigenic sources (two from an earthquake and one from a landslide).

The primary effects of the tsunami-like waves on the PLP site are flooding due to runup from the wave and loss of cooling water due to dry intakes during drawdown caused by the receding wave. Other effects that may cause damage to structures and the foundations of these structures include hydrostatic and hydrodynamic forces, debris projectiles, and sediment erosion and deposition.

3.6.2.2.1 Flooding due to Runup

From the NGDC database results in the regional survey, the maximum reported non-meteorological tsunami-like wave in any of the Great Lakes is 9 feet and is the result of an 1823 earthquake. Adding this value to the maximum recorded lake level of (582.35 feet (IGLD85) + 0.88 feet =) 583.2 feet MSL, which occurred in October 1986 (IDNR 2014), yields 592.2 feet MSL, and is 2.2 feet below the PLP flood protected elevation of 594.4 feet MSL.

3.6.2.2.2 Lack of Cooling Water Due to Drawdown

Lake water serves as the cooling medium (Ultimate Heat Sink) for the removal of waste heat (PLP 2012, Section 9.1.1). Lake water enters through an intake crib located off shore, approximately 3,000 feet from the Intake

Structure. The intake bell, located under the intake crib, is in approximately 25 feet of water; however, actual submergence varies based on Lake Michigan water level (PLP 2012, Sections 9.12.2.1).

Lake Michigan reached a historic minimum level of (576.02 feet (IGLD85) + 0.88 feet =) 576.9 feet MSL in January 2013 (IDNR 2014). Although the top of the sluice gate opening is at elevation 568.25 feet, the minimum water level for the service water pumps is at elevation 557.25 feet MSL which corresponds to the bottom of the pump suction bell (PLP 2014). Therefore, the margin between the historic minimum lake level and the service water pump minimum water level is 19.65 feet (576.9 feet – 557.25 feet). If a drawdown from a tsunami-like wave is assumed with an amplitude equivalent to that of runup (9 feet), in conjunction with the historic minimum lake level, there would still be sufficient margin to protect the service water pumps since there would be 10.65 feet (19.65 feet – 9 feet) of water above the bottom of the pump suction bell.

3.6.2.2.3 Other Effects

Other effects from a tsunami-like wave that may cause damage to structures and the foundations of these structures include hydrostatic and hydrodynamic forces, debris projectiles, and sediment erosion and deposition (NRC 2009, Section 3; NRC 2013, Section 4).

All below grade portions of PLP structures were designed for hydrostatic pressures based on a groundwater elevation of 585 feet (PLP 2012, Section 5.9.1.4). Referring to Section 3.6.2.2.2 above, the elevation for a tsunami drawdown of 9 feet, concurrent with the historic minimum lake level, would be 567.9 feet (576.9 – 9); hence the hydrostatic pressure for a tsunami drawdown would be less than the design hydrostatic pressure.

The nearest structure to Lake Michigan housing equipment important to safety is the Intake Structure (PLP 2012, Figure 1-1 and Table 5.2-2). It has previously been determined that the Intake Structure can withstand the dynamic effect of up to 8 feet of wave runup (PLP 2012, Section 5.4.1.2). Considering that the shoreline plant grade is at elevation 589 feet (PLP 2012, Section 5.4.1.1), an 8-foot wave run-up would be at elevation 597 feet which is 4.8 feet (597 – 592.2) above the elevation for the tsunami wave runup described in Section 3.6.2.2.1 above; hence, the hydrodynamic forces from a tsunami wave runup would be less than those previously considered for the Intake Structure. Similarly, considering that Class 1 structures (i.e., equivalent to seismic Category I structures (PLP 2012, Section 5.2.2.1)) have previously been determined to be able to resist the effects of the design basis flood (PLP 2012, Section 5.4.1.2), other structures housing equipment important to safety are judged to have sufficient margin to protect them from the effects of hydrodynamic forces from a tsunami wave runup.

Pursuant to debris projectiles, considering that PLP Class 1 structures are designed for tornado-borne missiles (PLP 2012, Section 5.5.1), PLP structures and their foundations are judged to have sufficient margin to sustain debris projectiles associated with a tsunami-like wave runup or drawdown. Similarly, considering that most Class 1 structures have thick concrete walls and roofs and that Class 1 structures are designed for tornado wind effects, except for the steel-framed structure over the spent fuel pool (PLP 2012, Section 5.3.2 and Table 5.5-1), PLP structures are also judged to be of sufficient robust design to withstand potential sediment and erosion deposition from a tsunami wave runup or drawdown. Note that although the steel-framed structure over the spent fuel pool could be damaged by tornado winds, it was previously determined that such damage would not adversely impact the safe shutdown of the plant (PLP 2012, Section 5.3.3). Additionally, the spent fuel pool structure is above the elevation of the tsunami-like wave elevation (PLP 2005).

3.6.3 Conclusions

Based on the regional survey and site screening results, the following conclusions are made:

- As an inland site, the PLP site is not subject to oceanic tsunamis; however, tsunami-like waves (seiches) have occurred in the Great Lakes region. Two of such were related to earthquakes and one to a landslide. The others were caused by meteorological conditions or by unknown events.
- Tsunami-like waves generated from
  - an earthquake are limited because the required level of seismic activity for development of a tsunami, i.e., an earthquake with a magnitude greater than 6.5, is essentially absent within a 100-mile radius of the PLP site;
  - a subaqueous landslide are unlikely to generate an observable tsunami-like wave due to the limited bathymetric relief of ridges and their respective slopes and orientation; and
  - a subaerial landslide around the west, north, and east perimeter of Lake Michigan are unlikely to affect PLP because topographic trends would direct any resultant tsunami-like wave away from the site. The exception is the southwest lake perimeter, where the topography is oriented such that a landslide and resultant tsunami-like wave, if it occurred, would be directed toward PLP. However, given a landside, it would cause little, if any, effect to the PLP site because of the limited topographic relief and slope angles.
- Notwithstanding the occurrence of tsunami-like waves, the potential effects on the PLP site (wave runup, drawdown, and other effects assessed) are negligible because there is sufficient physical margin to protect structures important to safety. The margin is based on the maximum recorded tsunami-like wave resulting from an earthquake in the Great Lakes region occurring coincident with the maximum and minimum lake levels as it applies to the tsunami-like wave runup and drawdown, respectively.

3.6.4 References

AREVA 2014. AREVA Document No. 51-9224283-000, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Screening for Tsunami, August 2014.

IDNR 2014. Indiana Department of Natural Resources, Lake Level Fluctuations, Lake Michigan Lake Level Graph 1960-2014; Available at: http://www.in.gov/dnr/water/3661.htm; Date accessed May 27, 2014. (See AREVA Document No. 51-9224283-000)

NOAA 2014a. National Oceanic Atmospheric Administration, National Geophysical Data Center, Tsunami Database; Available at: http://www.ngdc.noaa.gov/hazard/tsu_db.shtml; Date accessed May 27, 2014. (See AREVA Document No. 51-9224283-000)

NOAA 2014b. National Oceanic Atmospheric Administration, National Geophysical Data Center, Lake Michigan Bathymetry Website: http://www.ngdc.noaa.gov/mgg/greatlakes/lakemich_cdrom/html/geomorph.htm, date accessed May 28, 2014. (See AREVA Document No. 51-9224283-000)

NOAA 2014c. National Oceanic Atmospheric Administration, National Geophysical Data Center, Volcano Database; Available at: http://www.ngdc.noaa.gov/hazard/volcano.shtml; Date accessed May 30, 2014. (See AREVA Document No. 51-9224283-000)

NRC 2006. NUREG-1844, Safety Evaluation Report for an Early Site Permit (ESP) at the Exelon Generation Company, LLC (EGC) ESP Site, Section 2.5, Geology, Seismology, and Geotechnical Engineering, U.S. NRC, May 2006.

NRC 2009. NUREG/CR-6966, Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America, Final Report, U.S. NRC, March 2009.

NRC 2013. JLD-ISG-2012-06, Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment, Interim Staff Guidance, Revision 0, January 2013.

PLP 2005. Palisades Plant Drawing M-7, Equipment Location Reactor Building Section F-F, Revision 13, Dated September 14, 2005. (See AREVA Project Manager's approval of customer references on page 2.)

PLP 2012. Palisades Nuclear Plant Final Safety Analysis Report (FSAR), Revision 30, September 4, 2012. (AREVA Document No. 38-9223712-000)

PLP 2014. Palisades Nuclear Plant, Limiting Conditions of Operation, Section 3.7.9, Ultimate Heat Sink (UHS), revised April 14, 2011. (AREVA Document No. 38-9223712-000)

USGS 2014a. U.S. Geological Survey, Magnitude / Intensity Comparison, Website: http://earthquake.usgs.gov/learn/topics/mag_vs_int.php; Date accessed May 27, 2014. (See AREVA Document No. 51-9224283-000)

USGS 2014b. U.S. Geological Survey, Seismic Seiches, Website: http://earthquake.usgs.gov/learn/topics/seiche.php; Abridged from Earthquake Information Bulletin,January-February 1976, Volume 8, Number 1; Date accessed May 28, 2014. (See AREVA Document No. 51-9224283-000)

Table 3-8: Lake Michigan Prominent Bathymetric Feature Slopes

Map ID (see Figure 3-18)	Gradient Length (m)	Gradient Height (m)	Slope (degrees)	Tsunami-like Wave Direction
B1	1,856	45	1.39	NNW
B2	1,480	30	1.16	NNE
B3	900	35	2.23	WSW
B4	1,950	10	0.29	ESE
B5	1,600	40	1.43	S

Table 3-9: Lake Michigan Prominent Topographic Features

Map ID (see Figure 3-19)	Topographic Quadrangle	Gradient Length (feet)	Gradient Height (feet)	Slope (degrees)	Tsunami-like Wave Direction
T1	Highland Park, IL	90	50	29.05	NE
T2	Racine South, WI	335	50	8.49	ESE
T3	Racine North, WI	290	90	17.24	NE
T4	Port Washington East, WI	200	40	11.31	SE
T5	McGulpin Point, MI	650	80	7.02	SSW

Figure 3-16: NGDC Tsunami-Source-Event Database Region

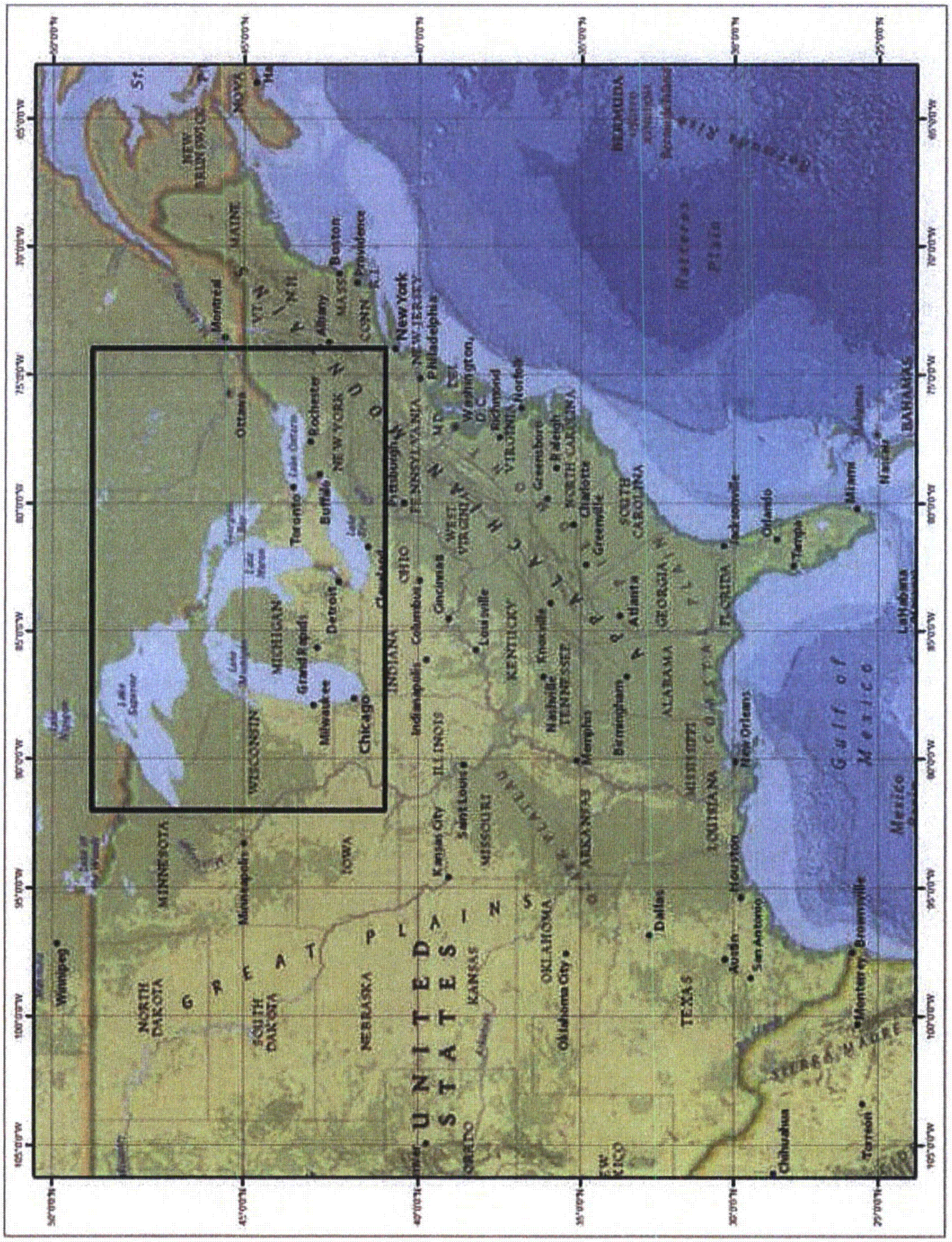


Figure 3-17: Lake Michigan Bathymetry Map

[Note: This figure is for illustration purposes and not pertinent to the technical content of this document.]

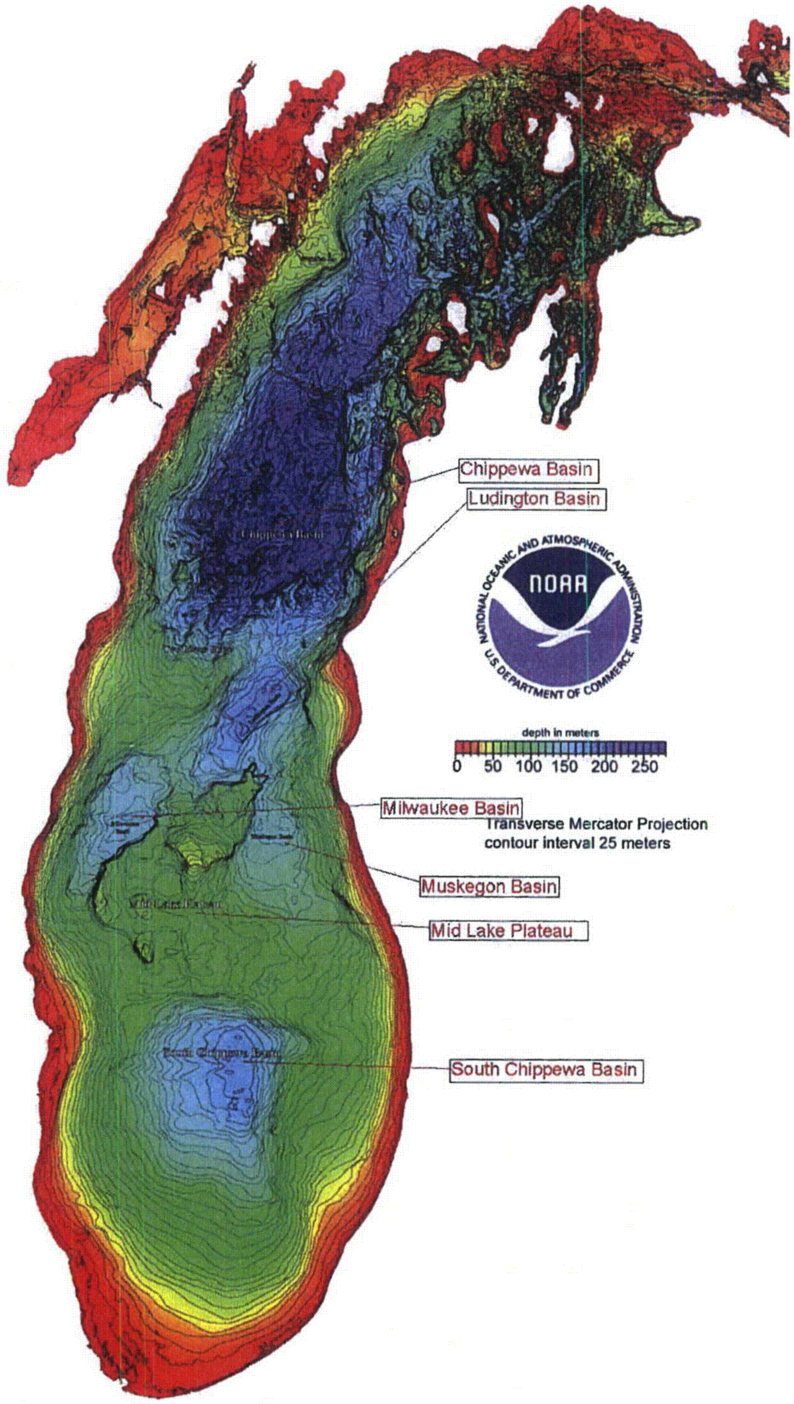


Figure 3-18: Selected Prominent Bathymetric Features

[Note: This figure is for illustration purposes and not pertinent to the technical content of this document.]

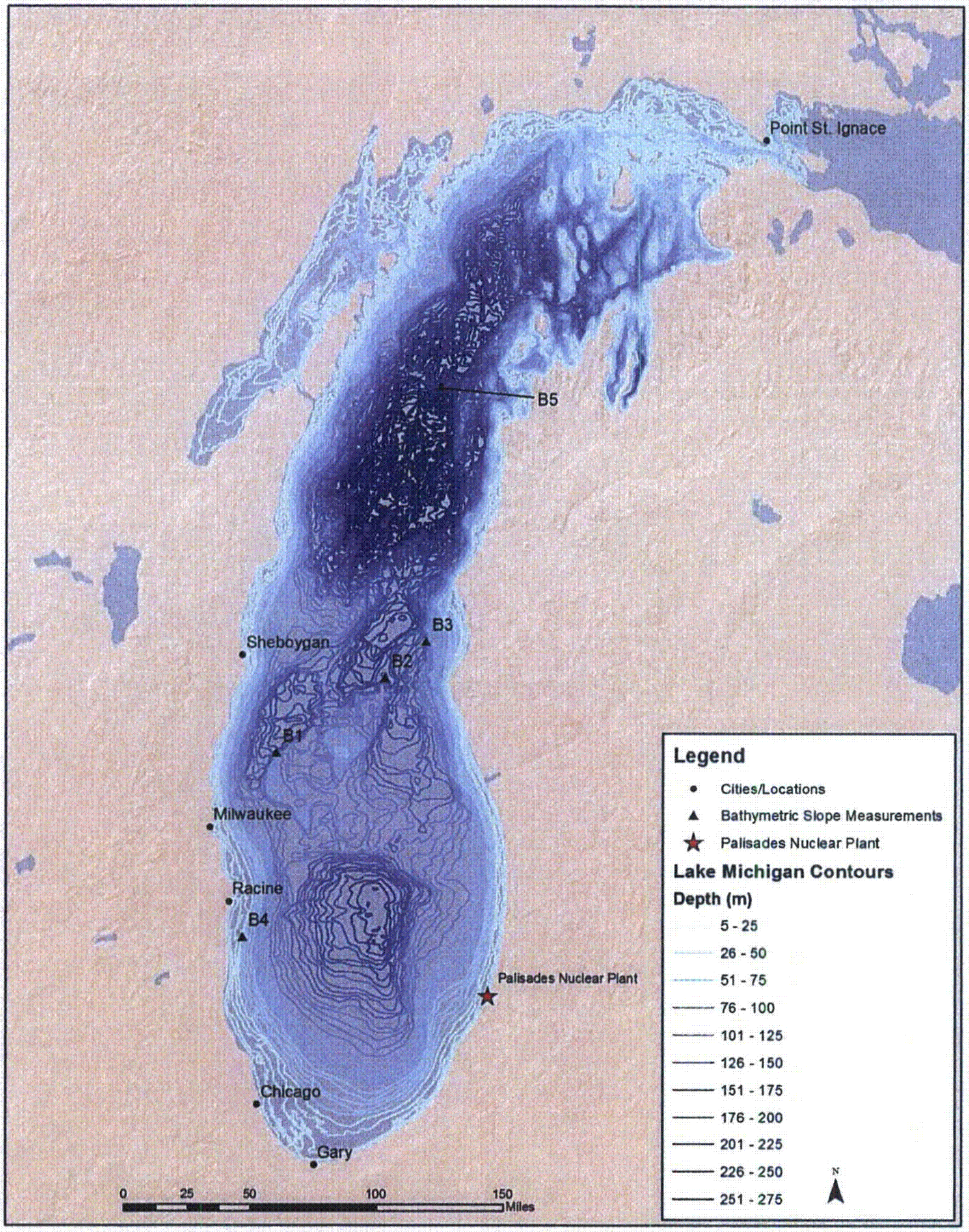
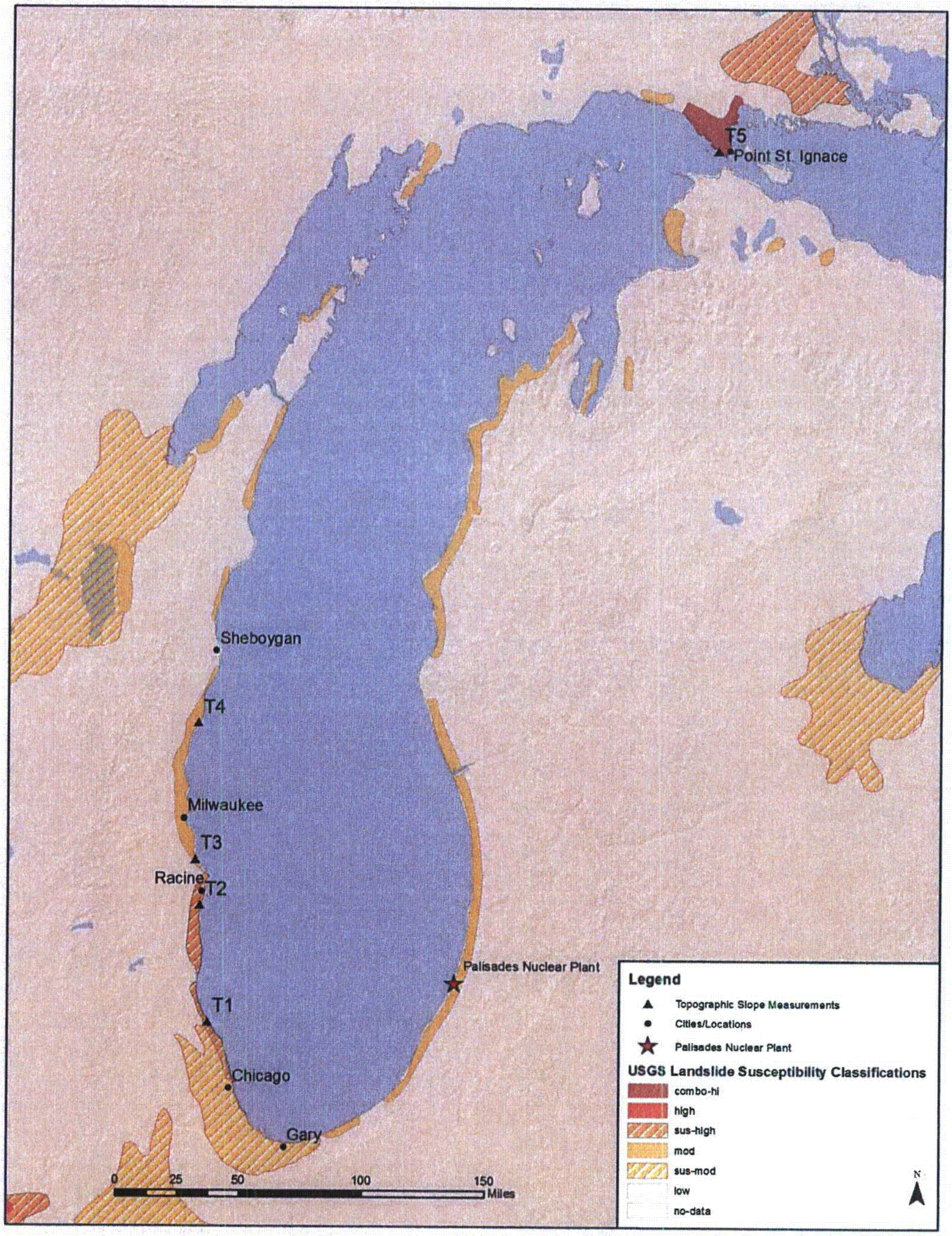


Figure 3-19: Selected Prominent Topographic Features

[Note: This figure is for illustration purposes and not pertinent to the technical content of this document.]



[Page replacement: See AREVA Document No. 159-9236786-000]

3.7 Ice Induced Flooding

Ice jams and ice dams can form in rivers and streams adjacent to a site and may lead to flooding by two mechanisms (NRC 2011):

- Collapse of an ice jam or a 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 a dam downstream of a site may impound water upstream of itself, thus causing a flood via backwater effects.

In addition, although frazil ice is not related directly to flooding, NUREG/CR-7046 (NRC 2011) recommends that air temperature data for meteorological stations located near the site be collected since frazil ice can be a precursor to the formation of ice jams or ice dams. Frazil ice forms in supercooled, turbulent water that is free of ice and snow cover. For supercooling to occur, the air temperature usually must be 18 °F or lower (NRC 2011, Appendix G).

The following summarizes the ice induced flooding assessment performed for the PLP site.

3.7.1 Methodology

The HHA approach described in NUREG/CR-7046 (NRC 2011) was used for ice induced flooding. As such, historical data was reviewed to assess if the site vicinity is subject to ice induced flooding and a site assessment was performed using conservative, simplifying assumptions to develop a conservative estimate of the effects at the site from the corresponding, historically observed event (AREVA 2014).

3.7.2 Results

3.7.2.1 Regional Findings

A search of the U.S. Army Corps of Engineers (USACE) Ice Jams Database (USACE 2014) was performed and revealed one ice jam north of PLP along the shores of Lake Michigan near South Haven, Michigan and another south of PLP along the shores of Lake Michigan near Benton Harbor, Michigan. The city of South Haven is approximately 4.5 miles north of PLP and where the Black River enters Lake Michigan, and the city of Benton Harbor is about 16 miles to the south. Both ice jams were then cross checked with a text search. One result was produced for South Haven, but no result was produced for Benton Harbor; therefore, a text search was performed for the town of St. Joseph, Michigan, which is adjacent to Benton Harbor. The historical ice jam on record for South Haven was for an event that occurred at PLP in February 2003 in which an offshore submerged intake was partially blocked by frazil ice. The St. Joseph historical ice jam occurred on the St. Joseph River in January 2005, causing flooding of some roads and homes in the city of St. Joseph.

Air temperature data summaries collected at the Muskegon, Michigan meteorological station, which is on the same (eastern) shore of Lake Michigan as the PLP site, were also reviewed and indicated that daily mean air temperatures were sometime near or below 18 °F, typically in the months of January and February for the period of 1977 to 1983 (NSIDC 2014). The average temperature for Muskegon for the months of January and February was also about 18 °F in 1982, 1994 and 2014 (NCDC 2014).

The Great Lakes systems, consisting of the five Great Lakes (i.e., Lakes Superior, Michigan, Huron, Erie and Ontario), Lake St. Clair and connecting channels (i.e., the St. Marys, St. Clair, Detroit and Niagara Rivers) is naturally regulated due to the large surface area of the lakes and the constricted outlets of the connecting channels/rivers (Derecki 1986). During the 1983-84 winter season, severe cold weather produced significant ice on Lake Huron which was subsequently forced down the St. Clair River by northerly winds, resulting in a record

ice jam (i.e., the largest ice jam in the historical record going back to 1900) that lasted 24 days in April of 1984. The change in water level on Lakes Huron and Michigan due to the ice jam was calculated to be an increase of 0.20 feet. Computer simulations at that time estimated that it would take approximately one year for most and three years (i.e., through 1987) for all excess water stored in Lakes Michigan and Huron during the St. Clair Ice Jam to dissipate and return to pre-ice jam lake levels (Derecki 1986).

Considering that tributaries to Lake Michigan are prone to ice formation, the Lake Michigan shoreline is prone to frazil ice production and ice jams can occur on connecting channels/rivers for the Great Lakes, a site assessment was subsequently conducted.

3.7.2.2 Site Findings

There are no rivers adjacent to PLP and the Black River and the St. Joseph River are approximately 4.5 miles and 16 miles, respectively from PLP. Similarly, there are no streams adjacent to PLP. PLP site drainage is independent of the Brandywine Creek drainage basin.

As noted in Section 3.7.2.1 above, although frazil ice has occurred at PLP, the development of frazil ice is prevented by discharging warm water recirculation pump flow into PLP's two intake bays, upstream from the trash racks and traveling screens. The warm water flow mixes with the intake water to heat it during conditions that promote the development of frazil ice (PLP 2012, Section 9.12.2.1).

Since ice jams have occurred in the site vicinity, a hypothetical ice jam at PLP was postulated. To estimate the maximum surface ice thickness that could form, accumulated freezing degree-day (AFDD) data was obtained. Freezing degree-days accumulated at a specific location are defined as the differences between mean daily air temperatures and the freezing point of water (32 °F). For Muskegon, Michigan, the maximum seasonal AFDD value was 1406 from April 1904 (NOAA 2014a and NOAA 2014b, Table 2.9). Using the modified Stefan equation presented by the U.S. Army Corps of Engineers, surface ice thickness was estimated as a function of AFDD as follows:

Ice Thickness (in), $t = C(AFDD)^{0.5}$ (USACE 2004)

Where: t = ice thickness, in inches

C = coefficient for water body size, wind conditions and snow cover. The 'C' value ranges from 0.12 to 0.8 with a usual range between 0.3 and 0.6.

AFDD = accumulated freezing degree-days, in °F

Using a conservative 'C' value of 0.8, representing a windy lake with no snow, and the maximum AFDD of 1406, gives an estimated ice thickness of 2.5 feet. Conservatively assuming that the estimated maximum ice thickness of 2.5 feet melts and is impounded within the intake bays when Lake Michigan is at its highest monthly mean stage lake level of 583.2 feet MSL, the level of impounded water would be at elevation 585.7 feet MSL, which would be 3.3 feet below the plant's shoreline grade at elevation 589 feet MSL (PLP 2012, Sections 2.2.2 and 5.4.1.1). In the event that the maximum estimated ice thickness of 2.5 feet was to develop in the intake bays when the lake is at its historic minimum level of 576.9 feet MSL (IDNR 2014), the available water level would be at an elevation of 574.4 feet MSL. Considering that the PLP Service Water System is not anticipated to be adversely impacted unless the level of Lake Michigan falls below the top of the sluice gate opening at elevation 568.25 feet (PLP 2014), a margin of 6.15 feet would remain for flow passage.

3.7.3 Conclusions

Based on historical records and a hypothetical ice jam, ice induced flooding at PLP is not likely to impact SSCs important to safety considering the following:

- The nearest historical ice jam on record occurred on the St. Joseph River, which is 16 miles from the PLP site.
- There are no streams adjacent to PLP in which ice jams or ice dams could form and impact PLP. Site drainage is independent of the Brandywine Creek drainage basin.
- The largest, historical ice jam on the Great Lakes system did not result in flooding at PLP.
- Although frazil ice formation has occurred at PLP, frazil ice does not directly result in flooding.
- A potential rise in lake level due to ice melt would be below the site's shoreline elevation.
- Ice formation on the lake would still allow for sufficient service water flow.
- Ice formation on the lake will not lead to flooding conditions at the PLP site.

3.7.4 References

AREVA 2014. AREVA Document No. 51-9224838-000, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Screening for Ice Induced Flooding, August 2014.

Derecki, 1986. Record St. Clair River Ice Jam of 1984, Jan A. Derecki and Frank H. Quinn, Paper No. 21103, Part of the Journal of Hydraulic Engineering, Vol. 112, No. 12, December, 1986; Available at: http://www.glerl.noaa.gov/pubs/fulltext/1986/19860008.pdf; Date accessed August 4, 2014. (See AREVA Document No. 51-9224838-000)

IDNR 2014. Indiana Department of Natural Resources, Lake Level Fluctuations, Lake Michigan Lake LevelGraph 1960-2014; Available at: http://www.in.gov/dnr/water/files/02LakeLevelGraph1960-2014.pdf; Date accessed July 9, 2014. (See AREVA Document No. 51-9224838-000)

NCDC 2014. National Climatic Data Center, Climate at a Glance for Muskegon, Michigan; Available at: http://www.ncdc.noaa.gov/cag/; Date accessed June 27, 2014. (See AREVA Document No. 51-9224838-000)

NOAA 2014a. Great Lakes Monthly and Seasonal Accumulations of Freezing Degree-Days – Winters 1898-2002, Raymond A. Assel, NOAA Technical Memorandum GLERL-127, National Oceanic and Atmospheric Administration – Great Lakes Environmental Research Laboratory; Available at: http://glerl.noaa.gov/ftp/publications/tech_reports/glerl-127/tm-127.pdf; Date accessed June 18, 2014.

(See AREVA Document No. 51-9224838-000)

NOAA 2014b. Great Lakes Monthly and Seasonal Accumulations of Freezing Degree-Days – Winters 1898-2002, Raymod A. Assel, NOAA Technical Memorandum GLERL-127, National Oceanic and Atmospheric Administration – Great Lakes Environmental Research Laboratory, Table 2.9 – Seasonal FDD Accumulation on BOM Dates at Muskegon, Michigan; Available at: http://www.glerl.noaa.gov/ftp/publications/tech_reports/glerl-127/Tables/Table2.pdf; Date accessed June 18, 2014. (See AREVA Document No. 51-9224838-000)

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

NSIDC 2014. National Snow and Ice Data Center, Muskegon, Michigan, Average Temperature, January-February; Available at: ftp://sidads.colorado.edu/pub/DATASETS/NOAA/G00801/msk83.dat; Date accessed July 8, 2014. (See AREVA Document No. 51-9224838-000)

PLP 2012. Palisades Nuclear Plant Final Safety Analysis Report (FSAR), Revision 30, September 4, 2012. (AREVA Document No. 38-9223712-000)

PLP 2014. Palisades Nuclear Plant, Limiting Conditions of Operation, Section 3.7.9, Ultimate Heat Sink (UHS), revised April 14, 2011. (AREVA Document No. 38-9223712-000)

USACE 2004. U.S. Army Corps of Engineers, Ice Engineering, ERDC/CRREL Technical Note 04-3, June 2004. (See AREVA Document No. 51-9224838-000)

USACE 2014. U.S. Army Corps of Engineers, Ice Jams Database, Ice Engineering Research Group, Cold Regions Research and Engineering Laboratory; Available at: http://icejams.crrel.usace.army.mil/; Date accessed June 13, 2014. (See AREVA Document No. 51-9224838-000)

3.8 Channel Migration or Diversion

Natural channels may migrate or divert either away from or toward the site. The relevant event for flooding is the diversion of water towards the site. There are no well-established predictive models for channel diversions. Therefore, it is not possible to postulate a probable maximum channel diversion event. Instead, historical records and hydro-geomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander towards the site (NRC 2011, Section 3.8).

This section summarizes the Channel Diversion evaluation performed in AREVA Document No. 51-9226164-000 (AREVA 2014).

3.8.1 Methods

The channel diversion flooding evaluation followed the HHA approach described in NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America (NRC 2011).

With respect to channel diversion, the following two steps were used for the HHA:

1. Channel diversion phenomena or mechanisms were identified by reviewing historical and hydro-geomorphological data and assessing the effects of the phenomena in the site region.
2. A conservative estimate of the effects at the site from historical and hydro-geomorphological data using conservative simplifying assumptions was developed.

3.8.2 Results

There are no channels, rivers or streams adjacent to the PLP site; therefore, channel diversion of Lake Michigan and the nearest stream to PLP, Brandywine Creek (USGS 1981 and USGS 2011), was evaluated.

3.8.2.1 Lake Michigan

The Lake Michigan shoreline is constantly reshaped by the effects of wind, waves and moving water. Flat, low-lying shorelines are susceptible to flooding and high bluff areas are prone to erosion. Erosion naturally occurs during periods of low, average or high water levels (USACE 1999). In the vicinity of PLP, the shoreline consists of rolling sand dunes which extend inland about 5,000 feet for approximately two miles north and five miles south of the PLP site. Although the lake's shoreline forms the west side of the PLP site, there are no readily apparent signs of shoreline erosion on Lake Michigan in the vicinity of the PLP site based on topography circa the mid-1960s to 2011 (PLP 2012, Figure 2-3, USGS 1981 and USGS 2011). As shown on Figure 3-20 and Figure 3-21, the absence of significant shoreline erosion is apparent on the 1981 and 2011 topographic maps at a 1:24,000 scale, by the close similarity of shoreline configuration maintained during the intervening 30 years.

However, the sand dunes situated along the shore of Lake Michigan, south of PLP, are indicated to be 'High Risk Erosion Areas' (MDEQ 1996). High Risk Erosion Areas are shore-lands where recession of the zone of active erosion has been occurring at a long-term average rate of one foot or more per year over a minimum period of 15 years. The determination of high risk erosion areas was initially conducted by the Michigan Department of Environmental Quality (MDEQ) between 1980 and 1986. Recession rate studies were subsequently updated on a county-by-county basis to incorporate changing shoreline conditions. Hence, it was concluded that erosion of the lake's shoreline may divert lake water towards PLP, potentially resulting in flooding.

As such, the stability of the PLP shoreline was subsequently evaluated. Although there are no dedicated flood or sea walls, the PLP site is stabilized against shoreline erosion from wind, currents and water fluctuations with stone riprap installed north and south of the Discharge Structure, along the plant's beach front (PLP 2014a through PLP

2014f). The riprap slope cover consists of a one foot thick layer of one to two inch diameter aggregate placed on filter cloth. The aggregate layer is overlaid by a one and a half foot thick layer of six to twelve inch diameter stone which is overlaid by a three foot layer of two to six ton Armor stone. The riprap cover provides protection against erosive forces and protects shoreline structures by preventing the erosion or failure of the underlying soils. In addition, the Discharge Structure and mixing basin/channel are constructed of concrete and steel sheet piling (PLP 2014g through PLP 2014i), and inspections of the Discharge Structure and its components are performed following unusual events (i.e., an earthquake, tornado) (PLP 2012, Section 9.12.3.2). Therefore, it is unlikely that the shoreline at PLP will experience changes due to shoreline erosion processes that would divert lake water towards the PLP site and impact components important to safety.

3.8.2.2 Brandywine Creek

Brandywine Creek and its tributaries drain an approximate 17 square mile area and flow into Lake Michigan through a gap in the dunes approximately 3,000 feet south of PLP. The deposits of the Brandywine Creek drainage basin are of low permeability, resulting in almost all of its runoff being to Lake Michigan. Due to sand dunes north, east and south of the PLP site, having an average width of 5,000 feet, the Brandywine Creek drainage basin is independent of PLP site drainage (PLP 2012, Sections 2.2 and 2.3).

In addition, Brandywine Creek shows no apparent signs of diversion based on review of the 1981 and 2011 site area topographic maps. The creek's pathway appears to be the same in both maps (USGS 1981 and USGS 2011), and the description of Brandywine Creek in the PLP FSAR, circa the mid-1960s, also appears similar to its depiction on the topographic maps. Therefore, based on available historical and topographical characteristics of the site region, it was concluded that flooding due to diversion of Brandywine Creek is not likely to impact the PLP site.

3.8.3 Conclusions

Since there are no perennial streams within the PLP watershed, flooding at PLP due to channel diversion is not possible. In addition, based on hydro-geomorphological data for the region and considering that the plant's shoreline was not identified as a high-risk erosion area, there is very limited potential for diversion of the Lake Michigan shoreline at the PLP site. Furthermore, a shoreline protection system at the plant has been effective in stabilizing the site's beach front since cooling towers were constructed in 1975.

3.8.4 References

AREVA 2014. AREVA Document No. 51-9226164-000, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Screening for Channel Diversion, August 2014.

MDEQ 1996. High Risk Erosion Areas and Critical Dune Areas Map, Covert Township, Michigan, Michigan Department of Environmental Quality, Amended February 29, 1996; Available at: http://www.michigan.gov/deq/0,4561,7-135-3307_3331-107407--,00.html; Date accessed July 16, 2014. (See AREVA Document No. 51-9226164-000)

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

PLP 2012. Palisades Nuclear Plant Final Safety Analysis Report (FSAR), Revision 30, September 4, 2012. (AREVA Document No. 38-9223712-000)

PLP 2014a. Palisades Drawing C-2001, Sheet 2, Rev. 1, Shore Protection GWO 1838 South Section Palisades Plant. (AREVA Document No. 38-9223712-000)

PLP 2014b. Palisades Drawing C-2001, Sheet 3, Rev. 2, Shore Protection GWO 1838 South Section Palisades Plant. (AREVA Document No. 38-9223712-000)

PLP 2014c. Palisades Drawing C-2002, Sheet 1, Rev. 0, Shore Protection North Shore. (AREVA Document No. 38-9223712-000)

PLP 2014d. Palisades Drawing C-2002, Sheet 2, Rev. 0, Shore Protection North Shore Existing Beach Plan and Cross Sections. (AREVA Document No. 38-9223712-000)

PLP 2014e. Palisades Drawing C-2002, Sheet 3, Rev. 0, Shore Protection South Shore. (AREVA Document No. 38-9223712-000)

PLP 2014f. Palisades Drawing C-2002, Sheet 4, Rev. 0, Shore Protection South Shore Existing Beach Plans and Cross Section. (AREVA Document No. 38-9223712-000)

PLP 2014g. Palisades Drawing C-31, Rev. 11, Discharge Structure. (AREVA Document No. 38-9223712-000)

PLP 2014h. Palisades Drawing C-32, Rev. 2, Circulating Water Discharge Channel. (AREVA Document No. 38-9223712-000)

PLP 2014i. Palisades Drawing C-440, Rev. 6, Discharge Channel Modifications Plan & Sections. (AREVA Document No. 38-9223712-000)

USACE 1999. Living with the Lakes: Understanding and Adapting to Great Lakes Water Level Changes, U.S. Army Corps of Engineers Detroit District and Great Lakes Commission, 1999. (See AREVA Document No. 51-9226164-000)

USGS 1981. Covert, Michigan Topographic Quadrangle Map, SE/4 South Haven 15' Quadrangle 1981, Scale 1:24 000, U.S. Geological Survey. (See AREVA Document 51-9226164-000)

USGS 2011. Covert Topographic Quadrangle Map, Michigan 2011–Van Buren County, 7.5 Minute Series, Scale 1: 24 000, U.S. Geological Survey. (See AREVA Document No. 51-9226164-000)

Figure 3-20: Covert, MI USGS Topographic Quadrangle Map 1981

[Source: USGS 1981]

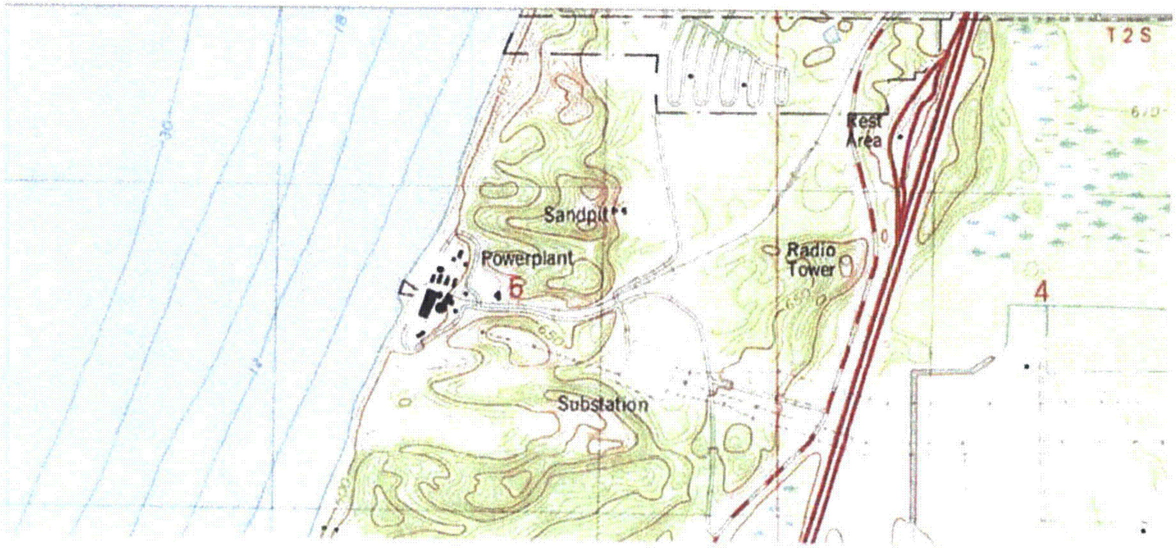


Figure 3-21: Covert, MI USGS Topographic Quadrangle Map 2011

[Source: USGS 2011]

3.9 Combined Effect Flood

This section addresses the combined effect flood at PLP. This evaluation includes consideration of the impacts of 1) the Probable Maximum Storm Surge (PMSS) on Lake Michigan, which includes the design antecedent water level; and 2) wave effects associated with the Probable Maximum Wind Storm (PMWS), which includes wave setup and wave runup. Other combined effect flood scenarios were assessed and screened out as not applicable at PLP.

This section summarizes the Combined Effect Flood evaluation performed in the AREVA Calculation "Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Combined Events" (AREVA 2015).

3.9.1 Methodology

The criteria for assessing combined events are provided in NUREG/CR-7046, Appendix H (NRC 2011). Of the five scenarios presented, one applies to PLP: Floods along shores of enclosed water bodies (Scenario H.4). Other combined effect flood scenarios described in NUREG/CR-7046 were screened out as not applicable to PLP. The flooding impact of the Scenario H.4 combined events flood mechanism was assessed either qualitatively or quantitatively, as described below.

Scenario H.4: In consideration of the site location on the shore of Lake Michigan, the H.4 combined flood event sub-scenario that is applicable to the site is:

Shore Location, which combines:

- 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.

The "streamside location" sub-scenario of H.4 does not apply.

The methodology used to evaluate the H.4 - Shoreside combined flood event at PLP consisted of the following steps:

1. Review of the USACE Wave Information Studies (WIS) for comparison to the simulated offshore, deepwater wave heights and periods. Historic wave data from two WIS stations near PLP, Station 94436 and Station 94437, were compiled to provide a comparison to simulated deepwater wave heights and periods.
2. Development of the deepwater waves resulting from the PMWS using the Advanced Circulation (ADCIRC) model of Lake Michigan developed in the PMSS Calculation (AREVA 2014a) coupled with the Delft University of Technology's (DUT) Simulating Waves Nearshore (SWAN) model Version 41.01. The coupled ADCIRC+SWAN model uses the USACE ADCIRC mesh used and described in the PMSS calculation (AREVA 2014a) as the computational domain and bathymetry, as shown in Figure 3-22. The coupling of ADCIRC and SWAN involves an integrated modeling process that is illustrated in Figure 3-23. ADCIRC passes the wind velocities, water levels and currents to SWAN, which uses those values as forcing to its calculations (Dietrich et al., 2012). The coupled ADCIRC+SWAN model outputs water level that includes wave setup because the coupled ADCIRC model takes into account wave radiation stress output by SWAN. The combined storm surge and wave setup stillwater hydrograph was used as input to the nearshore/shallow water SWAN model (described below) as it accounts for wave setup. Deepwater wave spectra outputted at the nearshore model boundary at four points were also used as the incoming parametric spectra for the nearshore model.
3. Development of the nearshore and shallow-water waves near PLP resulting from the PMWS using the SWAN Version 41.01 model. This finer-resolution SWAN model was used to simulate the nearshore and shallow-

water wave action in the immediate vicinity of PLP. This separate SWAN model was necessary to resolve the topography and buildings present in the PLP site area. Results of the nearshore/shallow-water SWAN model were verified by comparing wave parameters at three points in the overlapping section of the deepwater and nearshore models.

SWAN model input included: 1) the computational domain, 2) definition of the bottom (depth), 3) definition of the wave conditions at the model boundaries, and 4) definition of the wind field. The bottom depth was specified based on Lake Michigan bathymetry (JALBTCX 2014 and NGDC 1996) and the topographic site survey (AREVA 2014b). Deepwater bathymetry data for the SWAN mesh was obtained from the National Oceanic and Atmospheric Administration's (NOAA) National Geophysical Data Center's (NGDC) Great Lakes Bathymetry dataset (NGDC 1996) and nearshore bathymetry data were obtained from USACE LiDAR data (JALBTCX 2014). Fine resolution of the mesh near the site allowed buildings to be represented by elevated nodes based on building roof elevations from the topographic site survey (AREVA 2014b). The circulation water pipes were modeled as elevated nodes consistent with buildings. It was not necessary to account for the opening underneath the pipes as SWAN models any elevations below the water surface elevation (i.e., combined stillwater elevation) as inundated. Therefore, the model simulates a hydraulic connection between both sides of the pipe for stillwater level communication but not for wave action transmission. Waves are blocked by the pipe.

A spatially-constant, time-varying wind field was applied on the SWAN grid. A three by three rectilinear grid with grid cell size of 0.2 degrees longitude by 0.04 degrees latitude was used as the wind input grid such that it covered the computational grid. The depth above the model bottom (i.e., bathymetry) due to the PMSS combined stillwater hydrograph including wave setup from the deepwater model was incorporated into the nearshore/shallow-water SWAN simulation for each model time step. The same input grid used for wind input was used for water level input. A spectral boundary was used to represent the interaction between the deepwater coupled ADCIRC+SWAN and nearshore SWAN simulations. Deepwater wave spectra outputted at the nearshore model boundary at four points were used as the incoming parametric spectra for the nearshore model. The exclusion of currents as input to the SWAN model was determined to be appropriate based on a sensitivity analysis of wave action to currents. Because the influence of currents on waves was insignificant at the test locations and the conservativeness of including current as an input could not be determined near the site (i.e., nearshore), water currents were neglected in the model.

4. Calculation of wind wave effects at 12 important locations at PLP using the waves predicted by the nearshore SWAN model for the PMSS, and using FEMA and American Society of Civil Engineers (ASCE)/SEI 7-10 methodology (FEMA 2011 and ASCE 2010). The corresponding maximum wave height and maximum breaking wave height (depth-limited wave height, referred to as maximum breaker height in NRC guidance) at these locations were calculated as per NRC guidance (NRC 2013). Maximum wave heights were computed as 1.67 times the significant wave height (based on the SWAN model). Depth-limited wave heights were computed as 0.78 times water depth. For depth-limited waves, the depth at each structure was determined based on the difference between the PMSS combined stillwater elevation (including wave setup) and ground elevation from the topographic site survey (AREVA 2014b). The controlling wave was selected as the lesser of the maximum wave or the depth-limited breaking wave. These wave crest heights above combined stillwater elevation were added to the PMSS stillwater elevation and wave setup from deepwater waves to calculate the elevation of wave action at the important points. Wave crest heights above stillwater were computed as 50 percent of the overall wave height for non-breaking conditions and 70 percent of the overall wave height for breaking wave conditions. Additionally, three transects were used to represent wave conditions where head-on impact is expected at the circulation water pipes, Discharge Structure and Feedwater Purity Building.

5. Calculation of standing wave crest elevations against vertical structures. Vertical walls cause a reflected or standing wave against the waterward (i.e., lake-facing) side of the wall (ASCE 2010). Specific wave runup estimation methodologies are not available in the literature for standing wave runup on vertical structures; therefore, the standing wave height was used to represent the height of runup on the lake-facing side of vertical structures. American Society of Civil Engineers (ASCE)/SEI 7-10 guidance states that this standing wave crest reaches a height above combined stillwater of 1.2 times the depth at the wall, as shown in Figure 3-24 (ASCE 2010). This relationship assumes depth-limited conditions (ASCE 2010), which is conservative because the depth-limited wave is the maximum wave height possible for a given depth. As previously stated, the use of depth-limited wave height (i.e., maximum breaker height) is also consistent with NRC guidance (NRC 2013). This methodology was therefore used for this calculation to determine the standing wave height and maximum combined event water surface elevation on the lake-facing side of structures not otherwise shielded from wave action. The standing wave height was added to the PMSS combined stillwater elevation and wave setup from deepwater waves to calculate the maximum combined water surface elevation at the exposed, lake-facing external walls of the structures.
6. Calculation of the combined effect water surface elevations at important locations was completed by adding the PMSS stillwater elevation to the wave setup and the controlling wind wave effect (i.e., maximum wave height, breaking wave height, or standing wave height) at each important location.
7. Development of hydrostatic, hydrodynamic, and wave loads impacting the circulation water pipes during the PMSS due to wind-wave activity was also performed.

3.9.2 Results

3.9.2.1 Potential Shoreside Location on Enclosed Waterbody Combined Event

3.9.2.1.1 Review of Historical Wave Data

Hindcasts of deepwater significant wave heights resulting from historical storms range from 17.3 to 22.0 feet and the range of peak periods is 9.3 to 11.2 seconds for the top ten wave events reported at the WIS stations (USACE 2010). See Table 3-10. The WIS stations provide a good indication of deepwater wave conditions offshore of PLP. Because they are in deeper water than the SWAN output points discussed below, the wave height provided by WIS would become depth limited as they approach shore. However, because period is invariant, it can be compared to the shallow water wave periods predicted by SWAN.

3.9.2.1.2 Offshore Wave Results

Peak significant wave heights and periods for the coupled ADCIRC+SWAN model output locations are shown in Table 3-11. The peak wave setup simulated near PLP was 0.8 feet. Combining the wave setup of 0.8 feet with the PMSS stillwater elevation of 593.1 feet NGVD29 results in a combined stillwater elevation of 593.9 feet NGVD29 near PLP. At the peak of the PMWS, the significant deepwater wave height varies from 19.8 to 21.6 feet across the four boundary output locations. The associated peak spectral wave period associated with the significant wave height is 17.3 to 17.7 seconds at the output points.

Simulated largest significant wave heights and wave periods were compared to published data at the WIS stations to determine the conservativeness of the model. The 100-year significant wave height at station 94436 was 25.3 feet (USACE 2010). The coupled ADCIRC+SWAN output at that location (longitude -86.40, latitude 42.36) was 45.9 feet, which is 20.6 feet higher than the 100-year return period WIS hindcast data. The large difference is the result of the much higher return period storm (i.e. the PMWS) that was simulated and therefore, the coupled ADCIRC+SWAN simulation is considered conservative. The largest fetch-limited deepwater wave that could be

generated near PLP was calculated using nomograms contained in the Coastal Engineering Manual (USACE 2008). The maximum wave is approximately 49 feet which is based on a fetch of 217 miles and a wind speed of approximately 89 mph (the maximum wind speed near PLP). This compares well with the simulated deepwater wave height and shows the conservatism present in the coupled ADCIRC+SWAN simulation.

3.9.2.1.3 Nearshore Wave Results

Nearshore and shallow waves in the vicinity of PLP were simulated using the SWAN model. The nearshore/shallow-water SWAN grid extent and elevations are shown in Figure 3-25. Figure 3-26 shows the time-varying water level which was output from the deepwater coupled ADCIRC+SWAN model and was used as input to the nearshore/shallow-water SWAN simulation. Figure 3-27 shows the time-varying wind speed and wind direction which was used as input to the SWAN simulation. Output deepwater wave characteristics from the coupled ADCIRC+SWAN simulation were specified at seven locations offshore of PLP, shown in Figure 3-28. The output locations include point V2 (longitude -86.320234, latitude 42.324251), which is the same location where surge results were reported in the PLP PMSS calculation (AREVA 2014a).

Output of wave characteristics was specified at 12 output nodes which were representative of important locations and structures at PLP shown on Figure 3-29 and Table 3-12. Outputs from the nearshore/shallow-water SWAN simulation include characterization of shallow-water wave conditions in the vicinity of the PLP. The peak significant wave heights occur one hour after the time peak surge stillwater elevation but coincident with the time of peak wind speed at simulation hour 41. Peak significant wave heights are shown in Figure 3-30. The areas between circulation water pipes / Discharge Structure and the Turbine Building are sheltered from wind and external wave activity that cannot be accurately modeled using SWAN. These sheltered areas are not subject to wind-wave activity. Peak significant wave heights and periods for the output locations ranged from 0.0 feet (in sheltered areas) to 1.4 feet as shown in Table 3-12.

The fraction of breaking waves due to depth-induced breaking was output from SWAN and is shown in Figure 3-31. Large deepwater waves break along the beach and the sheet pile walls that define the Discharge Channel before reaching the site. Shoreward structures are located a distance of 50 to 70 feet away from the breaking zone and are therefore protected from the larger offshore waves. Waves that are smaller than the significant wave height may pass through this breaking zone without breaking. Additional wave growth inside the breaking zone occurs from wind. Unbroken waves within the plant area, which is inside the breaking zone, are therefore significantly smaller than waves on the open lake, as shown in Figure 3-30.

3.9.2.1.4 Incident Wave Characteristics

Wave conditions were determined at the 12 important locations in the PLP site area. Maximum wave heights and breaking (i.e. depth-limited) wave heights are reported in Table 3-12. The controlling (i.e. lesser of maximum or breaking) wave heights range from 0.0 (in sheltered areas) to 2.3 feet. The controlling wave crest elevations range from 593.9 (in sheltered areas) to 595.0 feet NGVD29.

It was assumed that the circulation water pipes will continue to serve as a barrier to wave action while subjected to the combined flood, wave, and debris loading coincident with the combined event flood (AREVA 2015). Therefore, although water flows beneath the circulation water pipes, the nearshore waves that propagate from offshore break on the circulation water pipes and do not propagate further to the Screen House (also referred to as the Intake Structure). Wind-wave activity is not anticipated within the sheltered area between the pipes and the Screen House/Intake Structure and Turbine Building. Waves in the vicinity of the south door to the Screen House/Intake Structure move parallel to the structure and door and do not break. The wave crest elevation of these maximum waves is 594.2 feet NGVD29 which is below the elevation of components important to safety inside the building. Wave conditions at the Screen House/Intake Structure and circulation water pipes are

illustrated in Figure 3-32. Note that water is expected to reach a minimum elevation inside the Screen House/Intake Structure of 593.1 feet NGVD29 (PMSS without wave set-up) due to direct connection to the lake, but not to exceed the elevation of the top of the maximum wave outside the south door.

3.9.2.1.5 Standing Wave Height at Vertical Structures

Standing wave crest elevations were calculated at the lake-facing side of the first structure for the three transects shown in Figure 3-29. Standing wave crest elevations are presented in Table 3-13. Standing wave action on a vertical wall or similar structure is analogous to runup on sloped beach or revetment. The first structure of Transect 1 is the circulation water pipes. It was assumed that the circulation water pipes can withstand the flood, wave, and debris loads coincident with the combined event flood (AREVA 2015). The combined stillwater elevation of 593.9 feet NGVD29 is 1.9 feet above the bottom of the circulation water pipes present in front of the lake side of the Screen House/Intake Structure. Therefore, although water flows beneath the circulation water pipes, the nearshore waves from Lake Michigan create a standing wave and break on the circulation water pipes and do not propagate further inland to the Screen House/Intake Structure. See Figure 3-32 for an illustration of wave conditions around the Screen House/Intake Structure and pipes.

3.9.2.1.6 Combined Events Water Elevations at PLP

The combined events water surface elevations at important locations are presented in Table 3-14. All combined events flood water surface elevations were calculated based on the previously developed antecedent water level used in the PMSS calculation (AREVA 2014a), consistent with Appendix H of NUREG/CR-7046 (NRC 2011). An antecedent water level of 583.4 feet NGVD29, which is the 100 year lake elevation, was conservatively used. Based on a probable maximum surge height of 9.7 feet, the PMSS stillwater elevation was found to be 593.1 feet NGVD29. Adding the PMSS stillwater elevation of 593.1 feet NGVD29 to the wave setup value of 0.8 feet and the maximum wave crest height at the Screen House/Intake Structure south wall, the resultant maximum combined events flood water surface elevation is 594.2 feet NGVD29 at the Screen House/Intake Structure. Wave conditions at the Screen House/Intake Structure are shown in schematic form in Figure 3-32.

3.9.2.1.7 Structure Loading and Associated Effects

Flood loading on the Screen House/Intake Structure was considered. There is a potential for hydrodynamic flood loading at the Screen House/Intake Structure as it is subject to flow velocity from water conservatively up to elevation 594.2 feet. Wave action will be blocked by the circulation pipes lake-ward of the Screen House/Intake Structure and velocities will be reduced as flow passes through the limited openings below the pipes. The FSAR (PLP 2012, Section 5.4.1.2) states that the Intake Structure can withstand the dynamic effect of wave runup of approximately eight feet. Runup height is typically measured from stillwater elevation (stated as 594.1 feet in the FSAR), but if runup is conservatively measured from ground surface elevation in front of the Screen House/Intake Structure, which is conservatively taken as 589.0 feet NGVD29 (AREVA 2014b), then the maximum allowable run-up elevation on the Screen House/Intake Structure would be elevation 597.0 feet NGVD29. The stated capacity of the Screen House/Intake Structure to withstand dynamic water loading up to elevation 597.0 feet NGVD29 bounds the calculated combined effects elevation of 594.2 feet NGVD29 at the Screen House/Intake Structure. The Screen House/Intake Structure side wall are not subject to wave loading as they are either sheltered or have parallel, nonbreaking waves.

Loading on the circulation water pipes lakeward of the Screen House/Intake Structure was calculated. The total breaking wave load on the pipes was 4,460 pounds per linear foot of pipe applied at elevation 594.5 feet NGVD29 and the total breaking wave load on the saddles was 1,770 pounds applied at elevation 593.9 feet NGVD29. There is a potential for horizontal hydrodynamic drag loads and vertical uplift loads on the circulation water pipes as the flood wave flows underneath them. The calculated upper-bound flood velocity of 11.2 feet per second was

used to calculate a hydrodynamic drag load of 280 pounds per foot of pipe applied at elevation 593.0 feet NGVD29. The uplift load was calculated based on the standing wave crest elevation on the lake-ward side of the pipe and the combined stillwater elevation on the Screen House/Intake Structure side of the pipe. The uplift load was calculated to be 2,120 pounds per linear foot of pipe acting 3.3 feet from the lake-ward edge of pipe.

Buildings and structures at PLP may also be exposed to debris impact loads or loads imposed on a building (or structure) by objects carried by moving water. Debris impact loads were calculated to be 24,640 pounds at the circulation water pipes and the pipe saddles which is applied at elevation 593.9 feet NGVD29.

Groundwater in the vicinity of the site is generally controlled by the level of Lake Michigan (PLP 2012). The surge hydrograph is above PLP site grade for approximately 25 hours. Permeability data from the FSAR indicates that the sandy lake deposits under the dunes have a slow percolation rate. Because of the relatively short duration of flooding and slow percolation rate of the underlying soil, short term water level changes (i.e., storm surge) is unlikely to affect groundwater levels in the vicinity of PLP. NRC guidance states that the impact of sediment erosion and deposition should be considered when storm surge flood levels impinge on flood protection, SSCs important to safety and foundation materials (NRC 2013). The coastline near PLP is not within a high risk erosion area as defined by the Michigan Department of Environmental Quality (MIDEQ 1996).

3.9.3 Conclusions

The following summarizes the results and conclusions relative to combined effect flooding at PLP:

- In consideration of the PLP site location on the shore of Lake Michigan, the H.4 combined flood event sub-scenario, as defined in NUREG/CR-7046 Appendix H (NRC 2011), that is applicable to the site is the combination of the probable maximum surge and seiche with wind-wave activity and the appropriate antecedent water level (100-year lake water surface elevation).
- Wind wave effects are added on top of an antecedent water level of 583.4 feet NGVD29, which is the 100 year lake elevation, and a probable maximum surge height of 9.7 feet, resulting in a PMSS stillwater elevation of 593.1 feet NGVD29.
- Wave setup on the lake is 0.8 feet on top of the PMSS stillwater elevation, resulting in a general water level at the site of 593.9 feet NGVD29.
- The results of the wind wave calculations indicate that the controlling wave crest elevations in the PLP site area range from 593.9 (in sheltered areas where no additional wave action is expected) to 595.0 feet NGVD29.
- The maximum combined events flood elevation including wave action at the circulation water pipes is 598.6 feet NGVD29, which is a result of a standing wave generated by waves breaking against the pipes. Similar standing waves will occur at other structures with lakeward facing walls.
- The maximum combined events flood elevation including wave action at the Screen House/Intake Structure is 594.2 feet NGVD29, which occurs near the south side where small waves move parallel or away from the structure.
- The capacity of the Screen House/Intake Structure to withstand dynamic water loading up to elevation 597.0 feet NGVD29 bounds the calculated combined effects elevation of 594.2 feet NGVD29 at the Screen House/Intake Structure.

Because wind-wave activity during the PMSS impacts the circulation water pipes, they will be subject to hydrostatic, hydrodynamic and wave loads. The breaking wave load at the circulation water pipes was calculated

to be 4,460 pounds per linear foot of pipe applied at 594.5 feet NGVD29. The breaking wave load at the pipe saddles was calculated to be 1,770 pounds applied at 593.9 feet NGVD29. The hydrodynamic drag load on the pipes was calculated to be 244 pounds per linear foot of pipe applied at 593.0 feet NGVD29. The vertical uplift load on the pipes was calculated to be 2,120 pounds per linear foot of pipe applied 3.3 feet from the lake-ward edge of the pipe. The debris impact load at the circulation water pipes and pipe saddles was calculated to be 24,640 pounds applied at 593.9 feet NGVD29. An earthen berm, the circulation water pipes, the Discharge Structure, and associated features form a barrier to wave action along the shoreline of the southern portion of the plant. Therefore, although water flows beneath the circulation water pipes, the nearshore waves that propagate from offshore break on the circulation water pipes and do not propagate further to the Screen House/Intake Structure (AREVA 2015).

3.9.4 References

AREVA 2014a. Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Probable Maximum Storm Surge and Seiche, AREVA Document No. 32-9226962-000, 2014.

AREVA 2014b. Palisades Nuclear Plant Topographic Survey, AREVA Document No. 38-9226943-000, 2014.

AREVA 2015. Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Combined Events, AREVA Document No. 32-9226981-000, 2015.

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

Dietrich et al., 2012. Performance of the Unstructured-Mesh, SWAN+ADCIRC Model in Computing Hurricane Waves and Surge, J.C. Dietrich, S. Tanaka, J.J. Westerink, C.N. Dawson, R.A. Luettich Jr., M. Zijlema, L.H. Holthuijsen, J.M. Smith, L.G. Westerink, and H.J. Westerink, Journal of Scientific Computing, Volume 52, Issue 2, August 2012. (See AREVA Document No. 32-9226981-000)

ESRI 2014. Bing Maps Aerial, ESRI Map Service, Date modified: November 25, 2013, Date accessed: November 26, 2014. (See AREVA Document No. 32-9226981-000)

FEMA 2011. Coastal Construction Manual, 4th Edition, FEMA P-55, Federal Emergency Management Agency, August 2011.

JALBTCX 2014. 2008 US Army Corps of Engineers (USACE) National Coastal Mapping Program Topobathy Lidar: Lake Michigan, Joint Airborne Lidar Bathymetry Technical Center of eXpertise, National Oceanic Administration Ocean Service, Coastal Services Center, Published May 7, 2014. (See AREVA Document No. 32-9226981-000)

MIDEQ 1996. Covert Township: High Risk Erosion Areas & Critical Dune Areas, Michigan Department of Environmental Quality, Date published: June 15, 1994, Date amended: February 29, 1996. (See AREVA Document No. 32-9226981-000)

NGDC 1996. Bathymetry of Lake Michigan, National Oceanic and Atmospheric Administration, National Geophysical Data Center, Date published: January 1, 1996. (See AREVA Document No. 32-9226981-000)

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

NRC 2013. Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment, JLD-ISG-2012-06, Revision 0, U.S. Nuclear Regulatory Commission, January 4, 2013.

PLP 2012. Final Safety Analysis Report, Revision 30, 2012. (See AREVA Document No. 38-9223712-000)

USACE, 2008. Meteorology and Wave Climate, Coastal Engineering Manual EM 1110-2-1100, Part II, Chapter 2, U.S. Army Corps of Engineers, August 1, 2008.

USACE 2010. Wave Information Studies: Great Lakes, U.S. Army Corps of Engineers, Engineer Research and Development Center Coastal and Hydraulics Laboratory, December 2010.

Table 3-10: Summary of Extreme Wave Conditions at WIS Stations near PLP

Rank	WIS Station 94436		WIS Station 94437	
	Significant Wave Height (feet)	Peak Wave Period (seconds)	Significant Wave Height (feet)	Peak Wave Period (seconds)
1	22.0	11.0	21.6	11.0
2	21.6	11.1	21.4	11.1
3	19.2	10.2	18.8	10.2
4	18.7	11.0	18.6	11.1
6	18.6	10.3	18.4	10.4
7	18.3	10.2	18.1	10.2
8	18.2	11.2	18.0	11.2
9	17.7	9.4	17.4	10.3
10	17.7	9.6	17.3	9.3

Table 3-11: Coupled ADCIRC+SWAN Simulation Results

Output Location	Peak Wave Height (feet)	Wave Period (seconds)
B1	21.6	17.3
B2	21.4	17.4
B3	20.0	17.7
B4	19.8	17.7
V1	16.6	17.4
V2	16.7	17.3
V3	15.6	17.8

Table 3-12: Nearshore/Shallow Water SWAN Simulation Results

ID Number	Location	Depth (feet)	Peak Significant Wave Height (feet)	Wave Period (seconds)	Peak Significant Wave Crest Elevation (feet, NGVD29)	Maximum Wave Height (feet)	Maximum Wave Crest Elevation (feet, NGVD29)	Maximum Breaking Wave Height (feet)	Maximum Breaking Wave Crest Elevation (feet, NGVD29)
19	Screen House/Intake Structure Roll-Up (Door #14)	4.0	0.4	N/A	594.1	**0.7**	**594.2**	3.1	596.1
20	North Entrance to Screen House/Intake Structure (Door #33)	3.9	0	N/A	593.9	**0**	**593.9**	2.9	595.9
21	Turbine Building Laydown Area Access (Door #12)	4.4	0.5	N/A	594.2	**0.9**	**594.3**	3.4	596.3
22	Turbine Building Southwest Roll-Up Door (Door #13)	4.4	0.5	N/A	594.2	**0.9**	**594.3**	3.4	596.3
23	Diesel Generator Fuel Oil Tank T-10A Vent	0.7	0.5	1.9	594.1	0.8	594.3	**0.5**	**594.3**
25	Door to Transformer Yard from Feedwater Pumps (Door #11)	4.0	0.3	N/A	594.0	**0.5**	**594.2**	3.1	596.1
26	Turbine Building Southside Roll-Up Door to Transformer Yard (Door #10)	4.0	0.3	N/A	594.0	**0.5**	**594.2**	3.1	596.1
27	Post Tension Tunnel Hatch Door 10A	4.0	0.5	N/A	594.1	**0.8**	**594.3**	3.1	596.0
29	North Chained Double Door to Diesel Generators (Door #170)	4.1	0.8	19.3	594.3	**1.4**	**594.6**	3.2	596.1
30	Auxiliary Building Addition Entrance Door	4.1	0.8	19.3	594.3	**1.4**	**594.6**	3.2	596.1

ID Number	Location	Depth (feet)	Peak Significant Wave Height (feet)	Wave Period (seconds)	Peak Significant Wave Crest Elevation (feet, NGVD29)	Maximum Wave Height (feet)	Maximum Wave Crest Elevation (feet, NGVD29)	Maximum Breaking Wave Height (feet)	Maximum Breaking Wave Crest Elevation (feet, NGVD29)
33	Turbine Building North Entrance Door	4.0	0.4	N/A	594.1	**0.6**	**594.2**	3.1	596.1
230	South Stairwell (Service Bldg Addition) Across from Elevator (Door #123)	4.7	1.4	19.2	594.6	**2.3**	**595.0**	3.6	596.4

Notes:

1) See Figure 3-29 for locations of points. ID Numbers 29 and 30 are at the same location (i.e., Door #170). Referring to Table 3-1, note that locations 28, 34, 35 and 36 were not inundated by the PMSS (i.e., they are located on the east side/upper level of the power block).

2) Controlling (lesser of maximum or breaking) wave height/elevation indicated in **bold text**.

Table 3-13: Wave Conditions at Shore-Facing Structures

ID Number	Location	Depth (feet)	Standing Wave Crest Height at Wall (feet)	Peak Standing Wave Crest Elevation (feet, NGVD29)
R1	West (Shoreward) Circulation Water Pipe	3.9	4.7	598.6
R2	West (Shoreward) Wall of Discharge Structure	6.9	5.4	602.2
R3	West (Shoreward) Wall of Feedwater Purity Building	4.0	3.1	598.7

Note: See Figure 3-29 for locations of points.

Table 3-14: Combined Effect Water Elevations

ID Number	Location	Combined Event Water Elevation (feet NGVD29)
19	Screen House/Intake Structure Roll-Up (Door #14)	594.2
20	North Entrance to Screen House/Intake Structure (Door #33)	593.9
21	Turbine Building Laydown Area Access (Door #12)	594.3
22	Turbine Building Southwest Roll-Up Door (Door #13)	594.3
23	Diesel Generator Fuel Oil Tank T-10A Vent	594.3
25	Door to Transformer Yard from Feedwater Pumps (Door #11)	594.2
26	Turbine Building Southside Roll-Up Door to Transformer Yard (Door #10)	594.2
27	Post Tension Tunnel Hatch Door 10A	594.3
29	North Chained Double Door to Diesel Generators (Door #170)	594.6
30	Auxiliary Building Addition Entrance Door	594.6
33	Turbine Building North Entrance Door	594.2
230	South Stairwell (Service Bldg Addition) Across from Elevator (Door #123)	595.0
R1	West (Shoreward) Circulation Water Pipe	598.6
R1	West (Shoreward) Wall of Screen House/Intake Structure	593.9
R2	West (Shoreward) Wall of Discharge Structure	602.2
R3	West (Shoreward) Wall of Feedwater Purity Building	598.7

Note: Combined Event Water Elevation is the controlling (lesser) of the breaking wave crest, maximum wave crest within the PLP site area, or the standing wave crest elevation on lake-facing structures.

Figure 3-22: USACE Lake Michigan ADCIRC Model Mesh Elevations

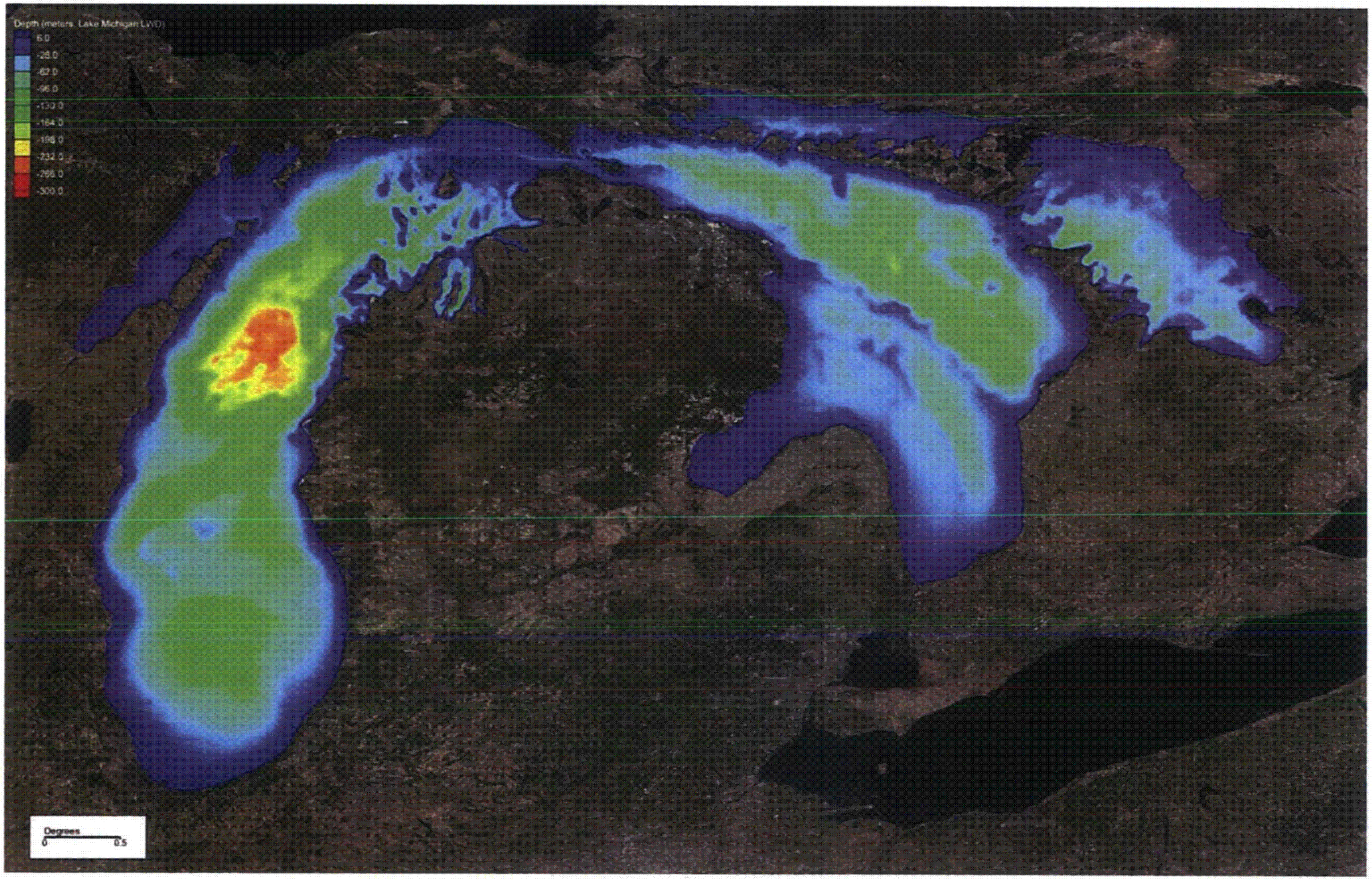


Figure 3-23: Coupled ADCIRC+SWAN Computation

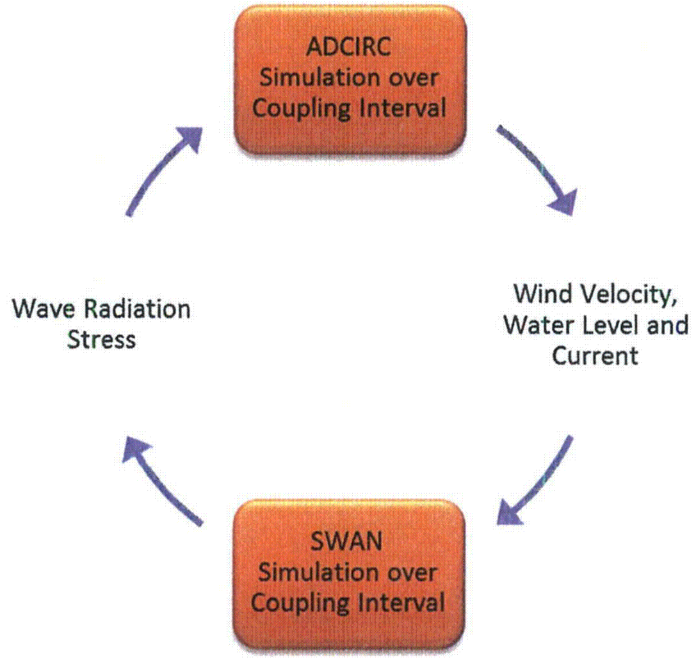
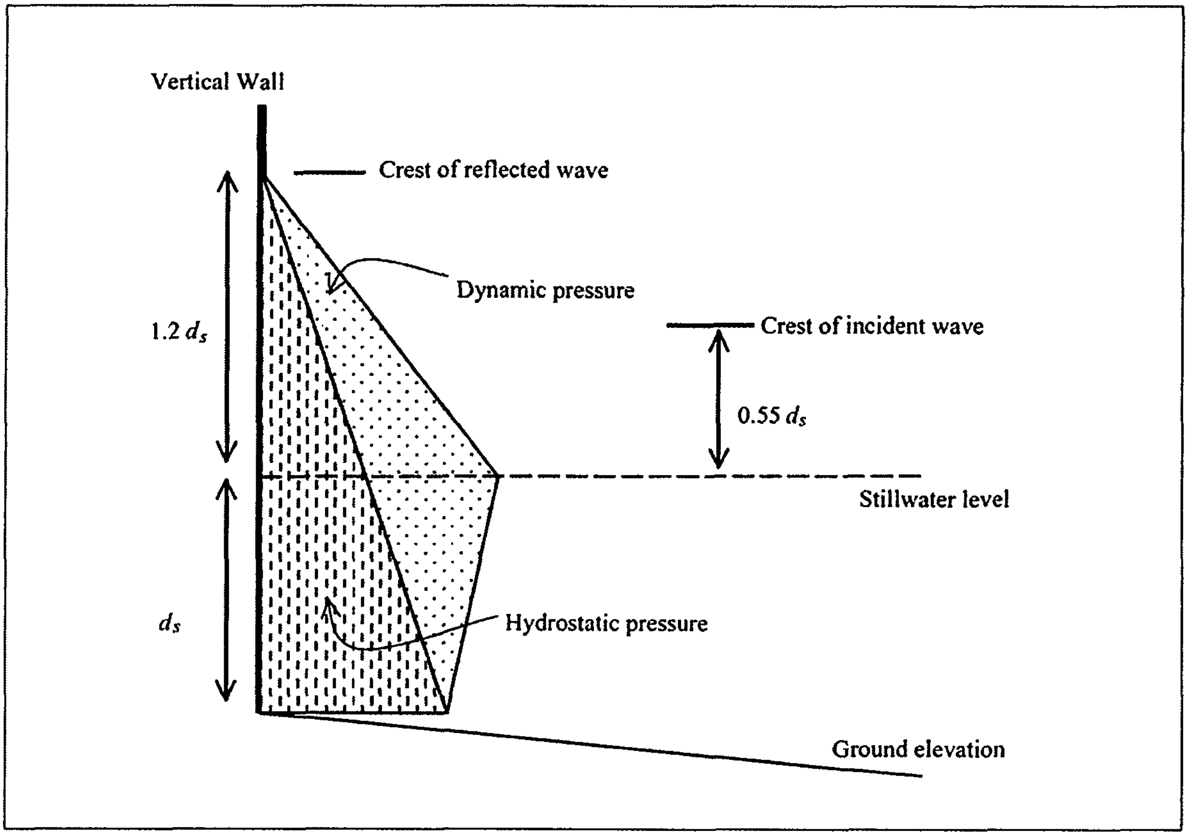


Figure 3-24: Normally Incident Breaking Wave Pressures against a Vertical Wall



Source: (ASCE 2010)
Note: Space behind the vertical wall is dry.

Figure 3-25: Nearshore/Shallow Water SWAN Simulation Model Elevations

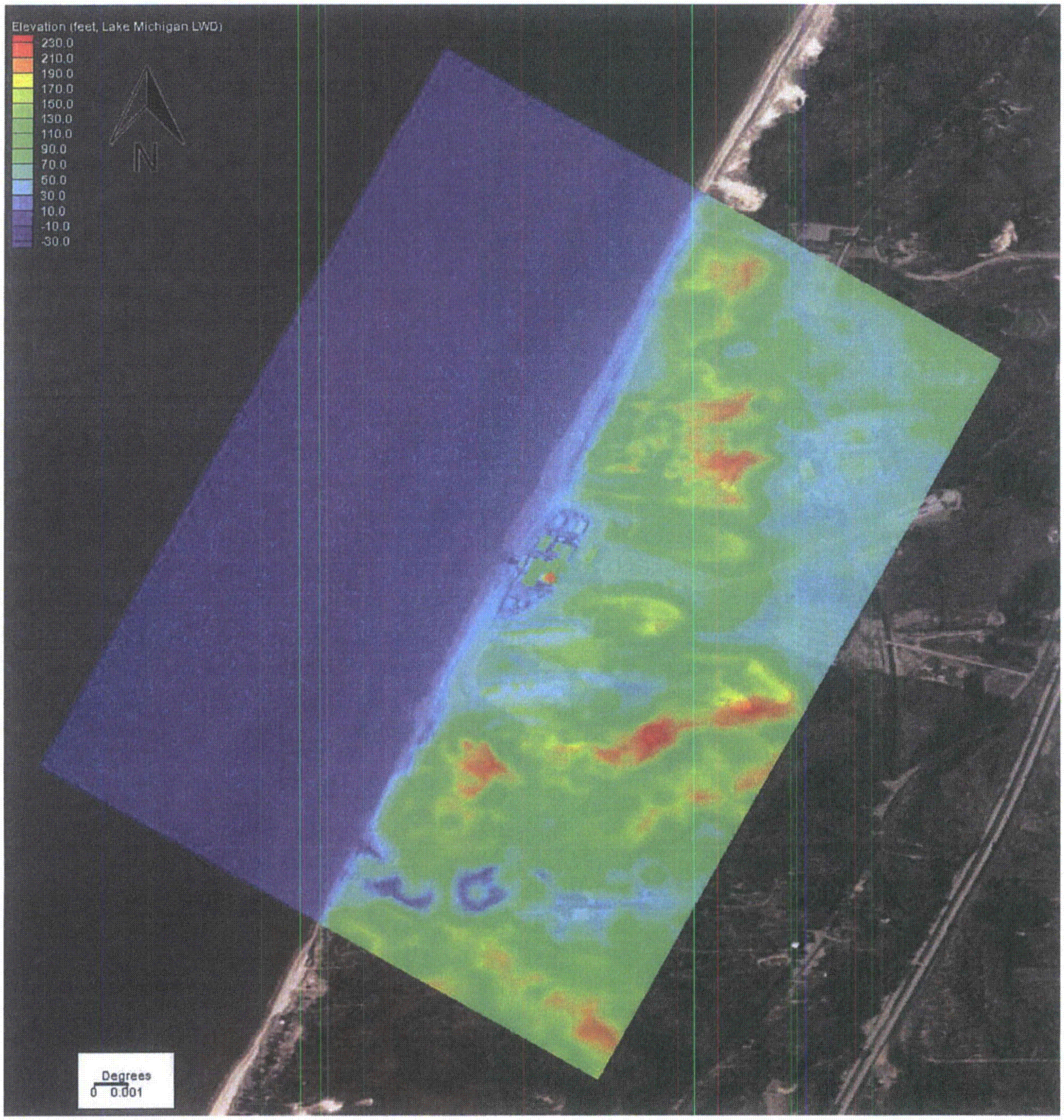
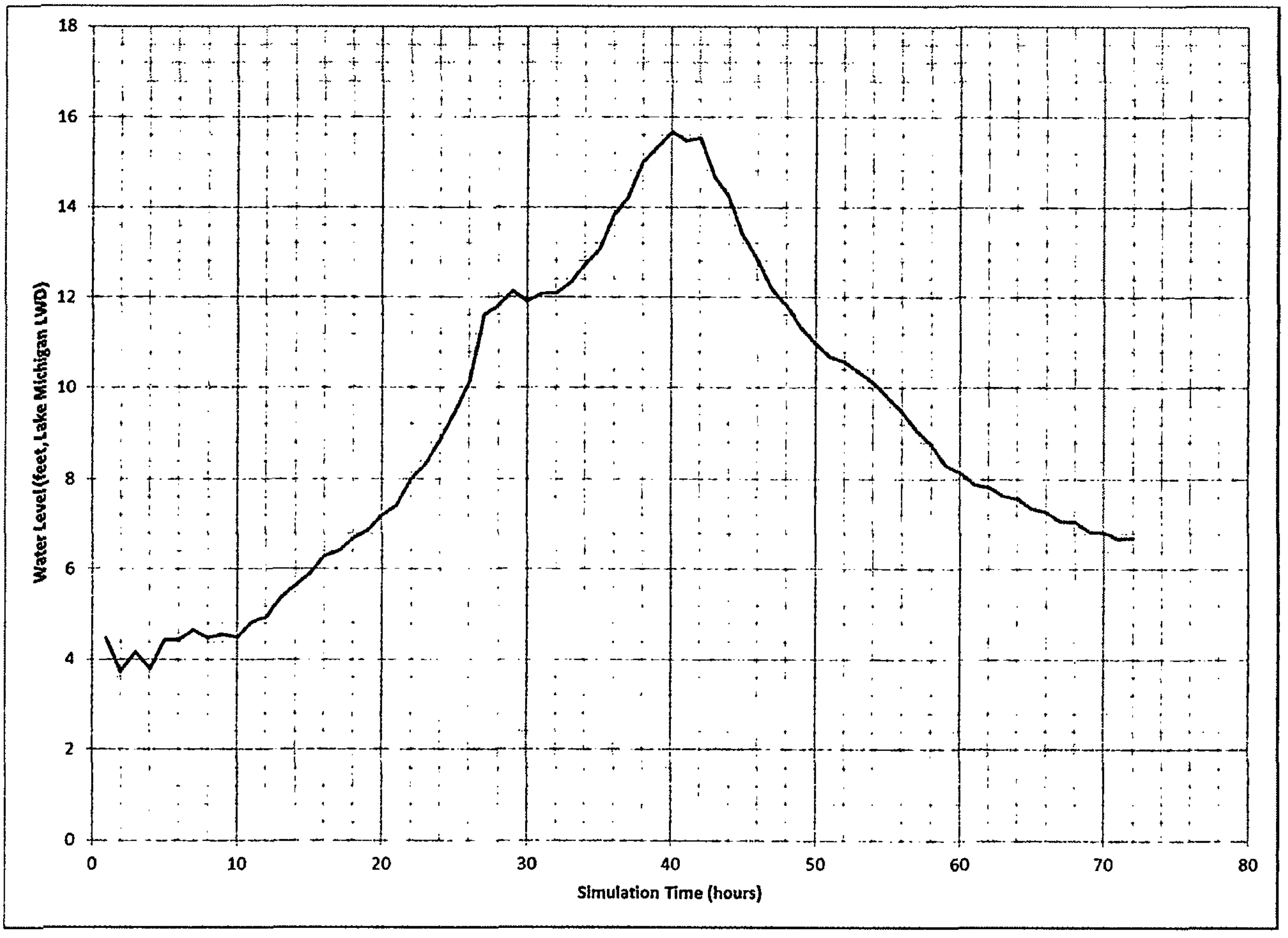
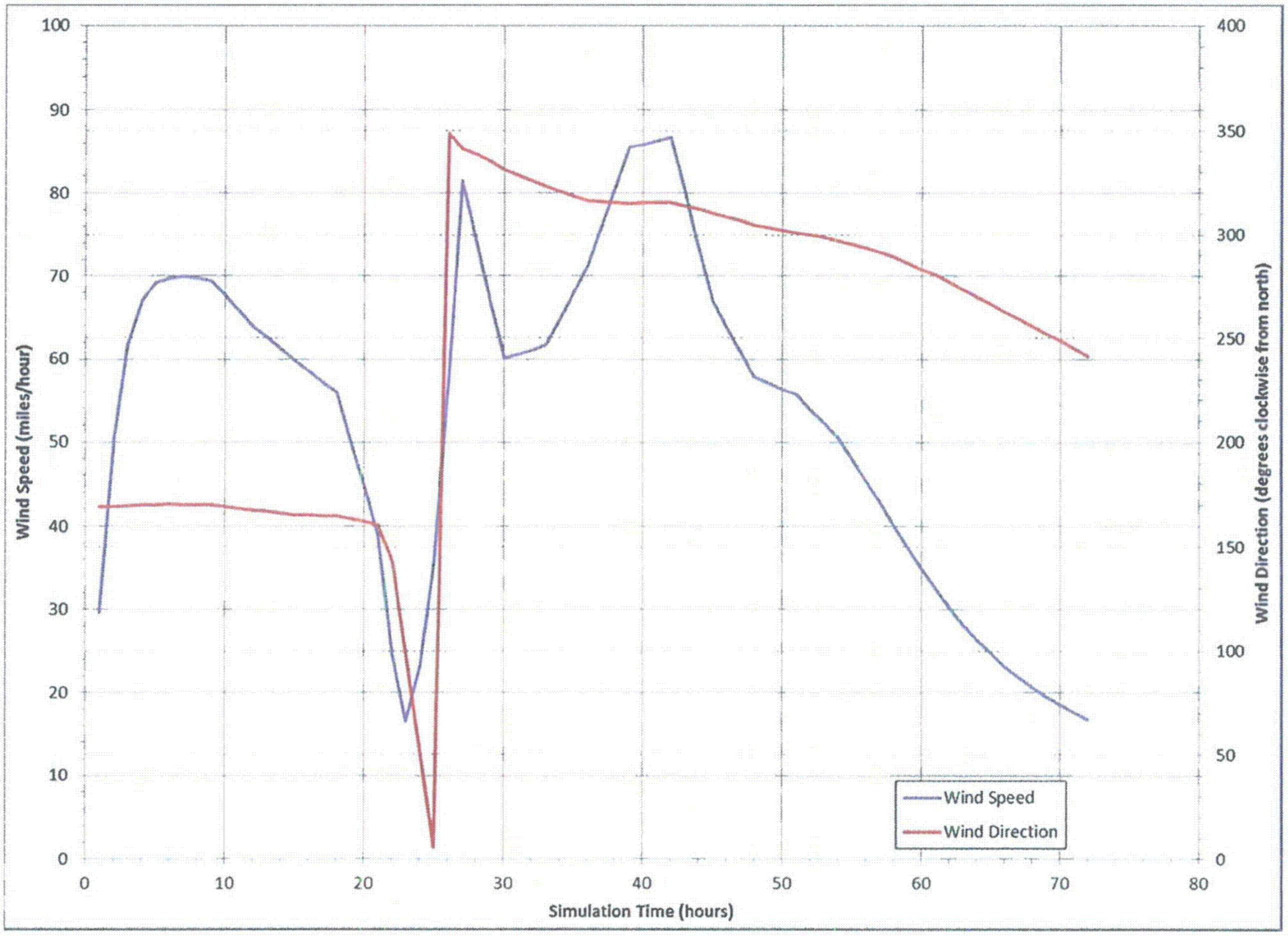


Figure 3-26: Nearshore/Shallow-Water SWAN Simulation Input Water Level



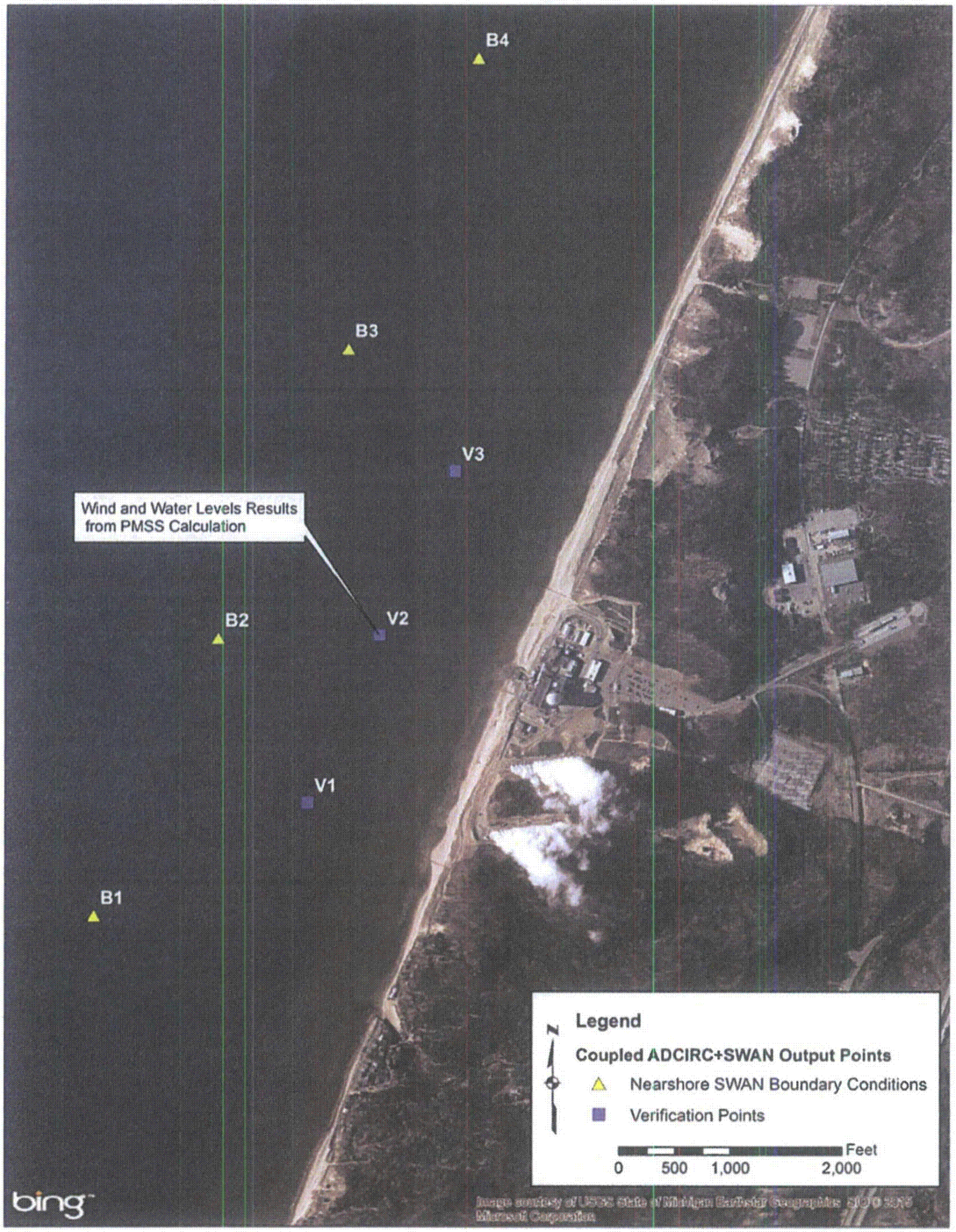
Note: Results from coupled ADCIRC+SWAN simulation at representative output location V2: -86.320234 longitude, 42.324251 latitude; see Figure 3-28 for location.

Figure 3-27: Nearshore/Shallow-Water SWAN Input Wind Speed and Wind Direction



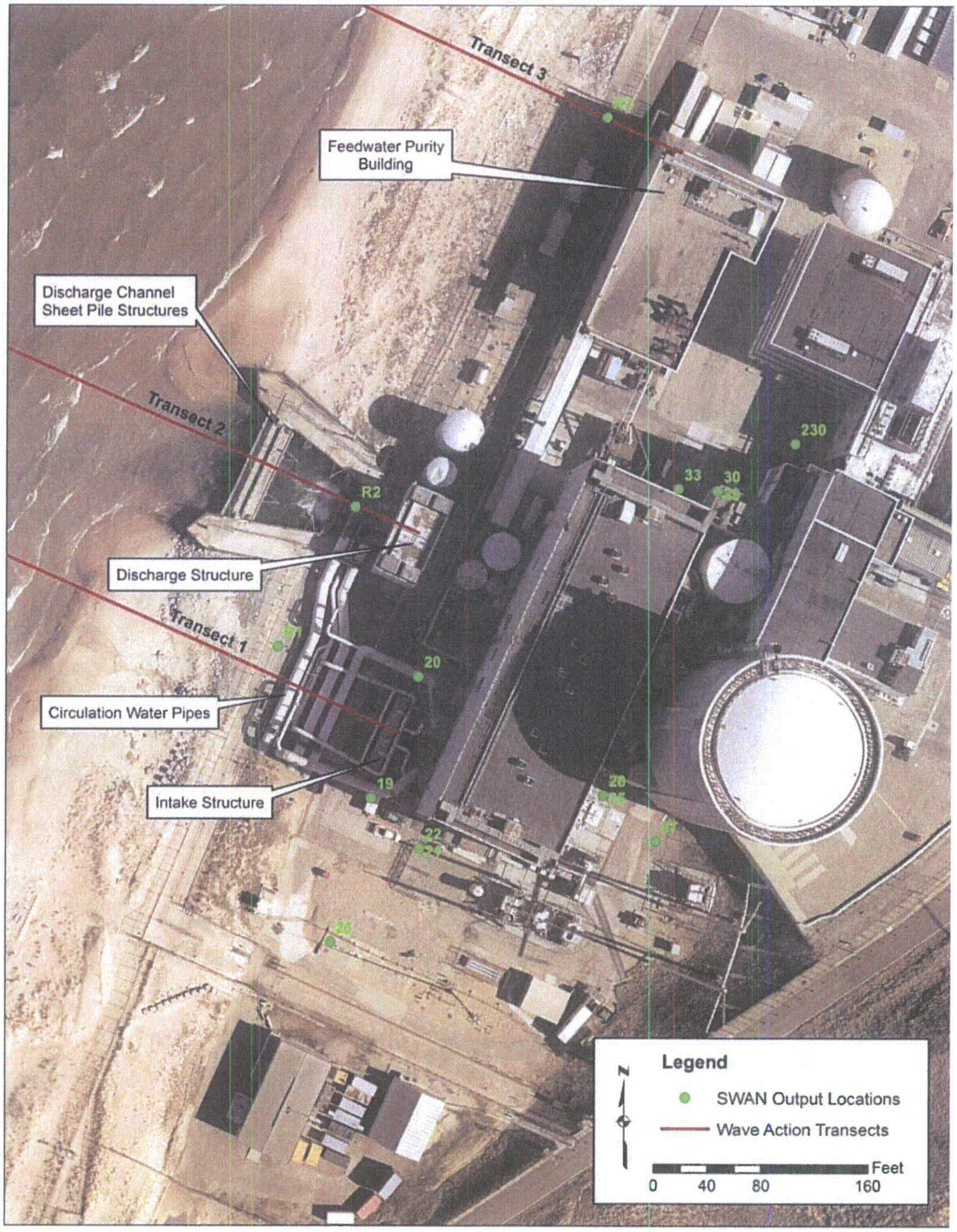
Note: Results from coupled ADCIRC+SWAN simulation at representative output location V2: -86.320234 longitude, 42.324251 latitude; see Figure 3-28 for location.

Figure 3-28: Coupled ADCIRC+SWAN Simulation Output Locations



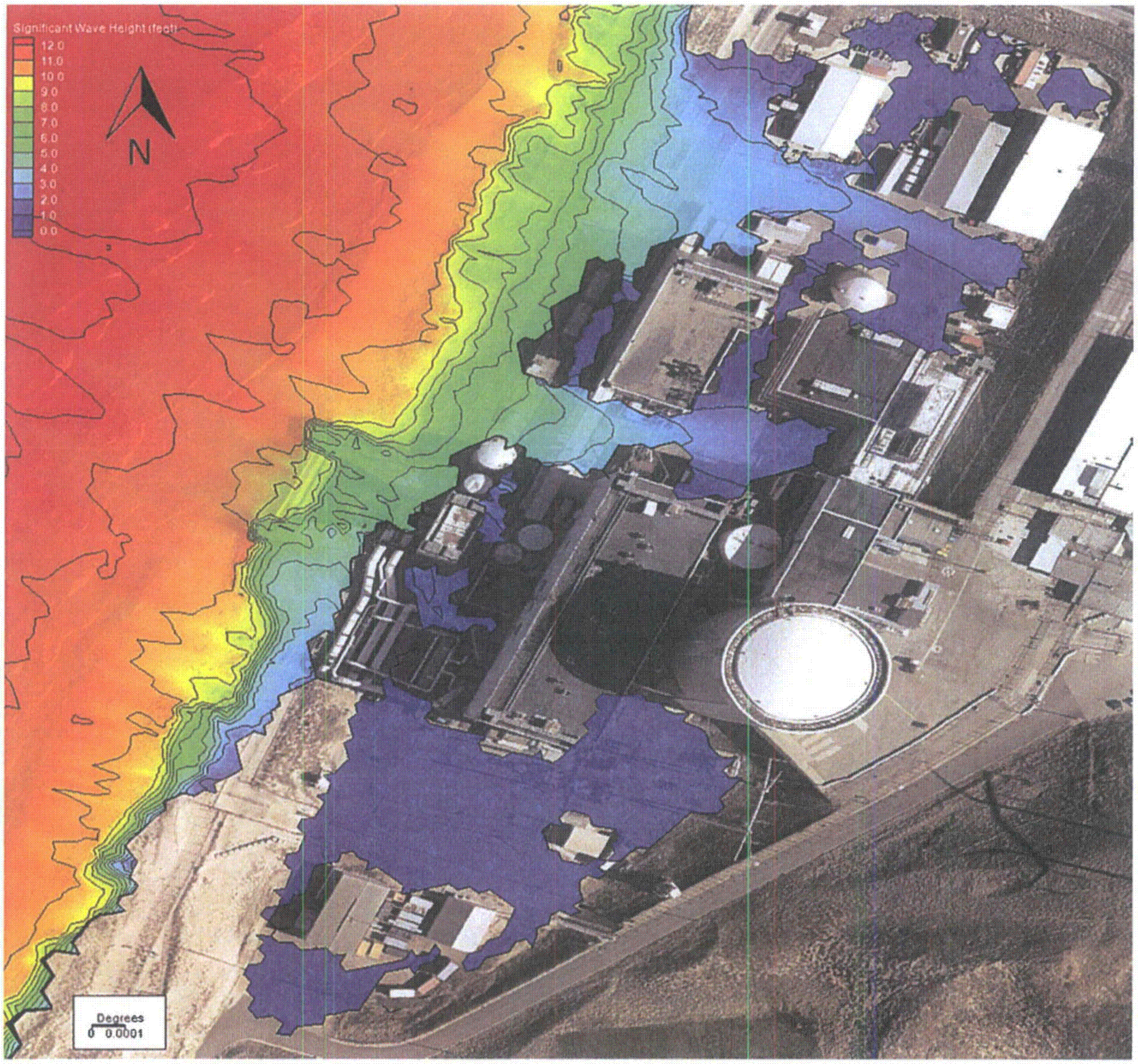
Basemap source: (ESRI 2014)

Figure 3-29: Nearshore/Shallow-Water SWAN Simulation Output Locations



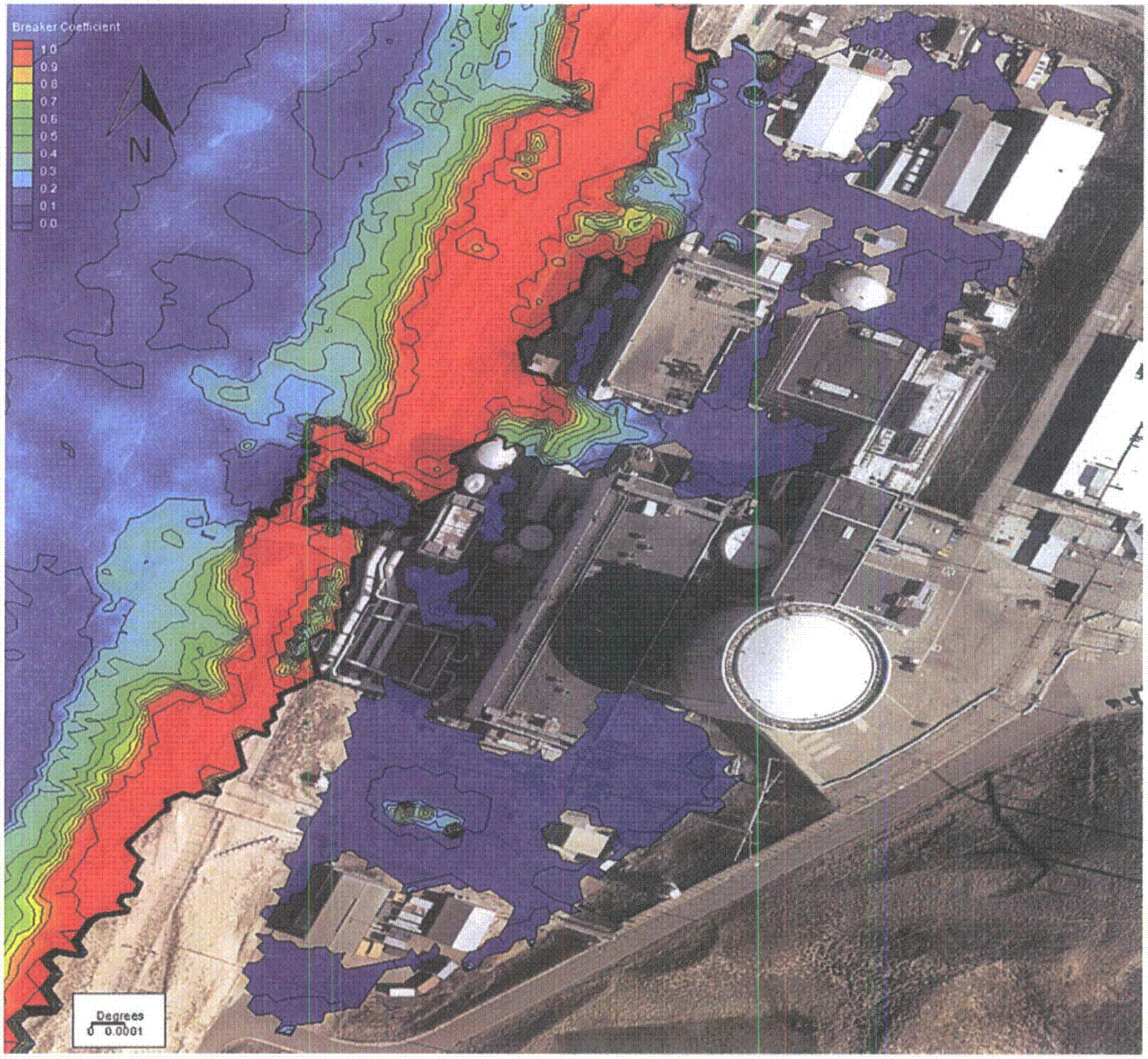
Basemap source: (AREVA 2014b)

Figure 3-30: Coupled ADCIRC+SWAN Simulation Output Locations



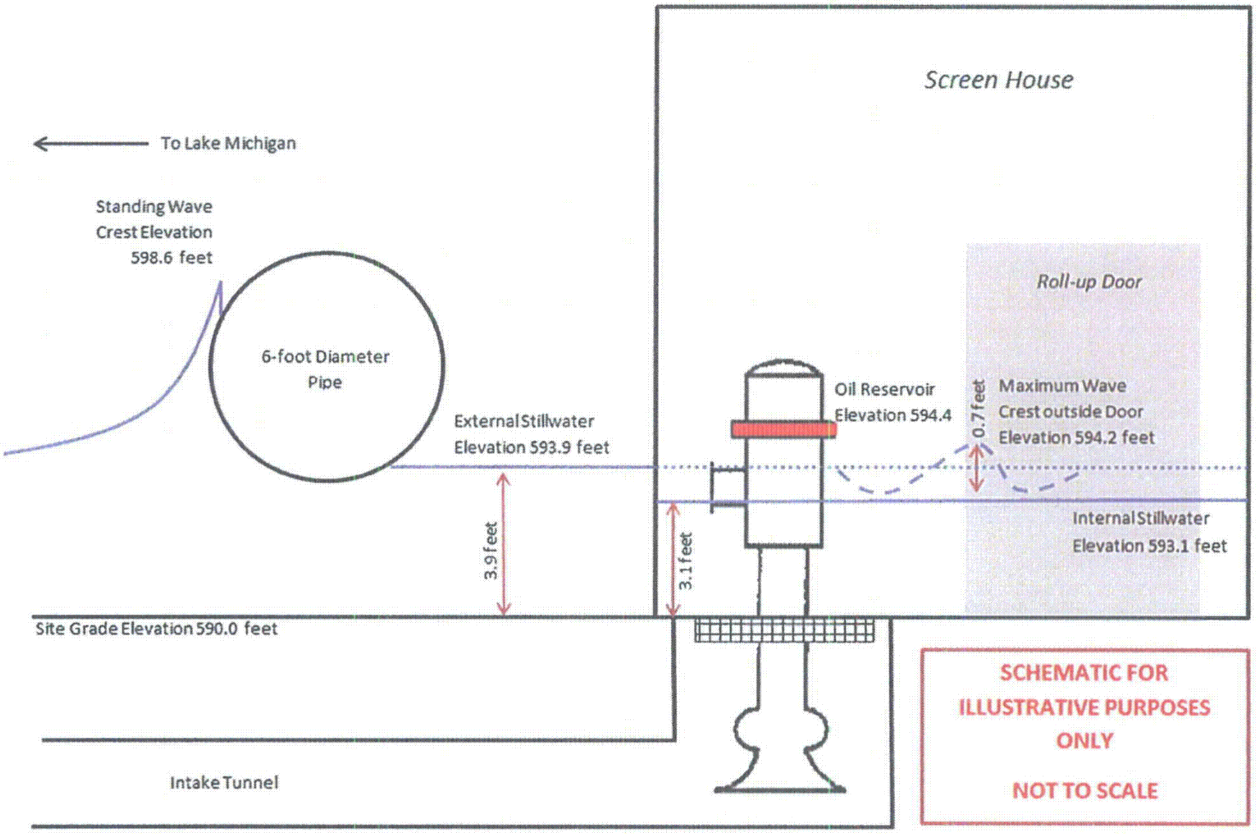
Note: Areas indicated between circulation water pipes, Discharge Structure and Turbine Building are sheltered from wind and external wave activity that cannot be accurately modeled using SWAN. These areas are therefore not subject to the wind-wave activity that is indicated in the figure.

Figure 3-31: Nearshore/Shallow-Water SWAN Simulation Wave Breaking Zone



Note: Areas indicated between circulation water pipes, Discharge Structure and Turbine Building are sheltered from wind and external wave activity that cannot be accurately modeled using SWAN. These areas are therefore not subject to the wind-wave activity that is indicated in the figure.

Figure 3-32: Screen House/Intake Structure Flooding Diagram



4.0 FLOOD PARAMETERS AND COMPARISON WITH CURRENT LICENSING BASIS

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

- Local Intense Precipitation;
- Flooding in Streams and Rivers;
- Dam Breaches and Failures;
- Storm Surge;
- Seiche;
- Tsunami;
- Ice Induced Flooding, and;
- Channel Migration or Diversion.

Some of these individual mechanisms are incorporated into alternative 'Combined Effect Flood' scenarios per Appendix H of NUREG/CR-7046 (NRC 2011).

The March 12, 2012, 10 CFR 50.54(f) letter, Enclosure 2, requests the licensee to perform an integrated assessment of the plant's response to the re-evaluated hazard if the re-evaluated flood hazard is not bounded by the current licensing basis (NRC 2012a). This section provides comparisons with the current licensing basis flood hazard and applicable flood scenario parameters per Section 5.2 of JLD-ISG-2012-05 (NRC 2012b), including:

1. Flood height and associated effects
   a. Stillwater elevation;
   b. Wind waves and run-up effects;
   c. Hydrodynamic loading, including debris;
   d. Effects caused by sediment deposition and erosion (e.g., flow velocities, scour);
   e. Concurrent site conditions, including adverse weather conditions; and,
   f. Groundwater ingress.
2. Flood event duration parameters (per Figure 6 (below) of JLD-ISG-2012-05 (NRC 2012b))
   a. Warning time (may include information from relevant forecasting methods (e.g., products from local, regional or national weather forecasting centers) and ascension time of the flood hydrograph to a point (e.g., intermediate water surface elevations) triggering entry into flood procedures and actions by plant personnel);
   b. Period of site preparation (after entry into flood procedures and before flood waters reach site grade);
   c. Period of inundation, and;
   d. Period of recession (when flood waters completely recede from site and the plant is in a safe and stable state that can be maintained).
3. Plant mode(s) of operation during the flood event duration.
4. Other relevant plant-specific factors (e.g., waterborne projectiles).

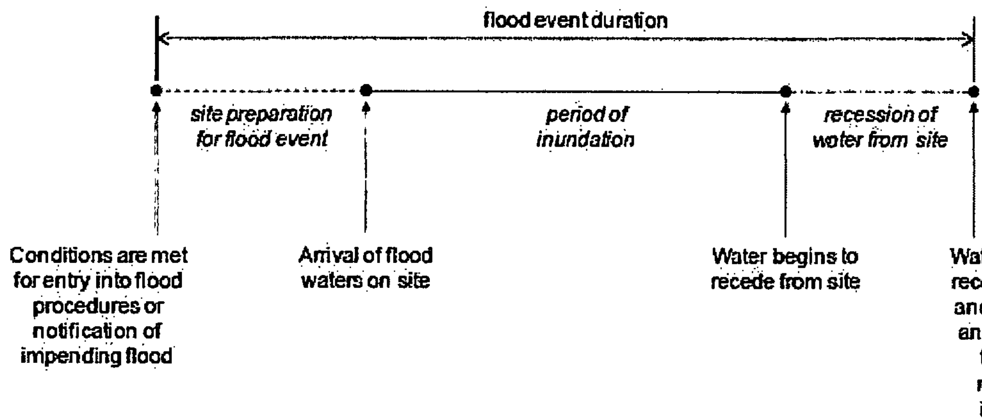


Illustration of Flood Event Duration (Figure 6 of JLD-ISG-2012-05 (NRC 2012b))

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

For PLP, the following flood-causing mechanisms were determined to result in no feasible flood hazard at the site:

- Flooding in Streams and Rivers;
- Dam Breaches and Failures;
- Seiche;
- Tsunami;
- Ice Induced Flooding, and;
- Channel Migration and Diversion

PLP was considered potentially exposed to the flood hazards listed below. In some instances, an individual flood-causing mechanism (e.g., Storm Surge) is addressed in one or more of the combined-effect flood scenarios.

- Local Intense Precipitation;

- Probable Maximum Storm Surge due to a Probable Maximum Wind Storm on Lake Michigan, and;
- Combined Effect Flood Scenarios consisting of the Probable Maximum Storm Surge with coincident wind-generated waves.

Section 4.1 summarizes the re-evaluated flood levels for each flood mechanism and compares the flood elevations to the CLB flood parameters.

4.1 Summary of Current Licensing Basis and Flood Re-Evaluation Results

This section compares the current and re-evaluated flood-causing mechanisms. It provides a comparison of the CLB flood elevation to the re-evaluated flood elevation for each applicable flood-causing mechanism. A comparison of the CLB elevations and the re-evaluated flood elevations is provided in Table 4-1.

Screened mechanisms have been evaluated at a high level and determined to not be applicable to the flooding hazard for PLP.

Flooding due to LIP and the Combined Effect Flood are the only flood mechanisms that could result in inundation in the vicinity of SSCs important to safety above CLB flood heights. Impacts of inundation due to these two flood mechanisms are addressed in Section 5.0.

4.1.1 Local Intense Precipitation

Precipitation induced flooding is currently included in the CLB. The local PMF is based on a PMP of 25.5 inches within six hours. Ponding was assumed to occur on the east side of the Service Building to a depth of five feet. Elsewhere on site, ponding was determined to be less than six inches.

As part of the flood hazard re-evaluation, the maximum water surface elevations due to the LIP flood mechanism result from a PMP depth of 17.3 inches in one hour and 25.5 inches in six hours. Based on the LIP model simulation, the maximum LIP flood elevations near the critical locations range from 592.5 feet (NGVD29) near the north entrance to the Screen House/Intake Structure (Door #33) and three locations in the courtyard, to 594.4 feet (NGVD29) near the Containment Post Tension Tunnel Hatch Door 10A. The maximum LIP flood elevation near the four critical locations on the upper level of the site (approximate site grade of 625 feet NGVD29), was calculated at 626.1 feet NGVD29. Calculated maximum flood depths near the critical locations range from 1.8 feet near the Diesel Generator Fuel Oil Tank T-10A Vent to 5.3 feet near the Containment Post Tension Tunnel Hatch Door 10A. The maximum flood depths near the critical locations in the courtyard generally range from 2.6 feet to 2.8 feet. The maximum flood depth at the east side of the Service Building is approximately 9.8 feet. Maximum flow velocities within the model computational area are up to 12.3 fps on top of the "bridge" west of the Discharge Channel (i.e., mixing basin).

Inundation durations at the critical plant locations are shown in time-series plots in Appendix A.

A comparison of the re-evaluated LIP flood hazard to the CLB is provided in Table 4-2. Flood elevations, depths and durations to maximum flood elevations at critical plant locations are summarized in Table 4-3.

Impacts of LIP flood elevations at critical plant locations are discussed in Section 5.0.

4.1.2 Flooding in Streams and Rivers

Flood hazard due to flooding in streams and rivers was not specifically addressed as part of the CLB and screened out as not impacting the site in the Flood Hazard Re-Evaluation Report for PLP.

4.1.3 Dam Breaches and Failures

Flood hazard due to dam failures was not specifically addressed as part of the CLB and screened out as not impacting the site in the Flood Hazard Re-Evaluation Report for PLP.

4.1.4 Storm Surge

The existing maximum probable storm surge elevation was determined to produce an onshore surge height of 10.9 feet and is the controlling flood event for the PLP CLB, resulting in a design basis flood level of 594.1 feet MSL (PLP 2012, Section 5.4.1.1).

Based on the evaluation summarized in Section 3.4, the predicted PMSS height is 9.7 feet, resulting in a PMSS stillwater elevation of 593.1 feet NGVD29.

4.1.5 Seiche

Flood hazard due to seiche was considered interchangeably with storm surge in the CLB (PLP 2012, Section 2.2.2), and screened out as not impacting the site in the Flood Hazard Re-Evaluation Report for PLP.

4.1.6 Tsunami

Flood hazard due to tsunami was not specifically addressed as part of the CLB and screened out as not impacting the site in the Flood Hazard Re-Evaluation Report for PLP.

4.1.7 Ice Induced Flooding

Flood hazard due to ice induced flooding was not specifically addressed as part of the CLB and screened out as not impacting the site in the Flood Hazard Re-Evaluation Report for PLP.

4.1.8 Channel Migration or Diversion

Flood hazard due to channel migration or diversion was not specifically addressed as part of the CLB and screened out as not impacting the site in the Flood Hazard Re-Evaluation Report for PLP.

4.1.9 Combined Effect

Flood hazard due to combined events was not specifically addressed as part of the CLB.

The primary combined effect flooding mechanism from the flood hazard re-evaluation is the combination of the probable maximum surge and seiche with wind-wave activity and the appropriate antecedent water level (100-year lake water surface elevation). The maximum combined events flood elevation including wave action at the Screen House/Intake Structure is 594.2 feet NGVD29.

A comparison of the re-evaluated combined effect flood hazard to the CLB is provided in Table 4-4. Table 4-5 provides the combined events surface water elevations at important plant locations.

Impacts at important plant locations due to the Combined Effect Flood are discussed in Section 5.0.

Table 4-1: Flood Elevation Comparison

Mechanism	CLB Flood Height	Re-Evaluated Flood Height	Difference
Local Intense Precipitation	Service Bldg: ponding to 5 feet Elsewhere: ponding < 0.5 feet	Service Bldg: 9.8 feet Upper level: up to 2.2 feet Lower level: up to 5.3 feet	+4.8 feet +1.7 feet +4.8 feet
PMF on Rivers and Streams	Not Evaluated	Screened	Not Applicable
Dam Breaches and Failures	Not Evaluated	Screened	Not Applicable
Storm Surge	594.1 feet MSL	593.1 feet NGVD29 [NGVD29 is equivalent to MSL.]	-1.0 foot
Seiche	Evaluated interchangeably with storm surge	Screened	Not Applicable
Tsunami	Not Evaluated	Screened	Not Applicable
Ice Induced Flooding	Not Evaluated	Screened	Not Applicable
Channel Migration or Diversion	Not Evaluated	Screened	Not Applicable
Combined Effect	Not Evaluated	See Table 4-5	A direct comparison is not applicable; however, several re-evaluated flood levels are above the CLB flood protected elevation of 594.4 feet. See Section 5.0.
Note: "Not Evaluated" indicates that this flood mechanism was not defined or addressed in CLB documents. As a result, no comparison can be made to re-evaluated results.			

Table 4-2: Local Intense Precipitation

Flood Scenario Parameter		CLB Flood Hazard	Re-Evaluated Flood Hazard	Bounded (B) or Not Bounded (NB)
Flood Level and Associated Effects	Max Stillwater Elevation	Ponding to a depth of five feet on east side of Service Building. Ponding elsewhere < six inches. (PLP 2012, Section 5.4.1).	9.8 feet on east side of Service Bldg. approx. 1.0 foot to 5.3 feet	NB (See Section 5.0)
	Max Wave Run-up Elevation	Not identified in the CLB pursuant to precipitation induced flooding.	Wind/wave interaction was not evaluated coincident with the LIP event.	B
	Max Hydrodynamic/Debris Loading	Not identified in the CLB.	Hydrodynamic loading was not evaluated. Debris loading was not considered a credible hazard due to limited debris sources within the protected area.	B
	Effects of Sediment Deposition/Erosion	Not identified in the CLB.	Flow velocities were below USACE standards for paved surfaces except for one roadway location; however, significant erosion of the roadway is not anticipated due to the short duration of high flow rates.	NB (See Section 5.0)
	Concurrent Site Conditions	Not identified in the CLB.	No antecedent storm was considered with the LIP event.	B
	Effects on Groundwater	Below grade portions of PLP structures can withstand hydrostatic pressures for a groundwater elevation of 585 feet (PLP 2012, Section 5.9.1.4)	Effect on groundwater was not evaluated.	B

Flood Scenario Parameter		CLB Flood Hazard	Re-Evaluated Flood Hazard	Bounded (B) or Not Bounded (NB)
Flood Event Duration	Warning Time	Not identified in the CLB.	Not evaluated.	B
	Period of Site Preparation	No preparation is indicated in the CLB.	Not evaluated.	B
	Period of Inundation (hours)	Not identified in the CLB.	0.2 to 0.5 hours at critical locations.	NB (See Section 5.0)
	Period of Recession (hours)	Not identified in the CLB.	Typically six hours; however, up to 24 hours at some critical locations. No underground drainage was assumed.	NB (See Section 5.0)
Other	Plant Mode of Operations	Not identified in the CLB.	No operational modes assumed or evaluated.	B
Note: B/NB indicates if the re-evaluation parameters or results are bound/not bound by the CLB evaluation parameters or results.				

Table 4-3: LIP Flood Depths and Durations at Select Locations

ID Number	Location Description	Ground Surface Elevation (feet, NGVD29)	Flood Elevation (feet, NGVD29)	Maximum Flood Depth (feet)	Time to Maximum Flood Elevation (hours)
19	Screen House/Intake Structure Roll-Up (Door #14)	589.9	593.1	3.3	0.5
20	North Entrance to Screen House/Intake Structure (Door #33)	589.7	592.5	2.7	0.5
21	Turbine Building Laydown Area Access (Door #12)	589.7	593.4	3.8	0.5
22	Turbine Building Southwest Roll-Up Door (Door #13)	589.7	593.4	3.8	0.5
23	Diesel Generator Fuel Oil Tank T-10A Vent	591.9	593.6	1.8	0.5
25	Door to Transformer Yard from Feedwater Pumps (Door #11)	589.8	594.2	4.4	0.5
26	Turbine Building Southside Roll-Up Door to Transformer Yard (Door #10)	589.7	594.3	4.6	0.5
27	Containment Post Tension Tunnel Hatch (Door #10A)	589.1	594.4	5.3	0.5
28	Manhole #4 (East of Containment Building)	623.9	626.1	2.2	0.2
29	North Chained Double Door to Diesel Generators (Door #170)	589.7	592.5	2.8	0.5
33	Turbine Building North Entrance Door	589.9	592.5	2.6	0.5
34	Track Alley Rollup Door	624.9	626.0	1.1	0.2
35	Admin Building Hallway East Entrance (Door #28)	624.7	626.1	1.4	0.2
36	North Penetration Room (Door #106)	625.0	626.1	1.0	0.2
230	South Stairwell (Service Bldg Addition) Across from Elevator (Door #123)	589.8	592.5	2.7	0.5

Note: LIP results are rounded to one decimal place. Therefore, the maximum flood depth was approximately calculated by subtracting the ground surface elevation from the flood elevation.

Table 4-4: Combined Effect Flood

Flood Scenario Parameter		CLB Flood Hazard	Re-Evaluated Flood Hazard	Bounded (B) or Not Bounded (NB)
Flood Level and Associated Effects	Max Stillwater Elevation	Not identified in the CLB.	See Table 4-5	NB (See Section 5.0)
	Max Wave Run-up Elevation	The Intake Structure can withstand the dynamic effect of wave run-up of approximately eight feet (PLP 2012, Section 5.4.1.2)	Wave effects associated with the PMWS, which includes wave setup and wave runup, were evaluated.	B
	Max Hydrodynamic/Debris Loading	Not identified in the CLB.	Hydrodynamic, hydrostatic and wave loads impacting the circulation water pipes were evaluated and the pipes were determined to be structurally adequate for the applied loads (AREVA 2015). Although water flows beneath the circulation water pipes, nearshore waves that propagate from offshore break on the circulation water pipes and do not propagate further to the Screen House/Intake Structure.	NB (See Section 5.0)
	Effects of Sediment Deposition/Erosion	Not identified in the CLB.	Not evaluated.	B
	Concurrent Site Conditions	Not identified in the CLB.	An antecedent water level of 583.4 feet NGVD29 was considered.	NB (See Section 5.0)
	Effects on Groundwater	Below grade portions of PLP structures can withstand hydrostatic pressures for a groundwater elevation of 585 feet (PLP 2012, Section 5.9.1.4)	Due to short duration of flooding and slow percolation rate of underlying soil, short term water level changes are unlikely to affect groundwater levels.	B
Flood Event Duration	Warning Time	Not identified in the CLB.	Not evaluated.	B
	Period of Site Preparation	No preparation is indicated in the CLB.	Not evaluated.	B
	Period of Inundation	Not identified in the CLB.	Not evaluated.	B
	Period of Recession	Not identified in the CLB.	Not evaluated.	B
Other	Plant Mode of Operations	Not identified in the CLB.	No operational modes assumed or evaluated.	B

Note: B/NB indicates if the re-evaluation parameters or results are bound/not bound by the CLB evaluation parameters or results.

Table 4-5: Combined Effect Wave Elevations

ID Number	Location	Combined Event Water Elevation (feet NGVD29)
19	Screen House/Intake Structure Roll-Up (Door #14)	594.2
20	North Entrance to Screen House/Intake Structure (Door #33)	593.9
21	Turbine Building Laydown Area Access (Door #12)	594.3
22	Turbine Building Southwest Roll-Up Door (Door #13)	594.3
23	Diesel Generator Fuel Oil Tank T-10A Vent	594.3
25	Door to Transformer Yard from Feedwater Pumps (Door #11)	594.2
26	Turbine Building Southside Roll-Up Door to Transformer Yard (Door #10)	594.2
27	Post Tension Tunnel Hatch Door 10A	594.3
29	North Chained Double Door to Diesel Generators (Door #170)	594.6
30	Auxiliary Building Addition Entrance Door [Note: Same location as ID Number 29 (i.e., Door #170)]	594.6
33	Turbine Building North Entrance Door	594.2
230	South Stairwell (Service Bldg Addition) Across from Elevator (Door #123)	595.0
R1	West (Shoreward) Circulation Water Pipe	598.6
R1	West (Shoreward) Wall of Screen House/Intake Structure	593.9
R2	West (Shoreward) Wall of Discharge Structure	602.2
R3	West (Shoreward) Wall of Feedwater Purity Building	598.7

Note: Combined Event Water Elevation is the controlling (lesser) of the breaking wave crest, maximum wave crest within the PLP site area, or the standing wave crest elevation on lake-facing structures.

4.2 References

AREVA 2015. Palisades Nuclear Plant Flooding Re-Evaluation – Wave Loadings on Cooling Tower Piping, AREVA Document No. 32-9234660-000, 2015.

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

NRC 2012a. 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, U.S. Nuclear Regulatory Commission, March 2012.

NRC 2012b. JLD-ISG-2012-05, Guidance for Performing the Integrated Assessment for External Flooding, Interim Staff Guidance, Revision 0, 2012.

PLP 2012. Palisades Nuclear Plant Final Safety Analysis Report (FSAR), Revision 30, September 4, 2012. (AREVA Document No. 38-9223712-000)

5.0 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED

Flooding due to LIP and the Combined Event Flood are the only flood mechanisms which could cause inundation of the PLP site in the vicinity of SSCs important to safety above CLB flood heights.

5.1 Impacts of Re-evaluated Flood Elevations

In response to the flood elevations at the site resulting from the LIP flood event and the Combined Effect Flood, an assessment was performed to determine the impact of inundation at the affected locations identified in Section 3.1 due to the LIP and in Section 3.9 due to the Combined Effect Flood. The results of the assessment indicate that the re-evaluated flood elevations would not inundate plant areas in the vicinity of equipment important to safety, as discussed further below.

5.1.1 LIP Affected Locations

Table 5-1 lists the three affected locations on the lower level of the PLP site where LIP elevations are close to or at the minimum flood protection elevation of 594.4 feet MSL (i.e., equivalent to 594.4 feet NGVD29); none of the locations exceed the minimum flood protection elevation of 594.4 feet MSL which is based on a component that is 4.4 feet above the 590 foot elevation of the plant around the Auxiliary and Turbine Buildings (see Section 2.1.2). Additionally, none of the three locations pose a threat to SSCs important to safety since there are either no SSCs important to safety located in the vicinity of the inundated areas or there are other internal barriers (i.e., flood doors) that preclude impact to SSCs important to safety from flood waters (PLP 2015).

Table 5-2 lists the four affected locations on the upper level of the PLP site which would allow ingress of water due to the LIP; however, none of the four locations pose a threat to SSCs important to safety since there are no SSCs important to safety located in the vicinity of the inundated areas (PLP 2015). Although the conduits in Manhole #4 need to be sealed, an evaluation (i.e., EA-EC55593-01) has been performed and has demonstrated that the LIP flood elevation at Manhole #4 is not a concern.

5.1.2 Combined Effect Flood Affected Locations

Table 5-3 lists the affected locations on the PLP site where the combined effect flood elevations are close to or above the minimum flood protection elevation of 594.4 feet MSL (i.e., equivalent to 594.4 feet NGVD29). As noted in Table 5-3, a structural evaluation was performed and determined that the circulation water pipes would break the wind-driven wave action associated with the Combined Effect Flood and thus, protect the Screen House/Intake Structure and the service water pumps within the Screen House/Intake Structure (AREVA 2015). Since SSCs important to safety would not be adversely impacted by the Combined Effect Flood, no other actions have been taken or are planned (PLP 2015).

5.2 Conclusions

Plant walkdowns have validated that the areas in the immediate vicinity of SSCs important to safety are not impacted by the LIP flood event or the Combined Effect Flood (PLP 2015). Therefore, except for sealing the conduits within Manhole #4, no additional actions are planned (PLP 2015).

5.3 References

AREVA 2015. Palisades Nuclear Plant Flooding Re-Evaluation – Wave Loadings on Cooling Tower Piping, AREVA Document No. 32-9234660-000, 2015.

PLP 2015. PLP Design Input Record, Document No. PLP RFI #2015-002, Dated February, 2015. (AREVA Document No. 38-9234957-000)

Table 5-1: Lower Level LIP Affected Locations

[Source: PLP 2015]

Location	Flood Elevation (feet MSL)	Disposition
Door to Transformer Yard from Feedwater Pumps (Door #11)	594.2	Below flood protected elevation of 594.4 feet MSL.
Turbine Building Southside Roll-Up Door to Transformer Yard (Door #10)	594.3	Below flood protected elevation of 594.4 feet MSL.
Containment Post-Tension Tunnel Hatch (Door #10A)	594.4	At flood protected elevation of 594.4 feet MSL. The hatch is maintained as a safety-related flood door with no leakage noted. Additionally, there is no safety related equipment under this flood hatch. The critical elevation of 594.4 feet MSL is in a separate building, several hundreds of feet away.
Note: Elevations in MSL are equivalent to elevations in NGVD29.		

Table 5-2: Upper Level LIP Affected Locations

[Source: PLP 2015]

Location	Flood Elevation (feet MSL)	Disposition
Manhole #4 (East of Containment)	626.1	Manhole #4 (MH-4) contains unsealed conduits which eventually track to the safety related 1C Switchgear room sump pit (manholes MH-1, -2 and -3). The 2.2 foot water level does not reach the top of the sump; there is one opening in the MH-4 lid accounting for a very low flow into the manholes. Additionally, the conduit runs are filled with cables and span a distance of several hundred feet. Evaluation EA-EC55593-01 has been issued to evaluate water leakage. There is no concern of operability for the 1C Switchgear. Currently, a Preventive Maintenance (PM) Model Work Order is in place (MH-4 Model Work Order 242311) to inspect and pump out the manhole, if necessary. Condition Report CR-PLP-2015-00784 has been initiated to document this condition and to track sealing of the conduits. Additionally, the bottom of the manhole may be removed to allow any rainwater entering the manhole to dissipate through the soil.
Track Alley Roll-Up Door	626.0	No safety related equipment exists in the vicinity of the track alley roll-up door. With the size of the roll-up door, there is adequate structural capability that approximately two feet of water pressure will not cause concerns.
Admin Building East Door (Door #28)	626.1	No safety related equipment exists in the vicinity of the Admin Building East Door, although there is a very indirect path into the Auxiliary Building if the water travels approximately 100 feet down a hallway, down several flights of stairs and through more offices. This amount of distance, combined with Palisades' heavy rainfall history never going through the double entry doors, alleviates concerns.
North Penetration Room (Door #106)	626.1	This exterior door leading into the north penetration room would allow for water ingress. From the north penetration room, there is an open (unsealed) pipe way that eventually leads into the safety related 1D Switchgear Room. Flooding during rainfall events has never been recorded in this area. Additionally, the pipe way is elevated and the entire room would have to flood before reaching the pipe way. By engineering judgment, this is not considered a concern.
Note: Elevations in MSL are equivalent to elevations in NGVD29.		

Table 5-3: Combined Effect Flood Affected Locations

[Source: PLP 2015]

Location	Flood Elevation (feet MSL)	Disposition
Screen House/Intake Structure Roll-Up (Door #14)	594.2	Below safety-related elevation of 594.4 feet MSL. No actions taken.
North Entrance to Screen House/Intake Structure (Door #33)	593.9	Below safety-related elevation of 594.4 feet MSL. No actions taken.
Turbine Building Laydown Area Access (Door #12)	594.3	Below safety-related elevation of 594.4 feet MSL. No actions taken.
Turbine Building Southwest Roll-Up Door (Door #13)	594.3	Below safety-related elevation of 594.4 feet MSL. No actions taken.
Diesel Generator Fuel Oil Tank T-10A Vent	594.3	The T-10A vent is open around elevation 597 feet MSL. No actions taken.
Door to Transformer Yard from Feedwater Pumps (Door #11)	594.2	Below safety-related elevation of 594.4 feet MSL. No actions taken.
Turbine Building Southside Roll-Up Door to Transformer Yard (Door #10)	594.2	Below safety-related elevation of 594.4 feet MSL. No actions taken.
Post Tension Tunnel Hatch Door 10A	594.3	Below safety-related elevation of 594.4 feet MSL. No actions taken.
North Chained Double Door to Diesel Generators (Door #170)	594.6	This flood elevation is above the lowest safety-related elevation. However, Door #170 is situated a substantial distance from the Screen House/Intake Structure. As such, the service water pump lower motor bearing oil reservoir is not a concern. For the diesel generators, Door #170 is a safety-related flood door maintained by in-place PMs. No actions taken.
Auxiliary Building Addition Entrance Door [Note: Same location as Door #170.]	594.6	This flood elevation is above the lowest safety-related elevation. However, Door #170 is situated a substantial distance from the Screen House/Intake Structure. As such, the service water pump lower motor bearing oil reservoir is not a concern. The Auxiliary Building is protected through various flood doors. No actions taken.
Turbine Building North Entrance Door	594.2	Below safety-related elevation of 594.4 feet MSL. No actions taken.

Location	Flood Elevation (feet MSL)	Disposition
South Stairwell (Service Bldg Addition) Across from Elevator (Door #123)	595.0	This flood elevation is above the lowest safety-related elevation. However, Door #123 is situated a substantial distance from the Screen House/Intake Structure. As such, the service water pump lower motor bearing oil reservoir is not a concern. There is no safety-related equipment within the stairwell/hallway, and there is a flood door that separates this area from the Auxiliary Building. No actions taken.
West (Shoreward) Circulation Water Pipe	598.6	This flood elevation combined with the west wall of the Screen House/Intake Structure would have been an issue since the water level at the Screen House/Intake Structure was originally at 598.6 feet MSL. However, a structural evaluation of the circulation water pipes was performed and determined that the circulation water pipes would break the wind-driven wave action (AREVA 2015). This protects the Screen House/Intake Structure and the service water pumps within the Screen House/Intake Structure. No further action is required.
West (Shoreward) Wall of Screen House/Intake Structure	593.9	This flood elevation is below the safety-related elevation of 594.4 feet MSL. As discussed above, a structural evaluation has determined that the circulation water pipes would break the wind-driven wave action (AREVA 2015). No actions taken.
West (Shoreward) Wall of Discharge Structure	602.2	No safety-related equipment is in the vicinity of the Discharge Structure. No actions taken.
West (Shoreward) Wall of Feedwater Purity Building	598.7	No safety-related equipment is in the vicinity of the Feedwater Purity Building. No actions taken.
Note: Elevations in MSL are equivalent to elevations in NGVD29.		

6.0 ADDITIONAL ACTIONS

Referring to Section 5.0, except for sealing the conduits within Manhole #4, no additional actions are necessary.

APPENDIX A: LOCAL INTENSE PRECIPITATION

A.1 LIP Time Series Hydrographs at Reporting Locations

See the following Appendix A pages.

A.2 LIP FLO-2D Input/Output Files

Due to the large size and formatting of the FLO-2D input/output files, this data is provided as an electronic attachment. The information has been archived in the AREVA file management system, ColdStor. The path to the file is: \cold\General-Access\51\51-9226987-000\official.

This information is also provided electronically with this report as 51-9226987-000_Appendix_A2.zip.

A.3 LIP Results – Large Format Figures

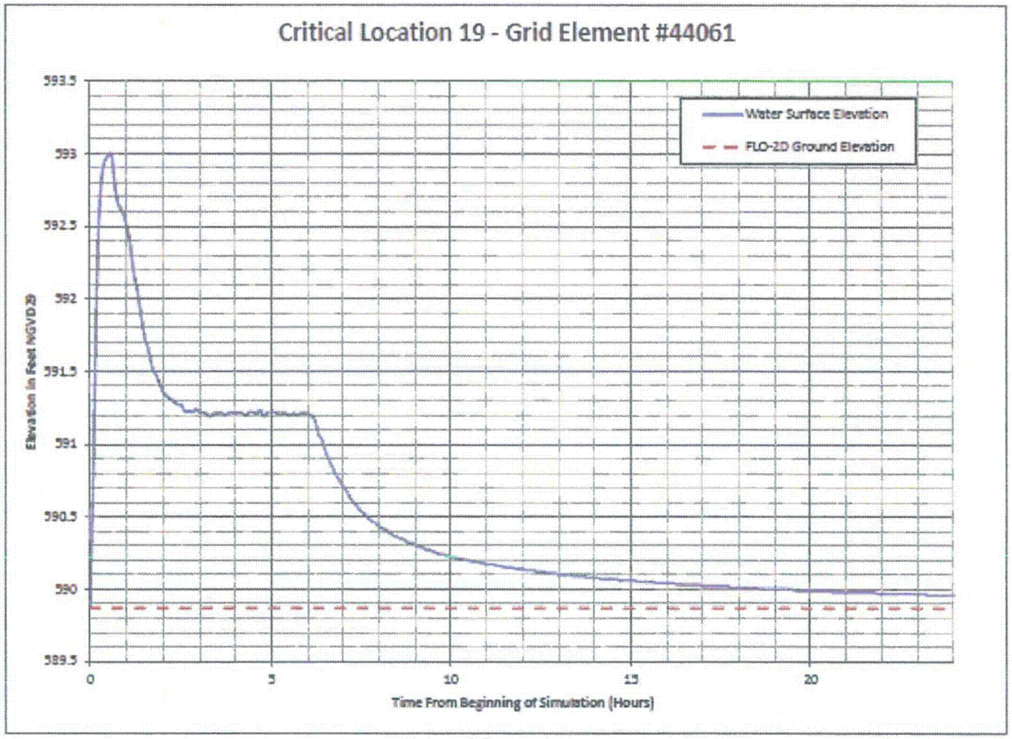
Due to the large file size of the large format figures, they are provided as an electronic attachment. The information has been archived in the AREVA file management system, ColdStor. The path to the file is: \cold\General-Access\51\51-9226987-000\official.

This information is also provided electronically with this report as 51-9226987-000_Appendix_A3.zip.

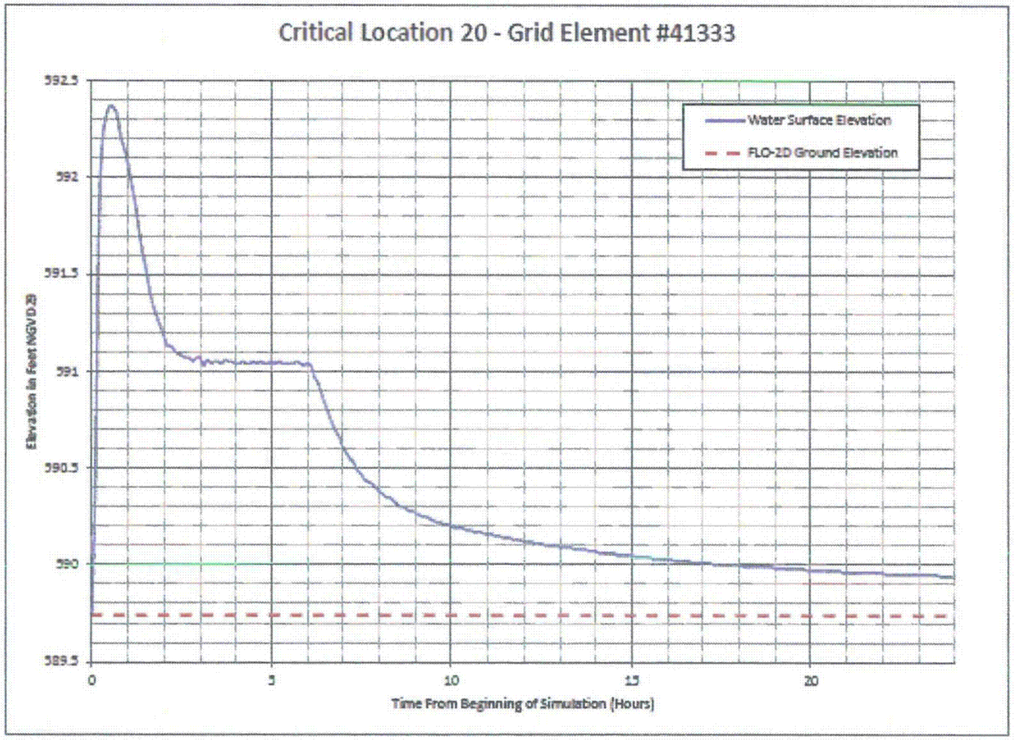
LIP FLO-2D CRITICAL GRID ELEMENT TIME SERIES HYDROGRAPHS

[Source: Appendix J of AREVA Document No. 32-9226944-002, Palisades Nuclear Plant Flooding Hazard Re-Evaluation – Local Intense Precipitation, 2015.]

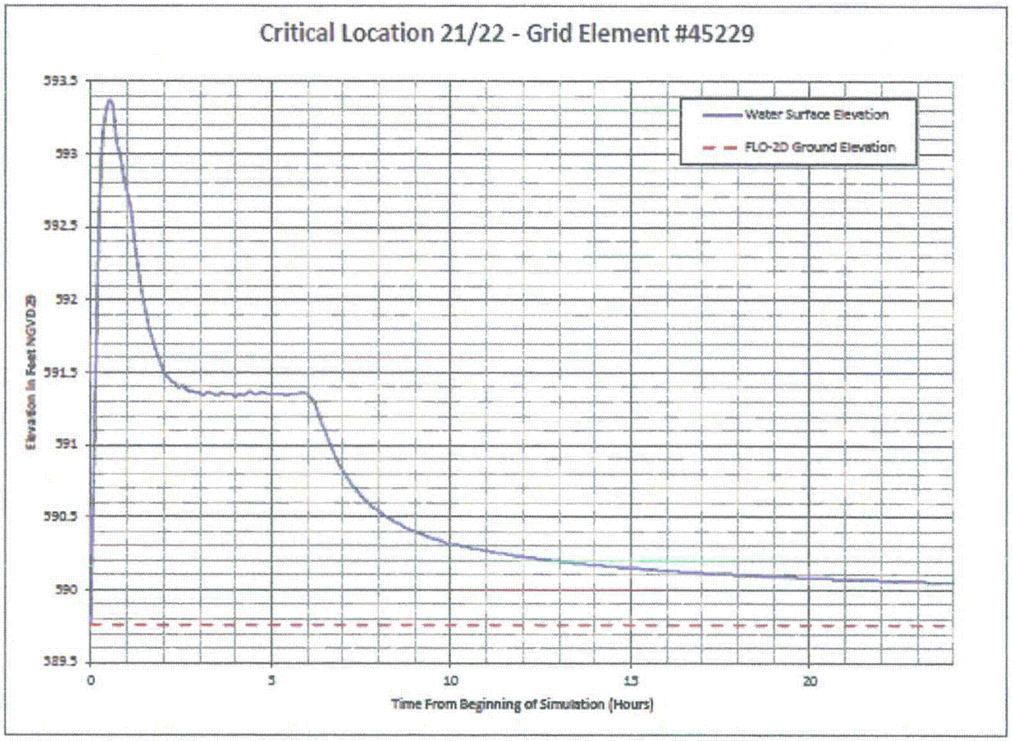
Screen House/Intake Structure Roll-Up (Door #14)



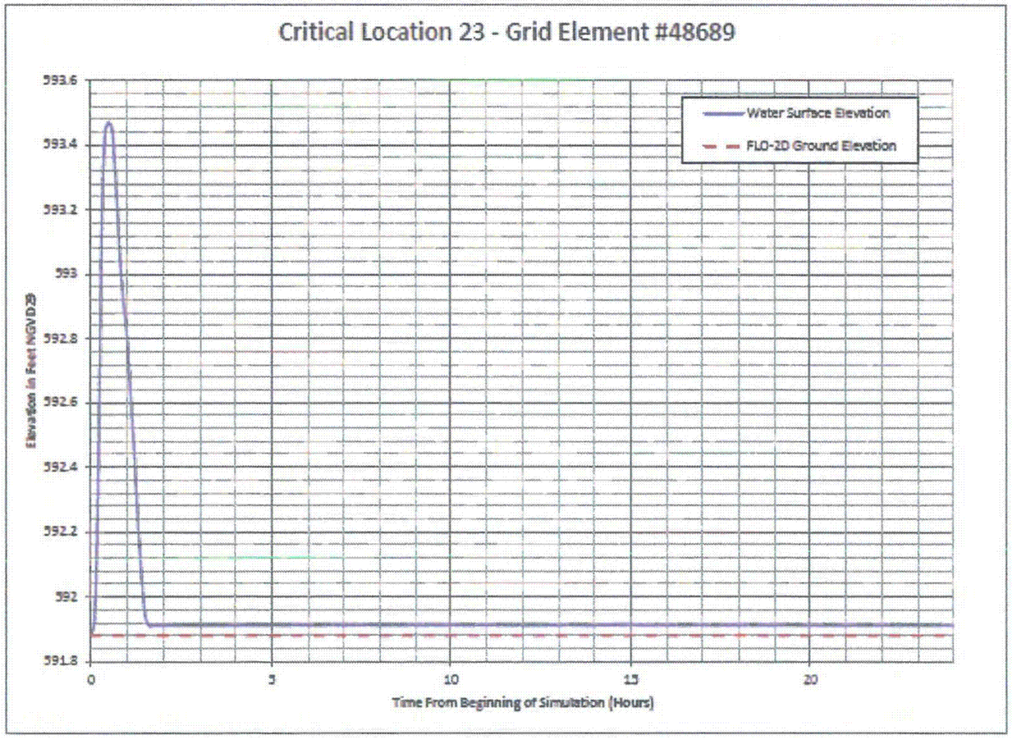
North Entrance to Screen House/Intake Structure (Door #33)



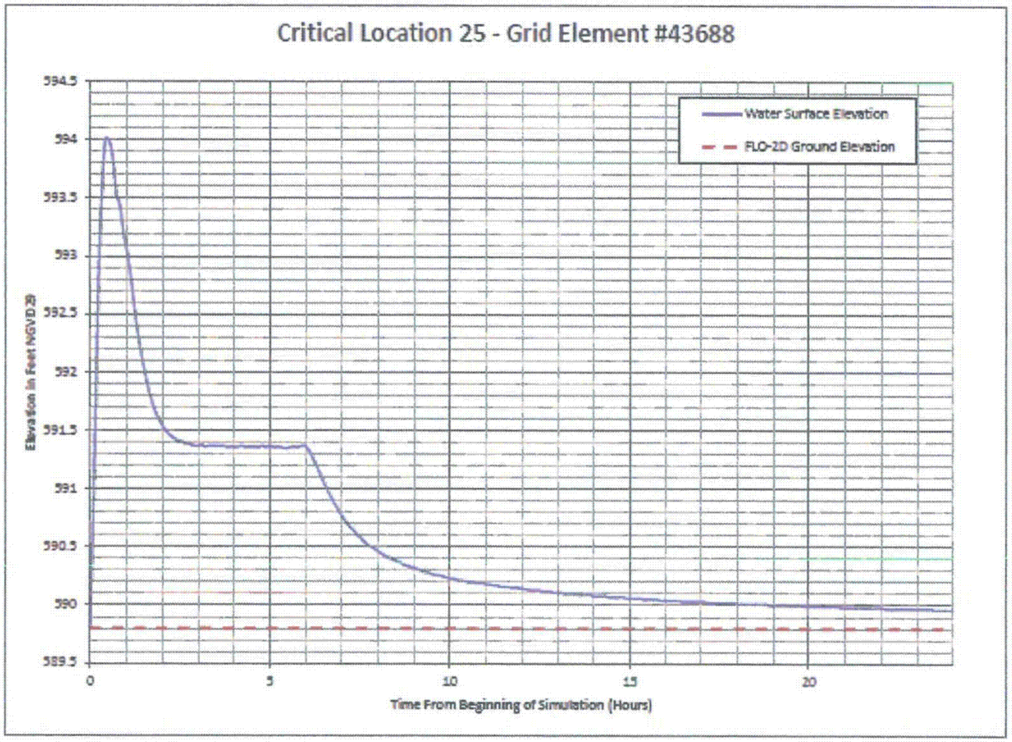
Turbine Building Laydown Area Access (Door #12) and Turbine Building Southwest Roll-Up Door (Door #13)



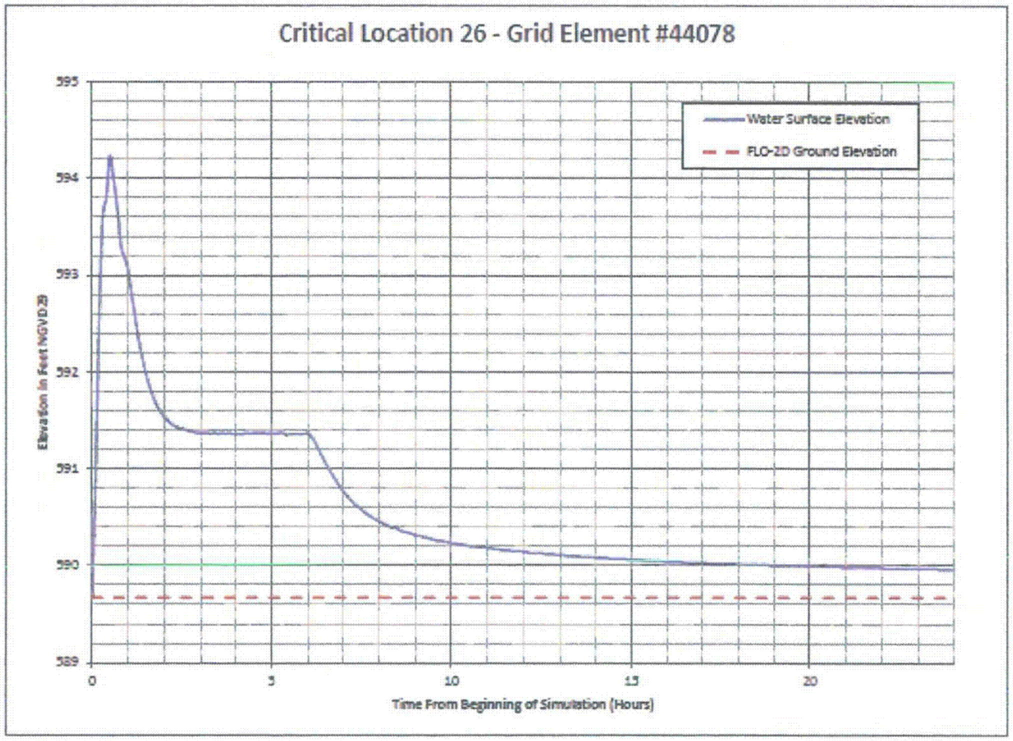
Diesel Generator Fuel Oil Tank T-10A Vent



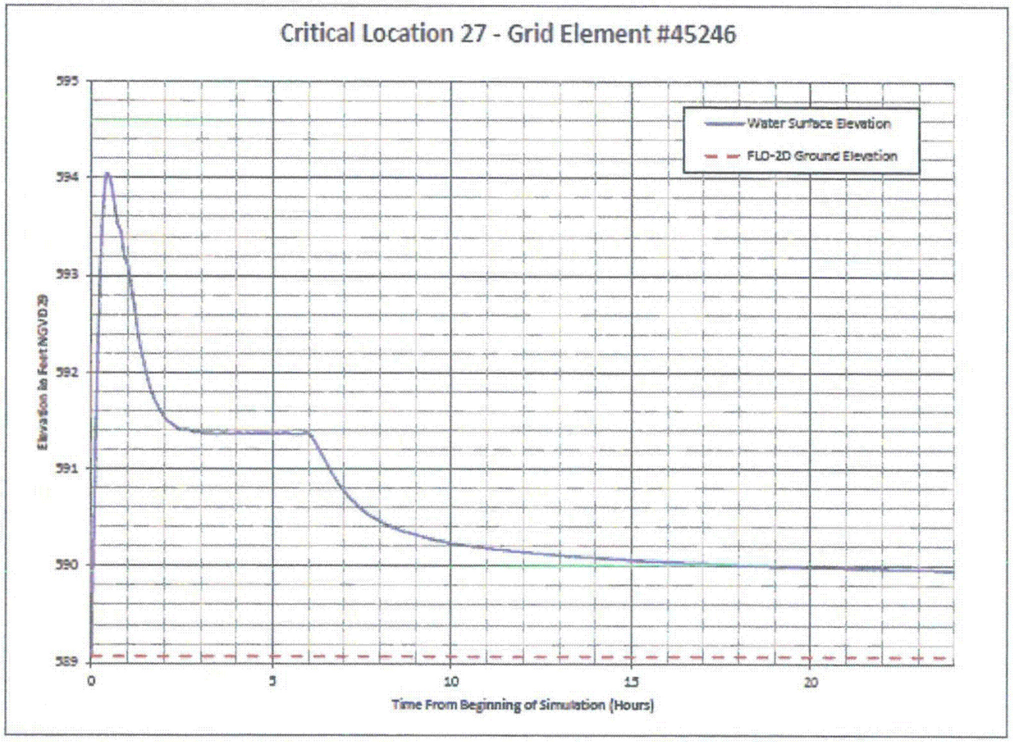
Door to Transformer Yard from Feedwater Pumps (Door #11)



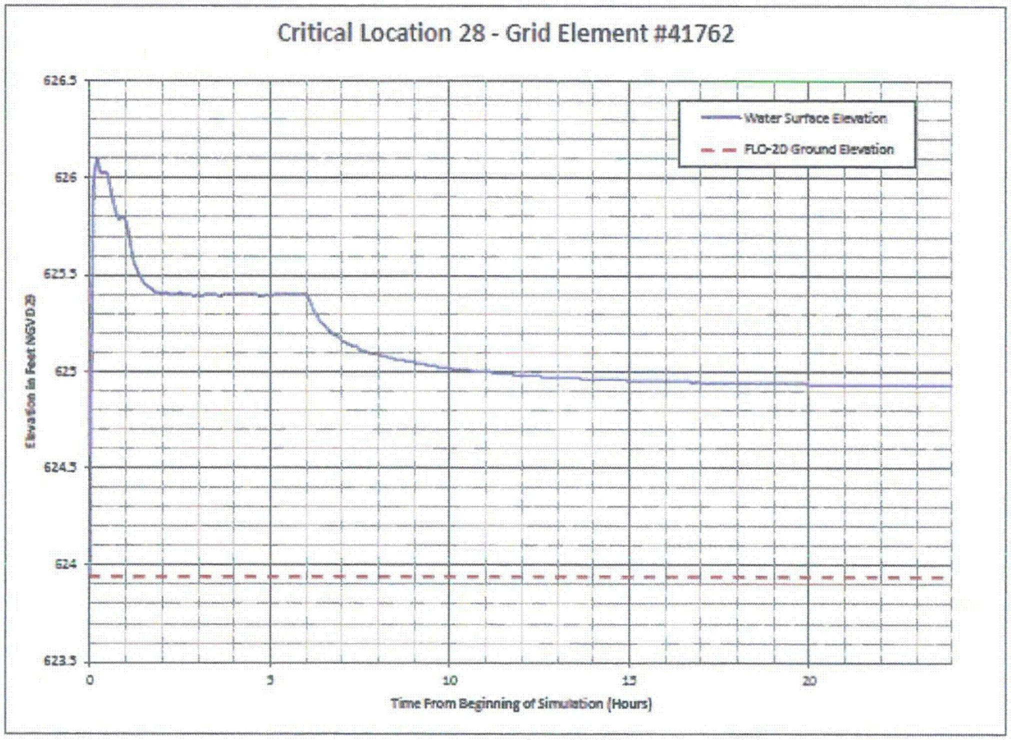
Turbine Building Southside Roll-Up Door to Transformer Yard (Door #10)



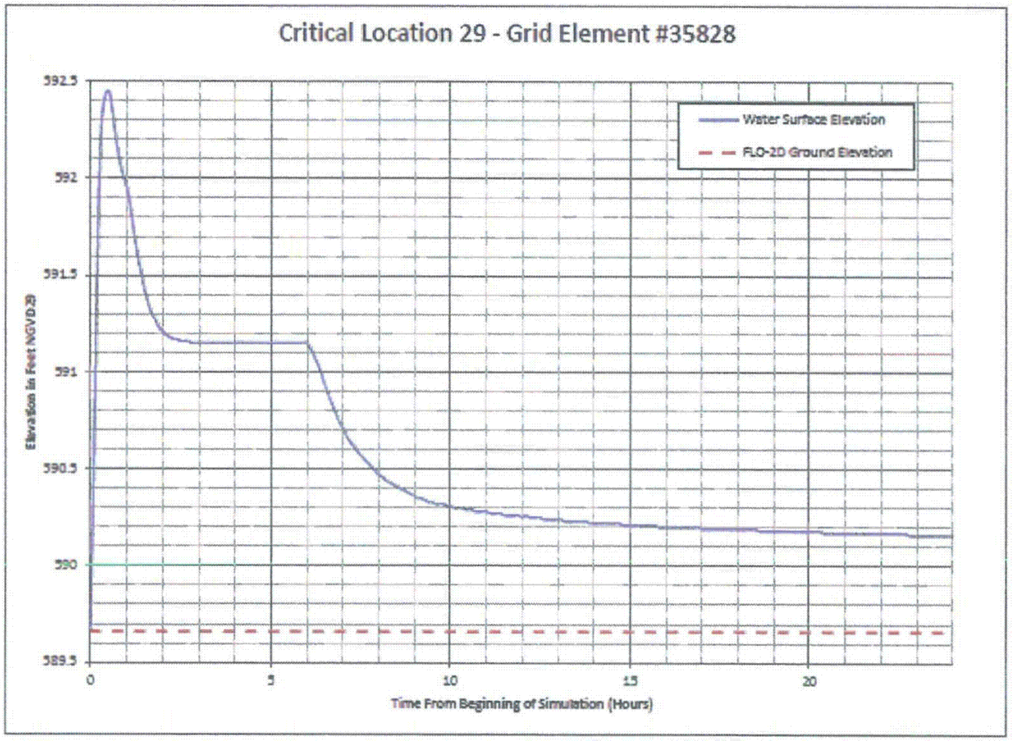
Containment Post Tension Tunnel Hatch (Door #10A)



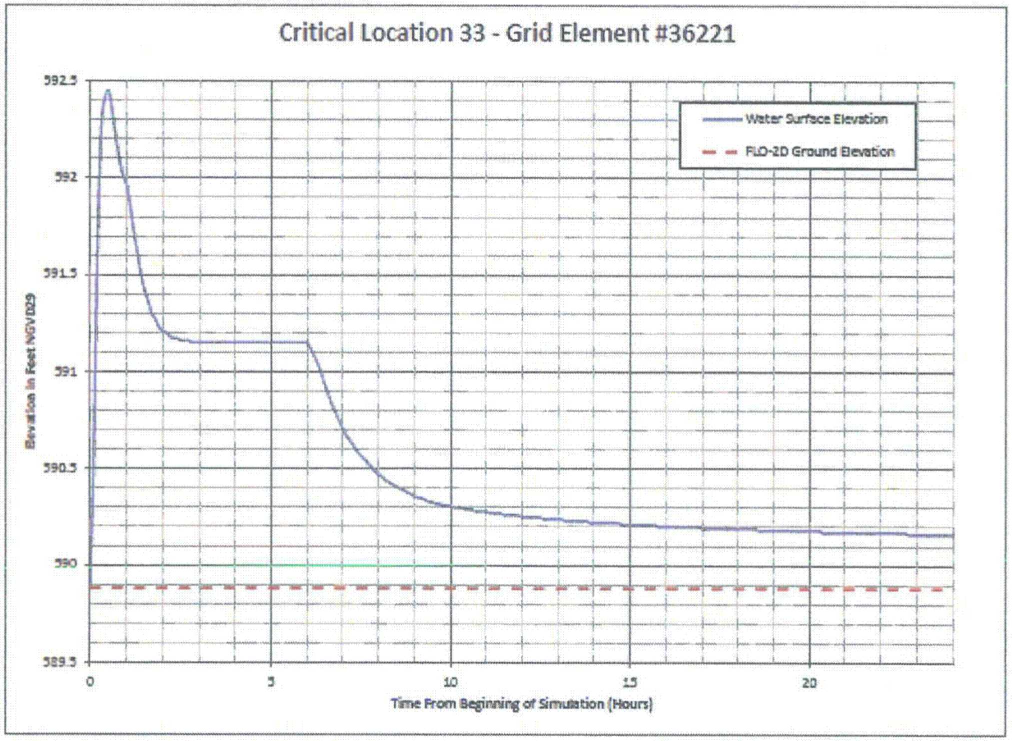
Manhole #4 (East of Containment Building)



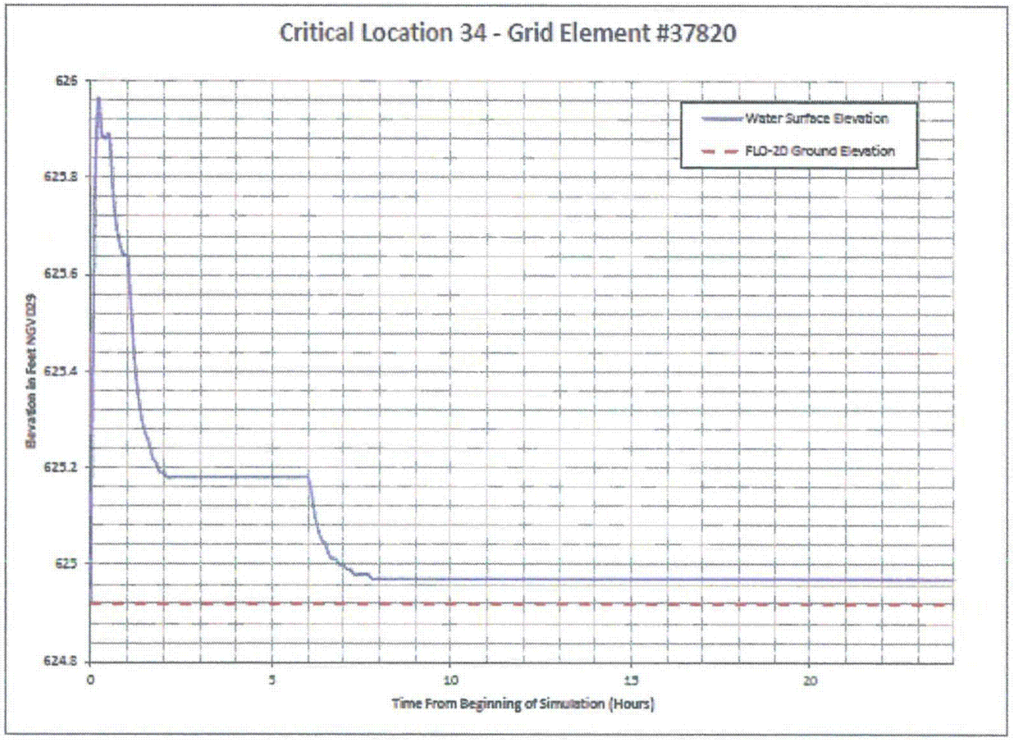
North Chained Double Door to Diesel Generators (Door #170)



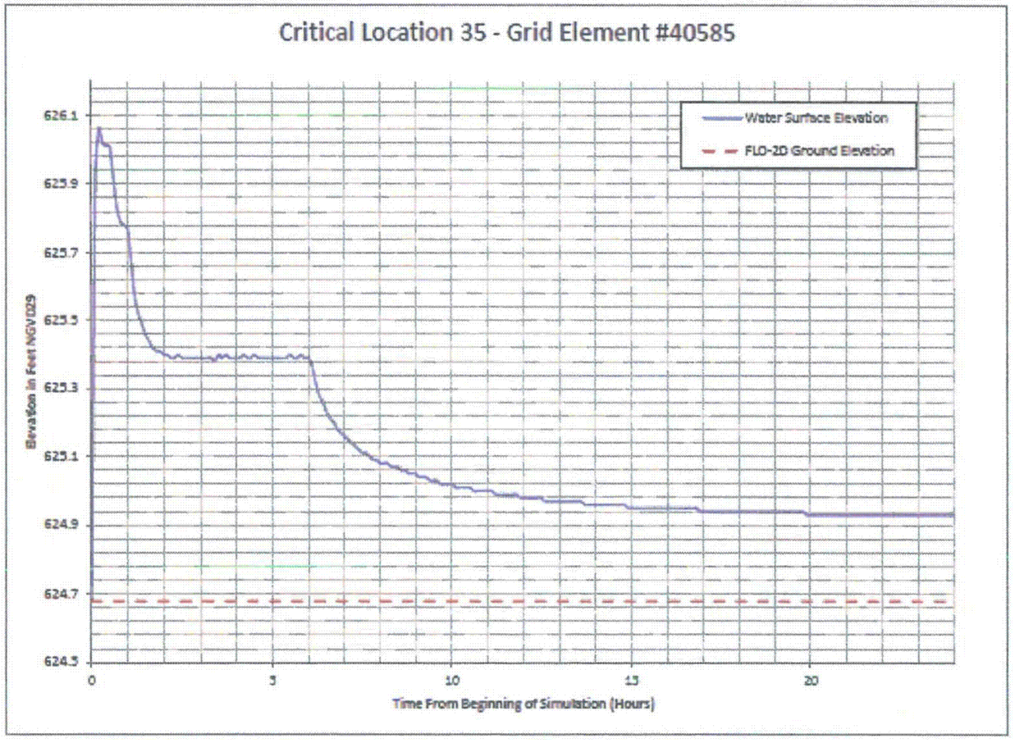
Turbine Building North Entrance Door



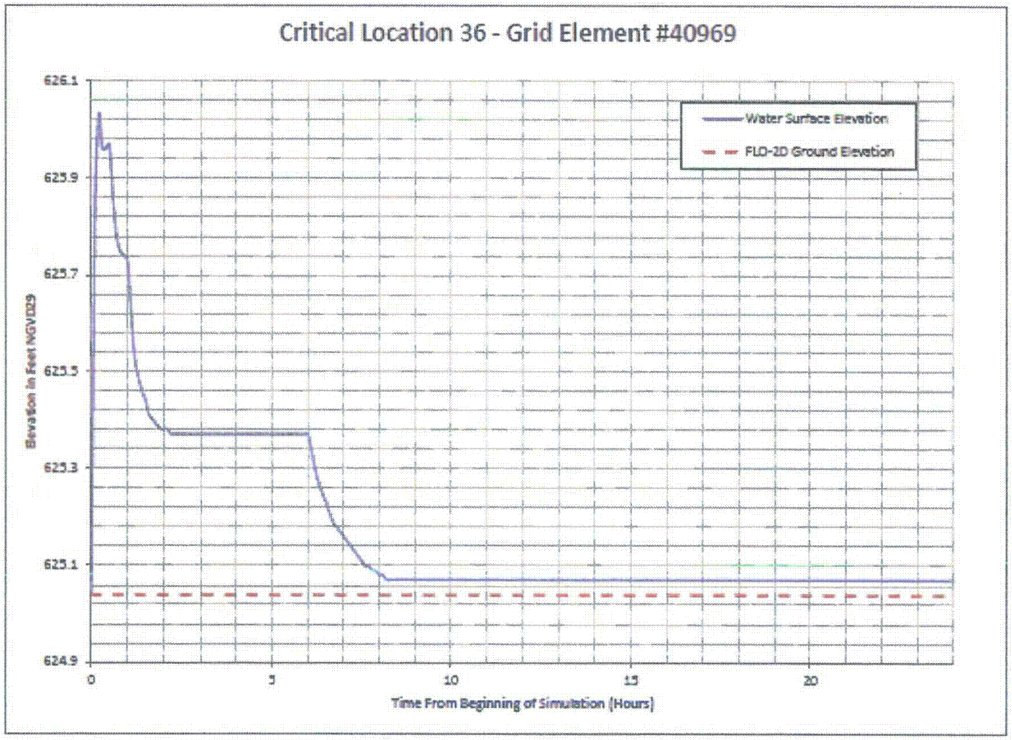
Track Alley Rollup Door



Admin Building Hallway East Entrance (Door #28)



North Penetration Room (Door #106)



South Stairwell (Service Bldg Addition) Across from Elevator (Door #123)