Entergy

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**John A Dent, Jr.**
Site Vice President

March 12, 2015

U.S. Nuclear Regulatory Commission
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SUBJECT: Entergy's Required Response of the Near-Term Task Force Recommendation 2.1: Flooding-Hazard Reevaluation Report

Pilgrim Nuclear Power Station
Docket No. 50-293
License No. DPR-35

REFERENCE: NRC Letter, "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", dated March 12, 2012 (ADAMS Accession No. ML12073A348)

LETTER NUMBER 2.15.016

Dear Sir or Madam:

On March 12, 2012, the NRC issued the referenced letter requesting information 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. Enclosure 2 of the referenced letter contains specific requested actions, requested information, and required responses associated with Recommendation 2.1: Flooding.

Pursuant to Required Response 2 of Enclosure 2, Entergy is providing the Hazard Reevaluation Report for Pilgrim Nuclear Power Station in Attachment 1.

This letter contains no new regulatory commitments.

Should you have any questions concerning the content of this letter, please contact Mr. Everett (Chip) Perkins Jr. at (508) 830-8323.

A010
NRR

I declare under penalty of perjury that the foregoing is true and correct; executed on March 12, 2015.

Sincerely,

John A. Dent Jr.
Site Vice President

JAD/rmb

Attachment: Pilgrim Nuclear Power Station Flooding Hazard Reevaluation Report

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**ATTACHMENT**

**To**

**PNPS Letter 2.15.016**

# PILGRIM NUCLEAR POWER STATION FLOODING HAZARD REEVALUATION REPORT

# AREVA Inc.

## Engineering Information Record

**Document No.:** 51 - 9226940 - 000

## Pilgrim Nuclear Power Station
## Flood Hazard Re-Evaluation Report

Pilgrim Nuclear Power Station
Flood Hazard Re-Evaluation Report

Safety Related? ☒ YES ☐ NO

Does this document establish design or technical requirements? ☐ YES ☒ NO

Does this document contain assumptions requiring verification? ☐ YES ☒ NO

Does this document contain Customer Required Format? ☐ YES ☒ NO

**Signature Block**

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## 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 Pilgrim Nuclear Power Station (PNPS).

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

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 storm surge and wave effects) was also evaluated. Flooding due to local intense precipitation and the combined effect flood are the only flood mechanisms that result in inundation at PNPS; however, plant walkdowns have confirmed that inundation associated with these two flood events will not impact systems, structures or components important to safety.

## Overview

This report describes the approach, methods, and results from the re-evaluation of flood hazards at the Pilgrim Nuclear Power Station (PNPS). 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 this report.

Section 2.0 describes detailed PNPS 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 PNPS 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 flood mitigating actions are planned.

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 |
| ADCIRC+SWAN | Advanced Circulation plus Simulating Waves Nearshore |
| AGMTHAG | Atlantic and Gulf of Mexico Tsunami Hazard Assessment Group |
| ANSI | American National Standards Institute |
| ARC | Antecedent Rainfall Condition |
| ASCE | American Society of Civil Engineers |
| ASCII | American Standard Code for Information Interchange |
| ASPRS | American Society for Photogrammetry and Remote Sensing |
| AWL | Antecedent Water Level |
| CAP | Corrective Action Program |
| CEM | Coastal Engineering Manual |
| CFR | Code of Federal Regulations |
| CLB | Current License Basis |
| CN | Curve Number |
| CO-OPS | Center for Operational Oceanographic Products and Services |
| DEM | Digital Elevation Model |
| DTM | Digital Terrain Model |
| DUT | Delft University of Technology |
| EDG | Emergency Diesel Generator |
| EVA | Extreme Value Analysis |
| FEMA | Federal Emergency Management Agency |
| FSAR | Final Safety Analysis Report |
| GIS | Geographic Information Systems |

| Acronym/Abbreviation | Description |
|---|---|
| GPD | Generalized Pareto Distribution |
| HEC | Hydrologic Engineering Center |
| HELB | High Energy Line Break |
| HHA | Hierarchical Hazard Assessment |
| HMR | Hydrometeorological Report |
| HUC | Hydrologic Unit Code |
| HURDAT | Hurricane Database |
| IJC | International Joint Commission |
| IPEEE | Individual Plant Examination of External Events |
| ISFSI | Independent Spent Fuel Storage Installation |
| ISG | Interim Staff Guidance (NRC) |
| LiDAR | Light Detection and Ranging |
| LIP | Local Intense Precipitation |
| MA | Massachusetts |
| MSL | Mean Sea Level |
| MLW | Mean Low Water |
| 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 |
| NHC | National Hurricane Center |

| Acronym/Abbreviation | Description |
|---|---|
| NHD | National Hydrography Database |
| NID | National Inventory of Dams |
| NOAA | National Oceanic and Atmospheric Administration |
| NRC | U.S. Nuclear Regulatory Commission |
| NRCS | Natural Resources Conservation Service |
| NTTF | Near-Term Task Force |
| NWS | National Weather Service |
| PDF | Probable Density Function |
| PDH | Probable Density Histogram |
| PMF | Probable Maximum Flood |
| PMH | Probable Maximum Hurricane |
| PMP | Probable Maximum Precipitation |
| PMSS | Probable Maximum Storm Surge |
| PMT | Probable Maximum Tsunami |
| PMWS | Probable Maximum Wind Storm |
| PNPS | Pilgrim Nuclear Power Station |
| POT | Peak over Threshold |
| RMSE | Root Mean Square Error |
| SCS | Soil Conservation Service |
| SLOSH | Sea, Lakes and Overland Surges |
| SLR | Sea Level Rise |
| SMF | Submarine Mass Failure |
| SSCs | Structures, Systems and Components |
| SWAN | Simulating Waves Nearshore |

| Acronym/Abbreviation | Description |
|---|---|
| USACE | U.S. Army Corps of Engineers |
| USGS | U.S. Geological Survey |
| UTC | Coordinated Universal Time |
| WIS | Wave Information Studies |
| WSEL | Water Surface Elevation |
| WRT | Wind Risk Tech |
| 1-min, 10-m | 1-minute, 10-meter |
| fdir | forward direction |
| fspd | forward speed |
| fps | feet per second |
| Hg | Mercury |
| km | kilometer |
| kt | knots |
| mb | millibars |
| $mi^2$ | square mile |
| mph | miles per hour |
| mxw | average wind speed |
| nm | nautical miles |
| psf | pounds per square foot |
| psi | pounds per square inch |

# 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 Pilgrim Nuclear Power Station (PNPS), located on the western shore of Cape Cod Bay in the Town of Plymouth, Plymouth County Massachusetts, 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 PNPS.

## 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 PNPS.

Each of the re-evaluated flood causing mechanisms and the potential effects on the PNPS 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 PNPS. 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

Elevations listed as mean sea level (MSL) or mean low water (MLW) in this report refer to elevations provided in plant documentation such as the FSAR. The datum relationship at PNPS is as follows:

MSL + 4.78 feet = MLW (PNPS 2013, Section 2.4.4.2).

Pursuant to United States Army Corps of Engineers (USACE) document EM 1110-1-1005 Appendix C (USACE 2007), National Geodetic Vertical Datum of 1929 (NGVD29) was originally named the Mean Sea Level Datum of 1929. Therefore, elevations in NGVD29 in this report are equivalent to elevations in MSL (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 NAD83 Massachusetts State Plane (horizontal) datum and elevations are in North American Vertical Datum of 1988 (NAVD88) (vertical) datum. The unit of the survey is U.S. feet (AREVA 2014 and AREVA 2015). In order to compare plant document elevations against the 2014 topographic survey results, a conversion factor of 0.827 feet was added to the NAVD88 elevations to obtain the NGVD29 (and MSL) elevations (AREVA 2015). The conversion factor between NAVD88 to NGVD29 was determined through the National Geodetic Survey (NGS) VertCon tool (NGS 2014).

## 1.6 References

**AREVA 2014**. AREVA Document No. 38-9226913-000, PNPS Topographic Survey Deliverables, 2014.

**AREVA 2015**. AREVA Document No. 32-9226914-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Local Intense Precipitation, 2015.

**NGS 2014**. VERTCON – North American Vertical Datum Conversion, National Geodetic Survey, http://www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html, Date accessed: August 2, 2014, Date modified: January 24, 2013. (See AREVA Document No. 32-9226914-000)

**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. (ADAMS Accession No. ML11321A195)

**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. (ADAMS Accession No. ML12053A340)

**PNPS 2013**. Pilgrim Nuclear Power Station Final Safety Analysis Report (FSAR), Revision 29, October 2013. (AREVA Doc. No. 38-9226908-000)

**USACE 2007**. Development and Implementation of NAVD88, EM 110-1-1005 Appendix C, U.S. Army Corps of Engineers, January 1, 2007. (See AREVA Document No. 32-9226914-000)

# 2.0 INFORMATION RELATED TO THE FLOOD HAZARD

## 2.1 Site Information

PNPS is situated on the western shore of Cape Cod Bay in the Town of Plymouth, Plymouth County, Massachusetts, and encompasses approximately 517 acres (PNPS 2013, Section 2.2.1). See Figure 2-1, Site Location Map. The PNPS site is surrounded by hills to the north, south and west, and is situated on the northeast side of Pine Hills, a north-south trending glacial ridge approximately four miles long with a maximum elevation of 395 feet (PNPS 2012, Section 2.0 and PNPS 2013, Sections 2.4.1.2 and 2.5.2.4.1).

### 2.1.1 Site Layout

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

### 2.1.2 Site Topography

The PNPS site varies from 14 to 32 feet above MSL. Station grade is at 20 feet MSL. The elevation in the vicinity of the power block is at 22 feet MSL and the building floor elevation at grade level is 23 feet MSL. The 40 foot MSL ground surface contour crosses Rocky Hill Road, a public road, and closes within the site boundaries and is open only to the bay. The 24 foot MSL contour closes on the bay side of Rocky Hill Road. (PNPS 2012, Section 2.0 and PNPS 2013, Sections 2.4.1.2, 2.5.2.1 and 2.5.2.4.1).

It is unlikely that the PNPS shoreline will experience changes due to shoreline erosion since the PNPS shoreline is stabilized against erosion from wind, currents, water fluctuations and storm conditions by a series of breakwaters and discharge channel jetties constructed of heavy rock (AREVA 2014). For further discussion, refer to Section 3.8.

The groundwater table generally follows the site surface topography, resulting in moderately steep groundwater gradients present beneath the PNPS site with flow towards Cape Cod Bay (PNPS 2013, Sections 1.6.1.1.6 and 2.4.1.3.2). The Reactor, Turbine and Radwaste Buildings have a waterproofing membrane designed to prevent or minimize groundwater in leakage (PNPS 2013, Section 12.2.4.4.3).

## 2.2 Current Design Basis Flood Information and Elevations

The current design basis flood is described in the PNPS FSAR (PNPS 2013, Section 2.4.4) and in the Pilgrim Nuclear Power Station Flooding Walkdown Submittal Report for resolution of Fukushima NTTF Recommendation 2.3 (PNPS 2012) required as part of the response to the 10 CFR 50.54(f) letter.

The PNPS design basis flood is the extreme design storm tide level of 13.5 feet MSL (18.3 feet MLW).

To assist in the design of PNPS waterfront structures (i.e., dredged channels, breakwaters, jetties and onshore revetments), a series of wave action model studies were performed in which the waterfront was subjected to three still water elevations: 1) 11 feet MSL (15.8 feet MLW), the level of the 100 year storm; 2) 13.5 feet MSL (18.3 feet MLW), the design maximum storm level, and; 3) 14.7 feet MSL (19.5 feet MLW), an arbitrary elevation which exceeds any postulated design condition and is the highest elevation at which the model could be operated satisfactorily (PNPS 2013, Section 2.4.4.3). The top elevation of the breakwaters and the nominal elevation for the discharge channel jetties are at 11.2 feet MSL (16 feet MLW) (PNPS 2013, Section 2.4.4.1). The top of the shorefront revetment is at elevation 20.2 feet MSL (25 feet MLW) (PNPS 2012 Section 2.0).

The probable maximum precipitation (PMP) event at PNPS was evaluated as part of the Individual Plant Examination of External Events (IPEEE). The PMP event results in flood depths of 24.5 feet MSL along the south side of plant buildings, 22.5 feet MSL along the north side of plant buildings, and ponding on building roofs of about 0.5 feet (PNPS 2012).

### 2.2.1 Elevations of Safety Structures, Systems and Components

The minimum entrance elevation for areas housing SSCs important to safety is 23 feet MSL (see Section 3.1).

## 2.3 Current Flood Protection

The CLB for flooding protection at PNPS is described in the FSAR (PNPS 2013, Section 2.4.4). Flood protection features for the PMP, evaluated as part of the IPEEE, are described in the 2012 walkdown report (PNPS 2012).

### 2.3.1 Current Flood Causing Mechanisms

The following is a summary of current flood causing mechanisms for PNPS.

#### 2.3.1.1 Extreme Storm Tide Level

The extreme storm tide event is the only CLB flood hazard. The maximum storm tide level of 13.5 feet MSL (18.3 feet MLW) may result from a tropical or an extra-tropical event (i.e., a nor'easter or a hurricane). For a standard project nor'easter for New England, established by the hydrometeorological section of the U.S. Weather Bureau, the extreme design storm tide level is based on a peak storm surge of 6.6 feet coincident with a high tide of 6.9 feet MSL. Similarly, for the most severe hurricane parameters from Hydrometeorological Branch Memorandum HUR 7-97, including a spring high tide of 6.9 feet MSL, the maximum hurricane produced storm surge results in a still water level of 13.5 feet MSL (18.3 feet MLW). (PNPS 2012 Section 2.0 and PNPS 2013, Section 2.4.4)

#### 2.3.1.2 Probable Maximum Precipitation

Although not part of the CLB, the PMP event was evaluated as part of the IPEEE and exceeds the CLB extreme storm tide level. PMP water depths along the power block buildings are based on one hour precipitation rates having a probability of occurrence of $1 \times 10^{-6}$ per year. The rainfall rates were developed from the National Weather Service HYDRO-35 report, and the U.S. Army Corp of Engineers (USACE) Flood Hydrograph Package HEC-1 was used to develop the runoff flowrate. The duration of the PMP event is one hour, during which time the flood level starts at zero height, increases to PMP levels, and then recedes back to zero height. The PMP results in water depths slightly (i.e., up to 1.5 feet) above power block building door sill elevations, and in roof ponding (PNPS 2012).

### 2.3.2 Current Flood Protection and Mitigation Features

Referring to Figure 2-2, the breakwaters protect the Intake Structure and revetment from excessive wave action and overtopping due to wave runup; they also minimize on-site flooding from storms. The revetments on either side of the Intake Structure provide shore stabilization and prevent the Reactor Building from being flooded during severe storms. Although open ocean wave heights up to 31 feet were generated during the model wave studies noted in Section 2.2 above, the test results demonstrated that the generated waves at the Intake Structure would be adequately reduced and that there would be no adverse impact to the service water pumps for still water elevations up to 14.7 feet MSL (19.5 feet MLW); in addition, the Reactor Building would not be subjected to flooding (PNPS 2013, Section 2.4.4.3).

Flooding protection against the PMP event includes exterior doors on power block buildings, roof drains and internal seals for conduits originating in manholes (PNPS 2012, Section 3.3). Although door sills on the south side of the plant would be 1.5 feet below the maximum predicted PMP flood depth, an evaluation determined that the doors could withstand the corresponding hydrostatic load. Potential water intrusion through door perimeters was also evaluated and found to be bounded by the plant's internal flooding analysis. Additionally, it was determined that roof ponding during a PMP event would not exceed roof design capacity if the roof drains were fully functioning. It was also determined that water ingress via manholes would be prevented by cable to conduit seals, or mitigated by tortuous conduit pathways. (PNPS 2012, Section 3.1).

Although no actions or procedures are credited for flooding protection, the plant's procedure for operation during severe weather (i.e., PNPS Procedure 2.1.42, Operation During Severe Weather) includes measures that can be used for mitigating external flood conditions (e.g., ensuring that exterior doors are closed, installing sandbags at door bottoms and drain scuppers) (PNPS 2012, Sections 3.1 and 5.2.2).

## 2.4 Licensing Basis Flood-Related and Flood Protection Changes

Following damage to the breakwaters during the winters of 1977-1978 and 1978-1979, the breakwaters were repaired both times to their original configuration. Resolution to NRC concerns included a commitment to monitoring of the breakwaters to ensure their integrity (PNPS 2013, Section 2.4.4.1).

## 2.5 Watershed and Local Area Changes

### 2.5.1 General PNPS Site Hydrological Description

There are no perennial or intermittent streams in the vicinity of PNPS. The closest hydrologic feature, Bartlett Pond, is approximately one and three-quarter miles southeast of the PNPS site (USGS 2012). Due to local topography, the PNPS drainage basin is isolated from other area watershed basins. All site surface drainage flows into Cape Cod Bay (PNPS 2013, Section 2.4.1.2).

### 2.5.2 Watershed Changes

Considering that there are no perennial streams on or adjacent to the PNPS site, no significant changes to the PNPS watershed were identified (USGS 1977 and USGS 2012).

### 2.5.3 Local Area Changes

The 2012 walkdown report indicates that plant changes since the time that the PMP analysis was performed, include the installation of security fences along the bay, and construction of the Engineering and Plant Support Building on the east side of the plant. However, despite these changes, the 2012 walkdown report found that the overland drainage path for the PMP analysis (i.e., rain water flowing around the east and west sides of plant structures and then northward over the shore revetment), still remained. Therefore, the 2012 walkdown report concluded that any changes in the overland drainage path due to plant changes, would be offset by conservatisms in the PMP analysis (PNPS 2012, Section 5.2).

Since the addition and relocation of PNPS vehicle security barriers may also impact localized surface water drainage, refer to the local intense precipitation evaluation in Section 3.1 for further discussion.

## 2.6 Additional Site Details – Walkdown Results

A total of 33 walkdown flood protection features, including 138 attributes, were reviewed during the 2012 walkdown. Of the total flood protection features, 28 were defined as passive – incorporated, none as passive – temporary, five as active – incorporated and none as active – temporary. The walkdown scope included a visual inspection of shorefront features, doors and conduit seals. The 2012 walkdown report indicates that the roof drains were not inspected at that time, since it was determined that the roof drains are periodically inspected and cleaned by plant maintenance personnel. Observations that did not meet the walkdown acceptance criteria were entered into the plant's corrective action program and subsequently rectified. (PNPS 2012, Section 7.0)

## 2.7 References

**AREVA 2014**. Channel Diversion AREVA Document No. 51-9226930-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Screening for Channel Diversion, October 2014.

**PNPS 2012**. Engineering Report No. PNPS-CS-12-00002, Pilgrim Station Flooding Walkdown Submittal Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding, Rev. 1. (see AREVA Doc. No. 38-9226908-000)

**PNPS 2013**. Pilgrim Nuclear Power Station Final Safety Analysis Report (FSAR), Revision 29, October 2013. (see AREVA Doc. No. 38-9226908-000)

**USGS 1977**. Manomet Topographic Quadrangle Map 1977, Massachusetts-Plymouth County, 7.5 Minute Series, Scale 1: 25 000, U.S. Geological Survey. (See AREVA Document No. 51-9226922-000)

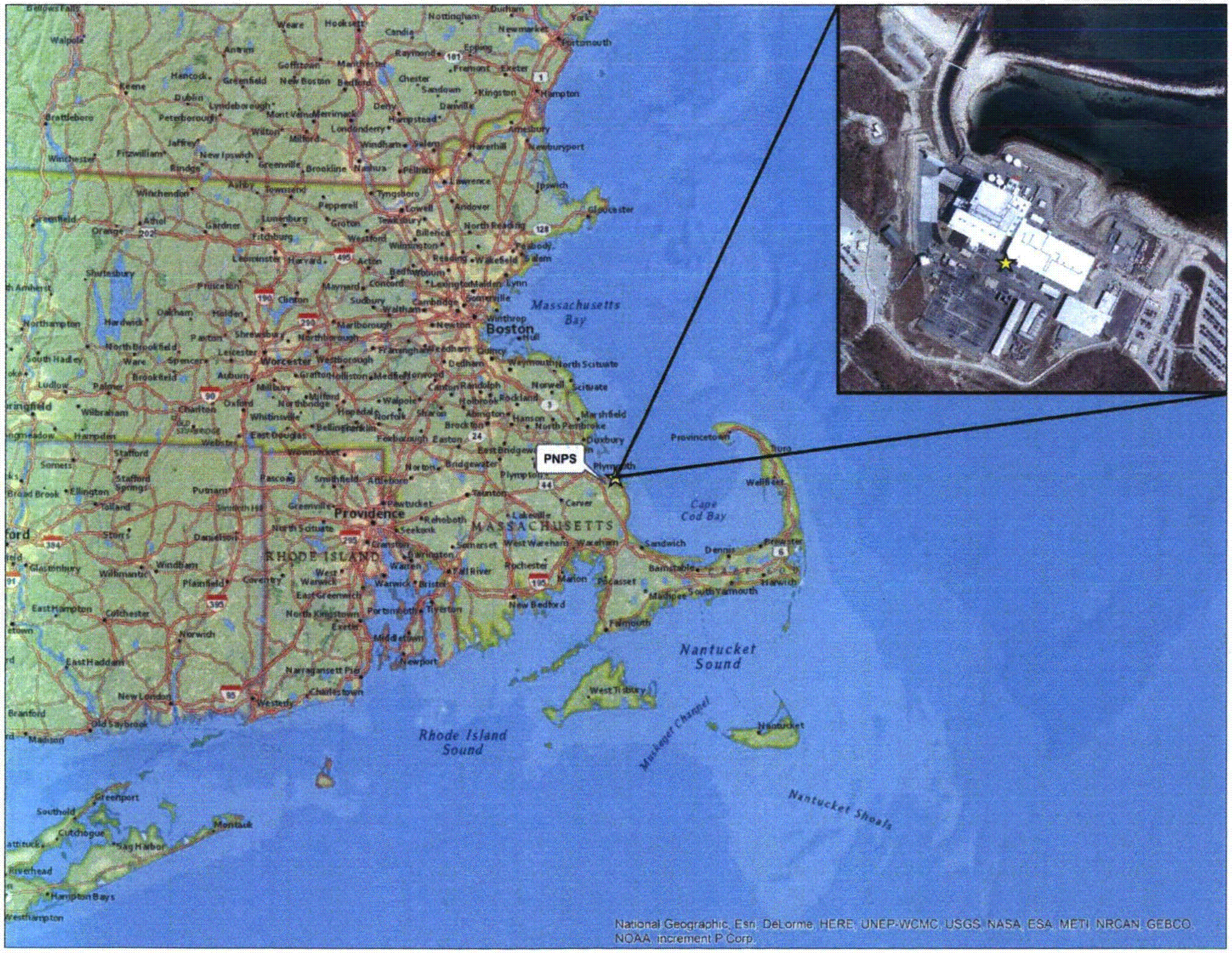
**USGS 2012**. Manomet Topographic Quadrangle Map 2012, Massachusetts-Plymouth County, 7.5 Minute Series, Scale 1: 24 000, U.S. Geological Survey. (See AREVA Document No. 51-9226922-000)

AREVA

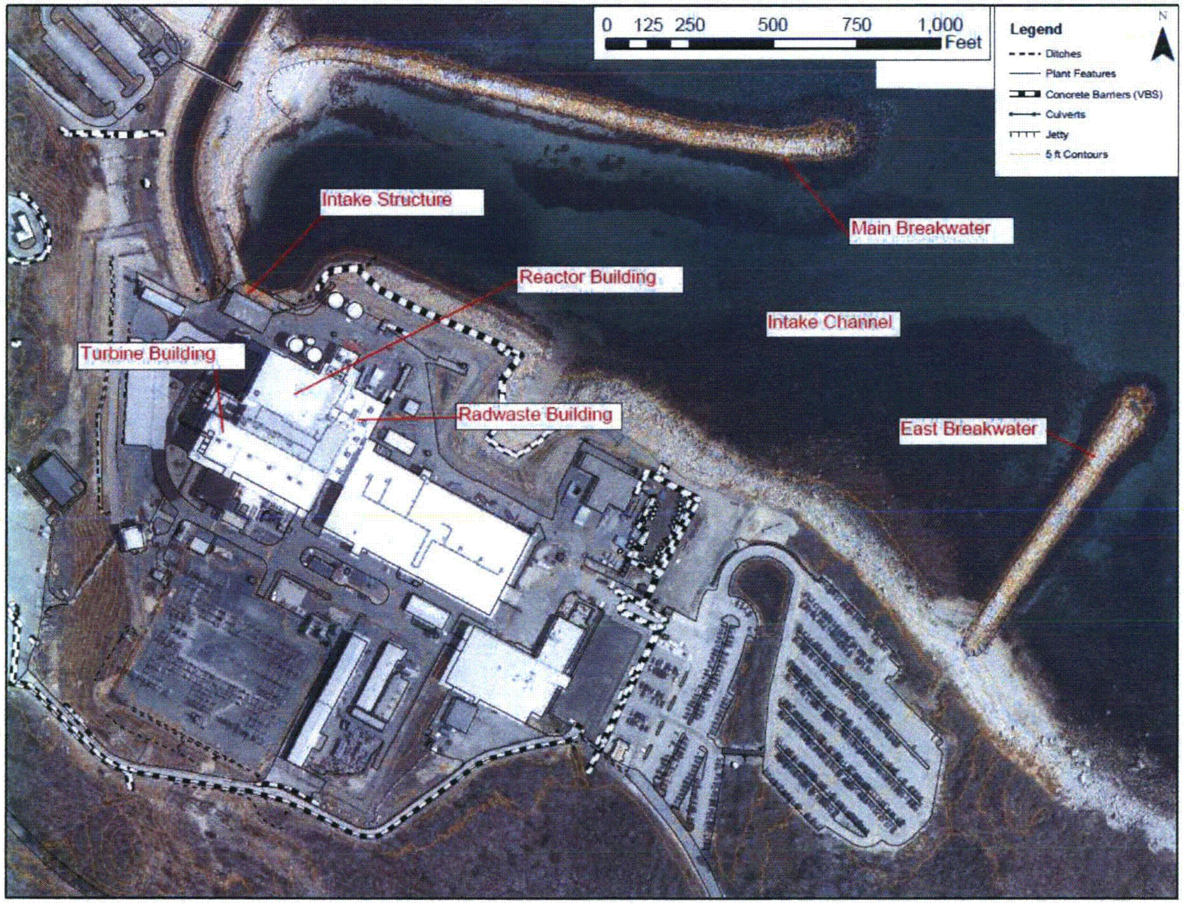
Document No.: 51-9226940-000



**Figure 2-1: Site Location Map**



## Figure 2-2: Site Topography and Layout



# 3.0 FLOOD HAZARD RE-EVALUATION

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

## 3.1 Local Intense Precipitation

This section addresses the potential for flooding at PNPS 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 Document No. 32-9226914-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Local Intense Precipitation (AREVA, 2014a).

### 3.1.1 Methodology

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 elevations at PNPS.

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

1. Develop LIP/PMP inputs.
2. Develop a FLO-2D hydrodynamic computer model with site features.
3. Perform flood simulations in FLO-2D and estimate maximum water surface elevations throughout the PNPS site.

### 3.1.2 Results

Recent survey data has confirmed that the minimum entrance elevation for all areas housing structures important to safety is 23.0 feet MSL (IPEEE 1994, Section 5.2.1). Maximum LIP flood elevations calculated by FLO-2D (Table 3-1) for selected important locations indicate flooding above elevation 23.0 feet MSL at several areas, with a maximum water surface elevation of 25.2 feet MSL. The highest LIP water surface elevations occur on the south side of the plant structures. Important locations examined in the LIP flood model are shown on Figure 3-1.

#### 3.1.2.1 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). See Appendix A. The FLO-2D model computational boundary is shown in Figure 3-2. The computational domain of the FLO-2D model encompasses the PNPS plant and its peripheral site features/structures. The model computational boundary also includes the PNPS site drainage basin; therefore, the LIP analysis bounds a PMF evaluation that considers only the PNPS drainage area, which is less than the LIP model boundary. The computational domain of the FLO-2D model is bounded by Cape Cod Bay and topographic ridges to the north, south and west. The total FLO-2D model extent is approximately 600 acres (0.94 square miles), while the PNPS drainage area is approximately 337 acres (BEC 1993).

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

### 3.1.2.2 FLO-2D Computer Model with Site Features

Site Topography and Buildings Alignment: Current site topography (digital terrain model (DTM)) and buildings alignment at PNPS were extracted from the site topographic survey provided in AutoCAD™ format (AREVA 2014c). Additional elevation data was used based on the topographic site plan (AREVA 2014c) produced along with the DTM. Topographic data for PNPS was 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 PNPS 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 and DTM points, at well-defined points. Additional designated important structures and locations with respect to site flooding impacts were identified and surveyed 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 important structures and locations (AREVA 2014d).

FLO-2D grid element elevation data was interpolated based on imported digital terrain (DTM) points from the topographic survey of the site that were added to the working region. The interpolation method used in the FLO-2D model was an inverse distance weighting formula exponent to assign elevations to the grid element from the DTM points.

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 spot elevations on the DTM provided for adequate coverage for each grid element.

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. LIP results were computed as maximum water surface depths, which were then compared to the known height of flood protection at important elements, thus reducing uncertainty related to potential issues with elevation datum normalization.

Levee Elements: The concrete shoreline barriers, which impede flow away from the site, were modeled as levee structures using the levee component within FLO-2D (FLO-2D 2013). The top elevation of the concrete shoreline barriers was interpolated between surveyed points from the additional topographic survey (AREVA 2014c). Simulation of the LIP with the concrete shoreline barriers results in a more conservative water surface elevation than without the concrete shoreline barriers as the barriers would prevent water from flowing freely into Cape Cod Bay. Vehicle barrier systems were conservatively not included in the model as they would only impede flow and are not considered flood protection elements.

Water Surface Elevation at Cape Cod Bay: With the general site grade at 20 feet MSL and the disparity between the highest still water surface elevation of 14.8 feet MSL, as listed in the PNPS FSAR (PNPS 2013), the original digital elevation model (DEM) water surface elevations (approximately 0.0 feet NAVD88) were unchanged for modeling simplicity.

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 2014d). 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 the 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 0.02 for concrete or paved areas to 0.4 for wooded areas. Figure 3-3 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. A land cover GIS shapefile 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).

A GIS shape file was created with CN values assigned by correlating land cover types with soil types 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 ground cover was assessed using aerial photography to determine the appropriate hydrologic condition.

Table 3-3 shows the relationship between CN values and selected land cover categories. The CN values for the grid elements generally range from 43 for wooded and brush areas to 99 for concrete or paved areas. Note that the CN values for the wooded areas were conservatively selected as the same as the "brush-brush-forbs-grass" areas (Table 9-1 of NRCS 2004a). Figure 3-4 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 4.4 inches or 18-percent of the total precipitation (25.5 inches) was infiltrated or decreased from the total precipitation before flood routing started.

Buildings and Roof Tops: Buildings at PNPS were incorporated into the FLO-2D model based on the high resolution orthoimagery and the site survey (AREVA 2014a) by changing the grid elevations to incorporate building height into the terrain. Buildings were modeled as elevated grid elements (i.e., higher than surrounding areas) in the FLO-2D model to ensure that rainfall runs off the building rooftops to the surrounding areas and to ensure overland flow around the buildings (i.e., not through the buildings). Building roof top elevations were assigned to represent the approximate runoff pattern (i.e., runoff flows from higher elevation roof top to lower elevation roof top), based on roof top elevation points within the "DTM-Buildings" layer of the surveyed DTM (AREVA 2014a). Area Reduction Factors and Width Reduction Factors were not used. This methodology accounts for the contribution of roof drainage on the ground surface runoff in accordance with Section 11.4 of ANSI/ANS 2.8 – 1992 (ANS 1992). Elevating building cells allows for FLO-2D to recognize those grid elements are obstructions relative to much lower ground grid elements. To evaluate the worst case for site surface drainage (ANS 1992), roof drains connected to subsurface drainage systems are assumed to be blocked and potential storage resulting from roof parapet walls was conservatively not incorporated.

#### 3.1.2.3 LIP/PMP Inputs

The LIP parameters were defined using Hydrometeorological Report (HMR)-51 and HMR-52; (NOAA 1977 and NOAA 1982, respectively) as prescribed in NUREG/CR-7046 (NRC 2011, Section 3.2). The total rainfall depth for the 1-hour, 1-$mi^2$ probable maximum precipitation (PMP) is 17.1 inches. 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 five hours (Figure 3-5).

### 3.1.2.4 LIP Simulation Results

The results of the LIP simulation are summarized in Table 3-1 and shown in Figure 3-6. Large plots showing the LIP grid element numbers, interpolated ground surface elevations, water surface elevations, depths, velocity and direction are provided in Appendix A.

The maximum LIP flood elevations near important locations range from 22.5 feet NAVD88 (23.3 feet MSL) near the Water Treatment Area ground level door on the west side of the buildings to 24.4 feet NAVD88 (25.2 feet MSL) near the O&M Building ground level door on the south side of the buildings. Calculated maximum flood depths near important locations range from 0.6 feet near the Emergency Diesel Generator Building door on the north side of the buildings to 2.6 feet near the O&M Building ground level door on the south side of the buildings. Maximum flow velocities within the computational area are up to 12.6 feet per second (fps) in the entrance of the discharge channel. This is reasonable given the difference in elevation between the channel water surface elevation and the ground surface elevation at the head of the channel. The channel is stone lined (BEC 1969) while the majority of the power block area is concrete. According to Table 2 of Fischenich 2001, the maximum permissible mean velocity threshold for large diameter stone is greater than 14 fps; while for concrete areas it is greater than 18 fps. Therefore, scour and erosion in the power block and channel is not expected.

Appendix A contains the stage hydrographs near important locations. Peak flood levels occur well after the peak rainfall intensity, which can be attributed to the contribution of off-site drainage (i.e., lag). Additionally, there is a smaller peak at the beginning of the stage hydrographs approximately coincident with the peak rainfall intensity. Generally, the maximum flood depths occur within the first two hours of the simulation. In some instances, these flood depths can take over ten hours to recede, specifically on the south side of the powerblock area.

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 0.000004 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 601.1 acres. The FLO-2D model is made up of 65,499 grid elements each 20 feet by 20 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 601.5 acres ((20 x 20 x 65,499) x (1 acre / 43,560 feet)). The FLO-2D calculated maximum inundation area virtually 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 or "VELTIMEC" (channel) 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 10.1 fps reported in the "VELTIMEFP.OUT" file and occurred at the interface between the floodplain and discharge channel. A similar review of the "VELTIMEC.OUT" file does not indicate unreasonably high velocities in the model runs. The maximum velocity is up to 12.6 fps, which

occurs at the drop off between the site grade and the channel. This indicates a successful project application.

#### 3.1.2.5 Review Areas of Supercritical Flow

FLO-2D does not simulate supercritical flow conditions (FLO-2D 2013). No Froude number limitations were used; therefore, FLO-2D does not adjust the Manning's roughness'n' coefficient. Grid elements are determined to be supercritical if the calculated Froude number is greater than 1.0 (FLO-2D 2013). The SUPER.OUT file reports grid elements that are supercritical. Review of the SUPER.OUT file indicated that supercritical flow is occurring generally at the intersection of building grid elements and grid elements representing the adjacent grade (see Figure 3-7) possibly due to the artificially high hydraulic gradient created by elevating grid elements to represent buildings. 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.

### 3.1.3 Conclusions

The maximum water surface elevations at the site due to the LIP at PNPS result from a PMP depth of 17.1 inches in 1 hour and 25.5 inches within six hours. The maximum flood depths range from 0.6 feet to locally as high as approximately 2.6 feet above grade near the important locations as shown in Table 3-1. The maximum LIP flood elevation at an important location examined in the LIP analysis is 24.4 feet NAVD88 (25.2 feet MSL).

### 3.1.4 References

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

**AREVA 2014a**. AREVA Document No. 32-9226914-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Local Intense Precipitation, 2014.

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

**AREVA 2014c**. PNPS Mapping Deliveries, June 17, 2014. (See AREVA Document No. 38-9226913-000)

**AREVA 2014d**. PNPS (Pilgrim) Critical Structures CAD Files, August 28, 2014. (See AREVA Document No. 38-9226913-000)

**BEC 1969.** Waterfront Development Detail Plan Intake and Discharge Area", Drawing No. C-416, Revision E1, Boston Edison Company, 1969. (See AREVA Document No. 32-9226914-000)

**BEC 1993.** IPEEE – External Flooding Analysis (Local Intense Precipitation), BEC-039, Boston Edison Company, 1993. (See AREVA Document No. 38-9226908-000)

**Fischenich 2001.** C. Fischenich, Stability Thresholds for Stream Restoration Materials, EMRRP, Publication No. ERDC TN-EMRRP-SR-29, May 2001.

**FLO-2D 2013**. FLO-2D® Pro Reference Manual, FLO-2D Software, Inc., Nutrioso, Arizona (www.flo-2d.com), 2013. (See AREVA No. 32-9226914-000)

**IPEEE 1994**. Pilgrim Nuclear Power Station – Individual Plant Examination for External Events (GL 88-20), Rev. 0, July 1994. (See AREVA Document No. 38-9226908-000)

**NOAA 1977**. Probable Maximum Precipitation Estimates - United States East of the 105th Meridian, Hydrometeorological Report No. 51 (HMR-51) by US Department of Commerce & USACE, August 1977.

**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.

**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. 32-9226914-000, Appendix E.2)

**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. 32-9226914-000, Appendix E.3)

**NRCS 2013.** Web Soil Survey, U.S. Department of Agriculture Natural Resource Conservation Service (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm), revised December 06, 2013, accessed September 26, 2014. (See AREVA Document No. 32-9226914-000, Appendix E.1)

**PNPS 2013**. Pilgrim Nuclear Power Station Final Safety Analysis Report, Revision 29, October 2013. (See AREVA Document No. 38-9226908-000)

**Table 3-1: LIP Model Results**

| Identification Number | Description | Representative Grid Element | Grid Element Ground Surface Elevation (feet, NAVD88) | Maximum Flood Elevation (feet, NAVD88) | Maximum Flood Elevation (feet, MSL) | Maximum Flood Depth (feet) | Time to Maximum Flood Elevation (hours) | Maximum Velocity (feet per second) |
|---|---|---|---|---|---|---|---|---|
| 8 | Emergency Diesel Generator Building Door – North Side | 7467 | 22.1 | 22.7 | 23.5 | 0.6 | 0.1 | 2.7 |
| 9 | Reactor Building Truck Lock Door – West Side | 7169 | 21.8 | 22.5 | 23.3 | 0.7 | 1.5 | 1.7 |
| 10 | Water Treatment Area Ground Level Door – West Side | 7606 | 21.7 | 22.5 | 23.3 | 0.8 | 1.5 | 0.7 |
| 11 | Turbine Building Truck Rollup Door – South Side | 10264 | 21.9 | 24.4 | 25.2 | 2.5 | 1.4 | 0.8 |
| 12 | O&M Building Ground Level Door - South Side | 10085 | 21.8 | 24.4 | 25.2 | 2.6 | 1.4 | 2.0 |
| 13 | Hatch A – Turbine Building – South Side | 10077 | 23.3 | 24.4 | 25.2 | 1.1 | 1.4 | 0.5 |
| 14 | Hatch B – Turbine Building – South Side | 9897 | 23.2 | 24.4 | 25.2 | 1.1 | 1.4 | 1.0 |
| 15 | Air Vent –Redline Building – South Side | 9728 | 22.1 | 24.4 | 25.2 | 2.2 | 1.4 | 1.1 |

Note: Due to rounding, maximum flood depths added to ground surface elevations may not be exactly equal to the maximum flood elevations (NAVD88) indicated above. The variance is within 0.1 feet.

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

| Land Cover Category | Manning's n |
|---|---|
| Paved / Concrete | 0.02 |
| Building Roofs | 0.02 |
| Water | 0.025 |
| Sandy Areas with No Cover | 0.10 |
| Grass | 0.20 |
| Trees | 0.40 |

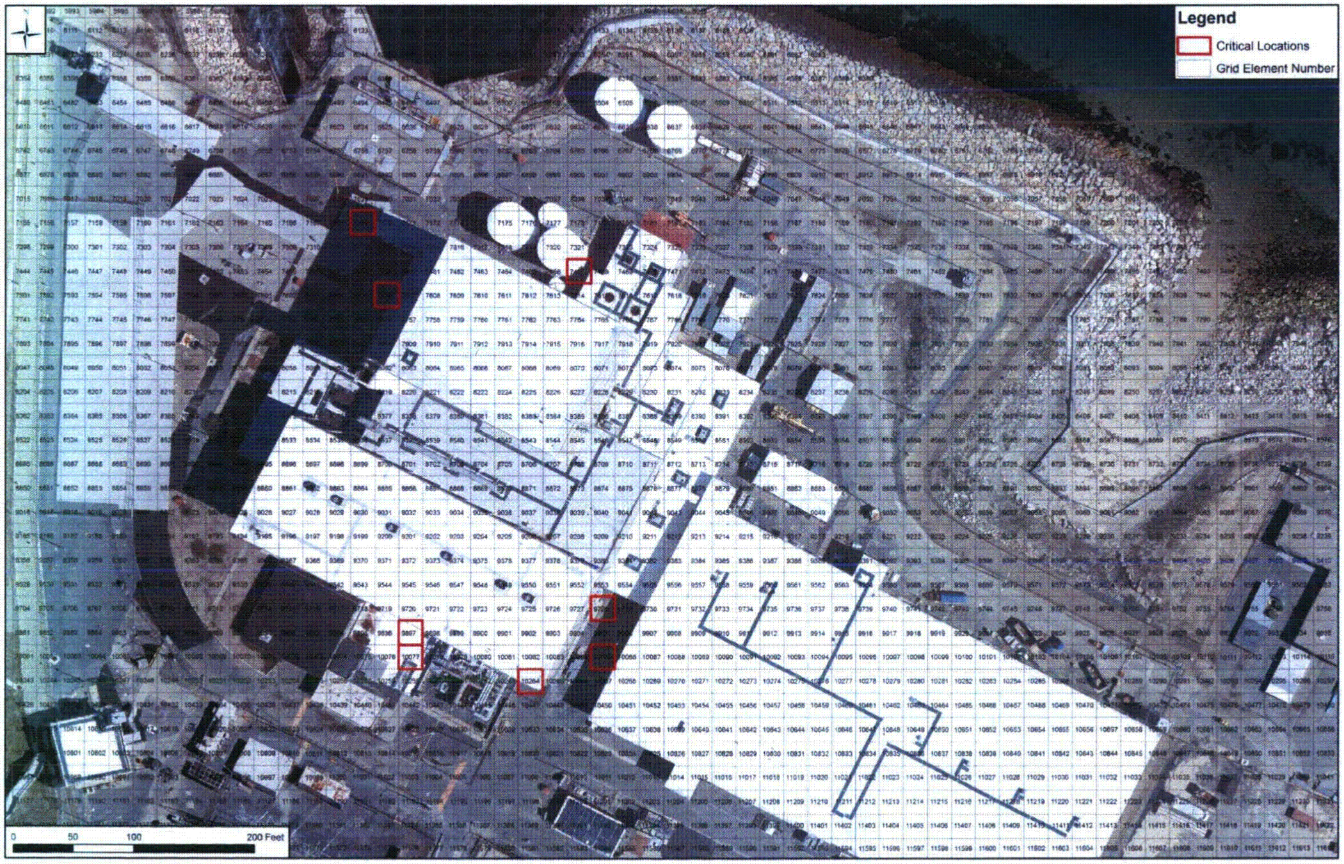
Notes: 1) The Manning's n value for "Open Ground, no Debris" was used for the "Sandy Area" (e.g., dunes) as the reference used for Manning's n (FLO-2D 2013) does not include a land use category related to sand.

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

| | | ARC III CN[2] for HSG | | | |
|---|---|---|---|---|---|
| **Land Cover Category** | **NRCS Cover Type[1]** | **A** | **B** | **C** | **D** |
| Cranberry Bog | Water | 99 | 99 | | 99 |
| Forest | Woods | 43 | 74 | 85 | 89 |
| Forested Wetland | Water | 99 | 99 | 99 | 99 |
| High Density Residential | 1/4 Acre Lot | 78 | 88 | 93 | - |
| Industrial | Impervious | 99 | 99 | 99 | 99 |
| Multi-Family Residential | 1/8 Acre Lot | - | - | 91 | - |
| Non-Forested Wetland | Water | 99 | 99 | 99 | 99 |
| Open Land | Impervious | - | 78 | - | 96 |
| Powerline/Utility | Brush | 50 | 68 | - | 87 |
| Saltwater Sandy Beach | Impervious | 99 | 99 | 99 | 99 |
| Very Low Density Residential | 2 Acre Lot | - | 82 | - | 92 |

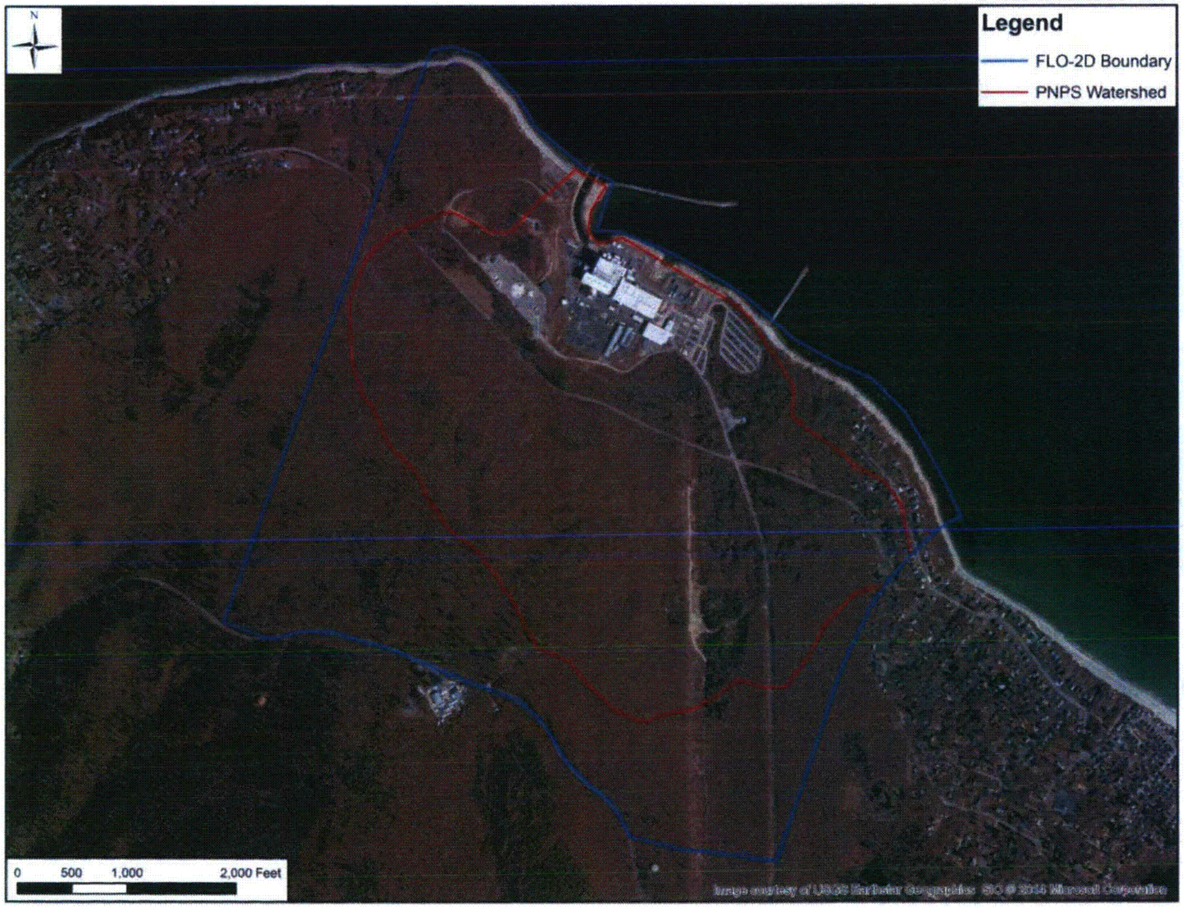
Notes: 1) All NRCS cover types were assumed to be in fair condition. 2) The CN values in the table above for ARC III were taken from Table 10-1 of NRCS 2004b.

**Figure 3-1: PNPS Important Locations**




Basemap Source: High resolution orthoimagery (AREVA 2014c). Note that a larger version of this figure is available in Appendix A.

**Figure 3-2: FLO-2D Computational Boundary**




Basemap Source: High resolution orthoimagery (AREVA 2014c)

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

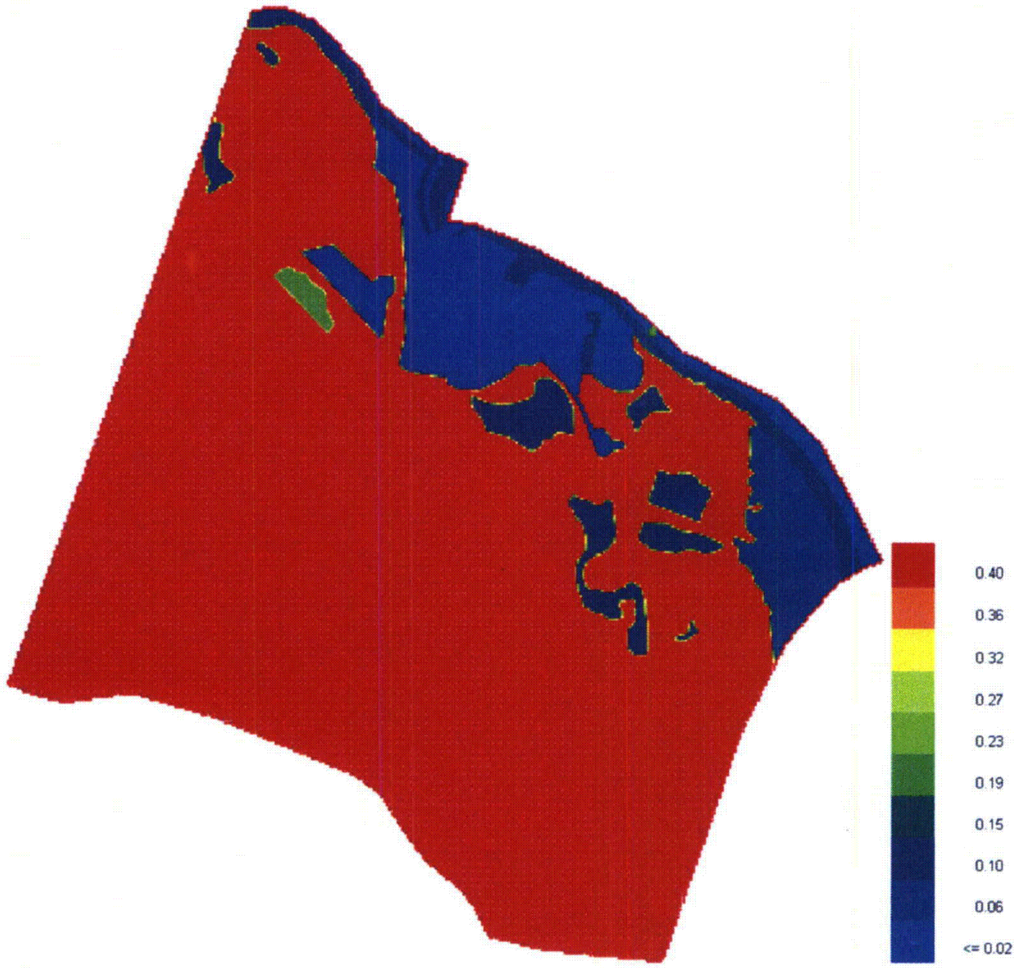


**Figure 3-4: FLO-2D Curve Number Selections**

**Figure 3-5: LIP Hyetograph**

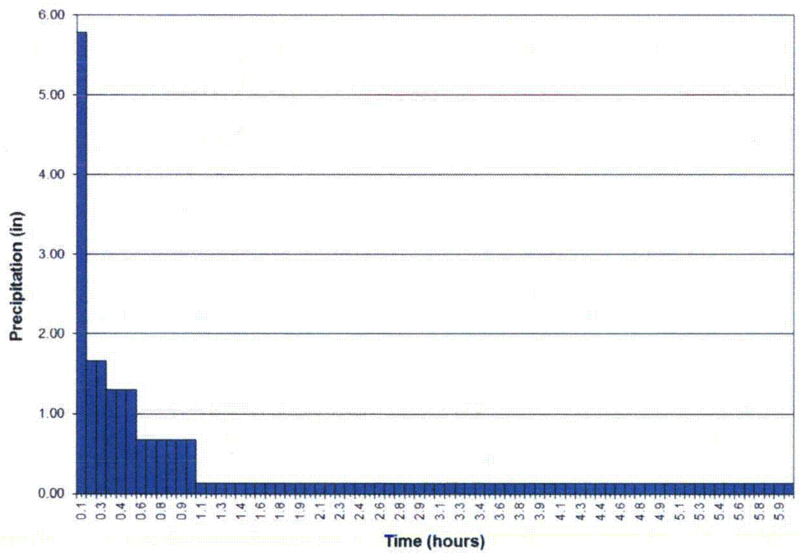
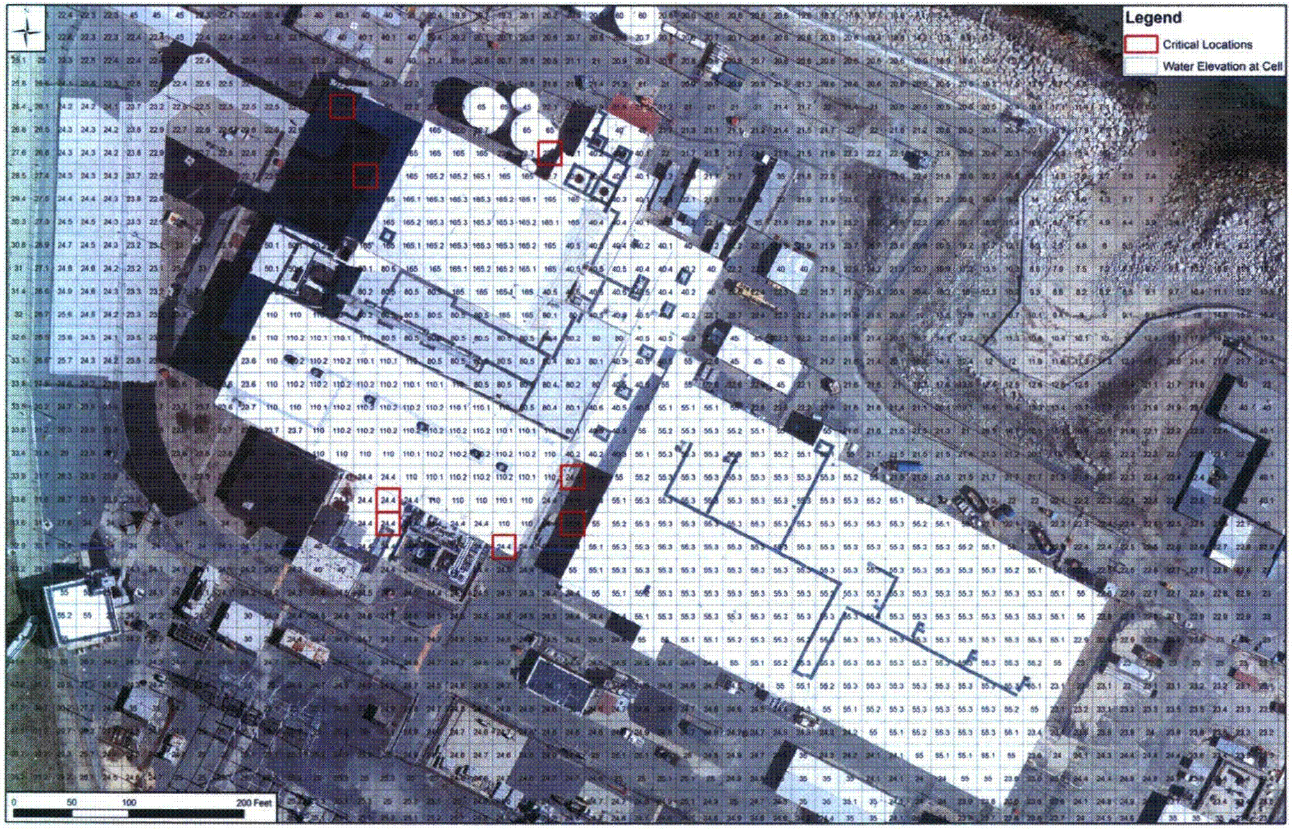


**Figure 3-6: PNPS Maximum Water Surface Elevations (feet, NAVD88)**




Basemap Source: High resolution orthoimagery (AREVA 2014c). Note that a larger version of this figure is available in Appendix A.

**Figure 3-7: Supercritical Flow Regime**

### 3.2 Flooding in Rivers and Streams

This section addresses potential flooding at PNPS due to the probable maximum flood (PMF) in rivers and streams. For further details of the assessment, refer to AREVA Document No. 51-9226922-000 (AREVA 2014).

#### 3.2.1 Methodology

The method used to address the PMF in rivers and streams followed the HHA approach as described in Section 3.3 of NUREG/CR-7046 (NRC 2011), which states that the following series of steps should be considered, as applicable:

1. Identify the plant's drainage basin and locate nearby surface water sources (i.e., rivers, streams, channels) with the potential to impact flooding at the PNPS site due to precipitation runoff.
2. Perform PMF maximum evaluations to assess the flooding hazard impacts for surface water sources identified in Step 1.

For the PNPS site, based on the findings for Step 1, it was not necessary to perform Step 2. Refer to the discussion below.

#### 3.2.2 Results

The PNPS site is located at the Atlantic Ocean shoreline with hills to the north, south and west, and Cape Cod Bay to the east. The site varies from 14 to 32 feet above MSL. Station grade is at 20 feet MSL. The 40 foot MSL ground surface contour crosses Rocky Hill Road, a public road, and closes within the site boundaries and is open only to the bay. The 24 foot MSL contour closes on the bay side of Rocky Hill Road. There are no rivers or streams on or adjacent to the site. As such, PNPS is located within an isolated drainage area on the northeast side of Pine Hills which is a north-south trending glacial ridge with a maximum elevation of 395 feet. All site surface drainage flows into the bay (PNPS 2013, Sections 2.4.1.2, 2.5.2.1 and 2.5.2.4.1).

The nearest, prominent inland bodies of water are Bartlett Pond which is approximately one and three-quarter miles southeast of PNPS at a topographic low point near the shore, and the Eel River which is approximately two and one-quarter miles west of PNPS and about three-quarters of a mile west of Pine Hills (Figure 3-8). The 1977 and 2012 topographic maps indicate that Bartlett Pond flows into Cape Cod Bay and that the Eel River flows into Plymouth Bay which is located in the northwestern portion of Cape Cod Bay. The topographic maps also depict a small body of water, essentially a wetland, at a topographic low point, about one and a quarter miles west of PNPS along the shoreline (USGS 1977 and USGS 2012). Refer to Figure 3-8. Several wetlands and a cranberry bog are also depicted on the 1977 topographic map in the site vicinity.

Similarly, although the United States Geologic Survey (USGS)'s National Hydrography Database (NHD) shows a few, small ponds near PNPS, there are no rivers or perennial or intermittent streams in the site vicinity (USGS 2014). Referring to Figure 3-9, the closest hydrologic feature is Beaver Dam Brook (USGS 1977) which flows into Bartlett Pond about one and three-quarter miles southeast of PNPS, at White Horse Beach. Figure 3-9 also indicates that the next closest hydrologic feature (the Eel River) is approximately two and one-quarter miles west of PNPS.

Since there are no rivers, streams or channels in the vicinity of the PNPS site, a PMF study in rivers and streams was not performed.

### 3.2.3 Conclusions

No surface water channels, streams or rivers are present within or adjacent to the PNPS drainage basin and no surface hydrologic features were identified by the USGS as a perennial or intermittent stream in the vicinity of PNPS. Due to local topography, the PNPS drainage basin is isolated from other area watershed basins.

### 3.2.4 References

**AREVA 2014**. AREVA Document No. 51-9226922-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Screening for Probable Maximum Flood, October 2014.

**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.

**PNPS 2013**. Pilgrim Nuclear Power Station Final Safety Analysis Report (FSAR), Revision 29, October 2013. (AREVA Doc. No. 38-9226908-000)

**USGS 1977**. Manomet Topographic Quadrangle Map 1977, Massachusetts-Plymouth County, 7.5 Minute Series, Scale 1:25 000, U.S. Geological Survey. (See AREVA Document No. 51-9226922-000)

**USGS 2012**. Manomet Topographic Quadrangle Map 2012, Massachusetts-Plymouth County, 7.5 Minute Series, Scale 1: 24 000, U.S. Geological Survey. (See AREVA Document No. 51-9226922-000)

**USGS 2014**. "The National Map – National Hydrography Database", United States Geologic Survey National Map Viewer, http://viewer.nationalmap.gov/viewer/nhd.html?p=nhd, date accessed 9/3/2014, date modified N/A.

(See AREVA Document No. 51-9226922-000)

**Figure 3-8: Plymouth County Massachusetts Topographic Map**

[Source: USGS 2012]

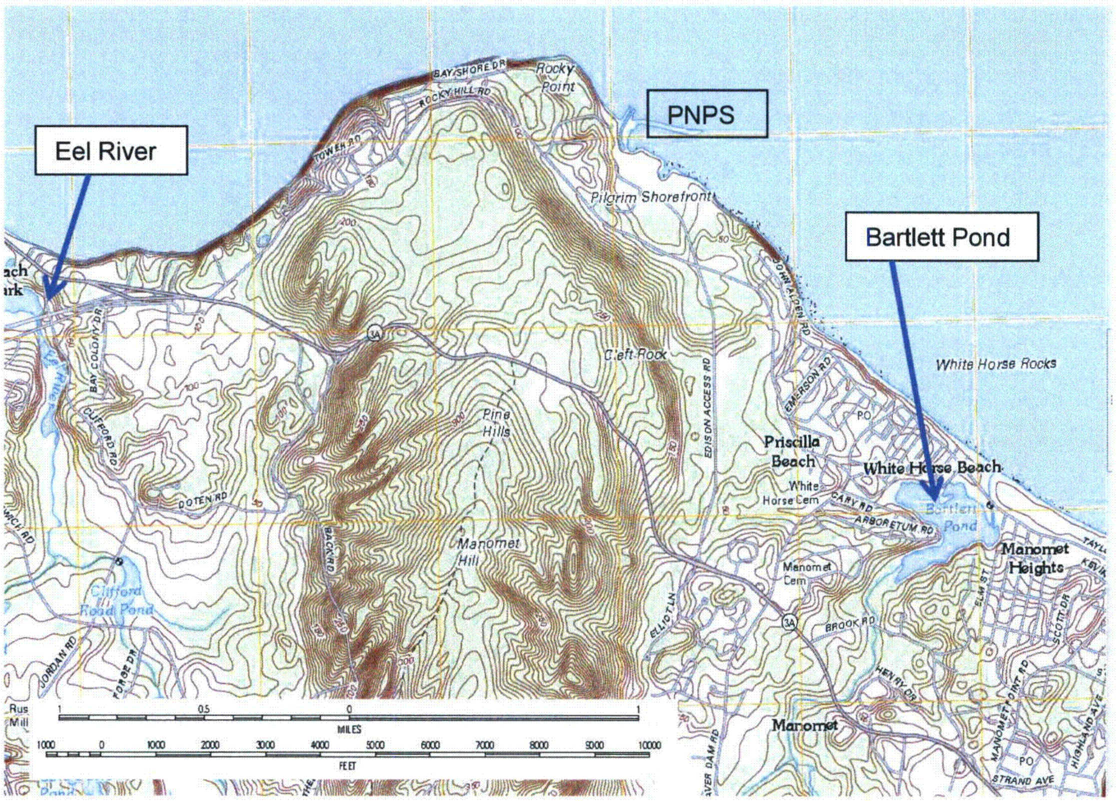
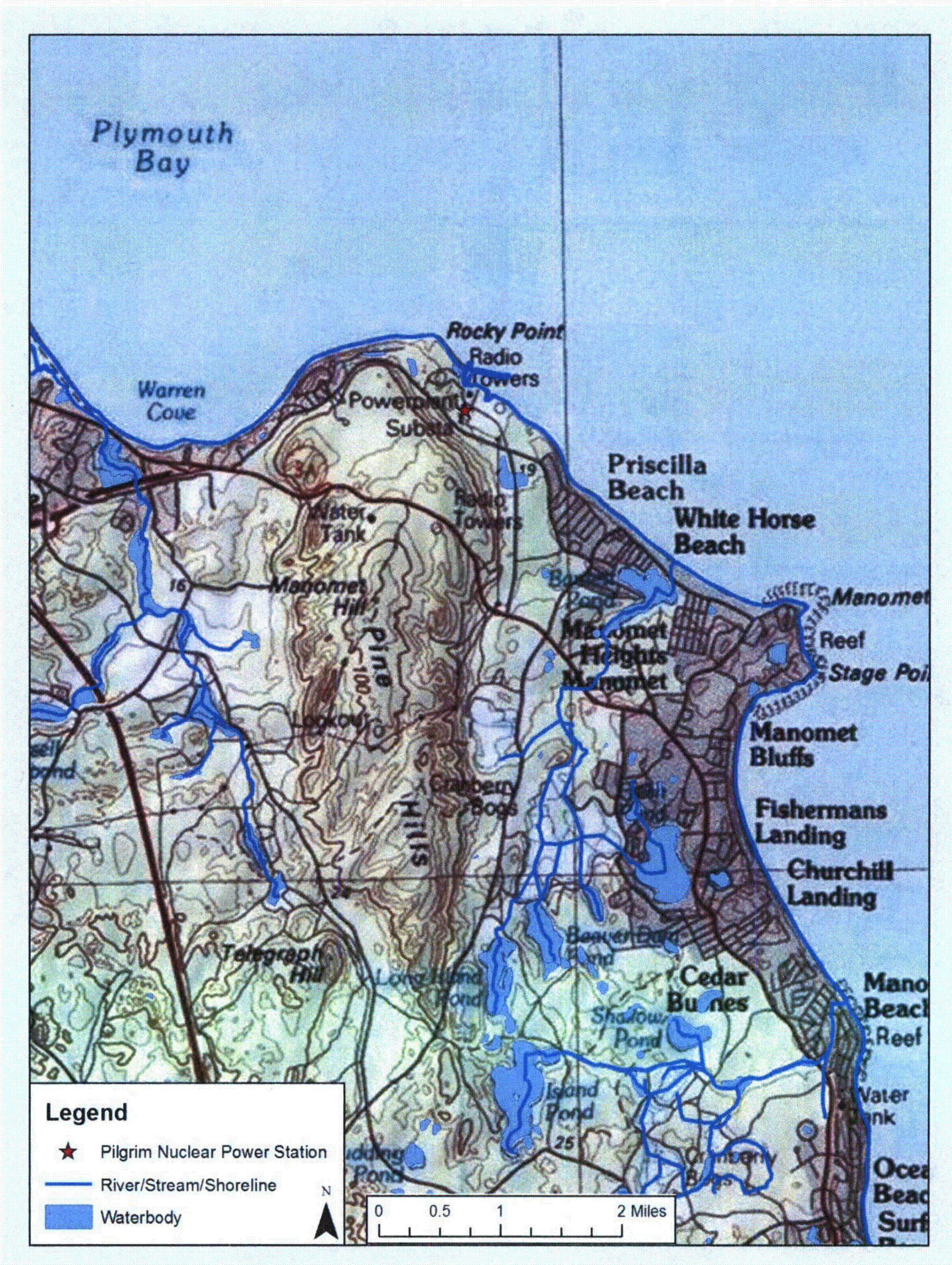


**Figure 3-9: Hydrologic Units Near PNPS**

[Source for hydrology: USGS 2014]



### 3.3 Dam Breaches and Failures

This section summarizes the assessment performed for flooding at PNPS due to potential dam breaches and failures. For further details of the assessment, refer to AREVA Document No. 51-9226924-000 (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 PNPS 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

The State of Massachusetts is hydrologically divided into 27 major watersheds. Each watershed has unique land and water features, history of water use, and development patterns influencing its water resources. PNPS is situated within the South Coastal Watershed Basin (see Figure 3-10). The South Coastal Watershed Basin discharges directly into the ocean and consists of 14 coastal river watersheds with a total drainage area of approximately 240.7 square miles. The basin spans over all of or part of 19 municipalities (MassGIS 2003 and MEEA 2014a). There are 61 dams within the South Coastal Watershed Basin with a total maximum storage capacity of 15,506.5 acre-feet (USACE 2013).

Referring to Figure 3-10, the Cape Cod Watershed Basin is situated south of the South Coastal Watershed Basin. It encompasses approximately 440 square miles and extends 70 miles into the ocean. It is surrounded by Buzzards Bay, Cape Cod Bay, the Atlantic Ocean and Nantucket Sound (MEEA 2014b). There are seven dams on tributaries to Cape Cod Bay within the Cape Cod Watershed Basin with a total maximum storage capacity of 656.9 acre-feet (USACE 2013).

Cape Cod Bay constitutes the southernmost part of the Gulf of Maine. The bay is bordered by land to the west, south and east and it is open to Massachusetts Bay and the Gulf of Maine to the north. The surface area of Cape Cod Bay is approximately 795 square kilometers (307 square miles) and it has a water volume of about $4.5 \times 10^{10}$ cubic meters ($1.6 \times 10^{12}$ cubic feet) (Davis 1992). If the total volume of all 61 South Coastal Basin dams and the seven dams on bay tributaries were to simultaneously fail and be instantly added to Cape Cod Bay, the water level increase within the bay would be less than one inch. Thus, the postulated failure of dams would have a negligible flooding effect at the PNPS site.

#### 3.3.3 Conclusions

Based on conservative assumptions, potential breaches of dams within the South Coastal Watershed Basin and on tributaries to Cape Cod Bay within the Cape Cod Watershed Basin, would not impact SSCs important to safety at PNPS considering the following:

- The failure of dams on watersheds that discharge to Cape Cod Bay or to Massachusetts Bay, just north of Cape Cod Bay, would have an insignificant impact on the water level within Cape Cod Bay.

- Once the flood waves from dam breaks reach Cape Cod Bay, water levels would be attenuated by the size and storage volume available in Cape Cod Bay as well as in Massachusetts Bay and the Gulf of Maine.

### 3.3.4 References

**AREVA 2014**. AREVA Document No. 51-9226924-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Screening for Dam Failures, October 2014.

**Davis 1992.** Western Cape Cod Bay: Hydrographic, Geological, Ecological, and Meteorological Backgrounds for Environmental Studies by J. D. Davis, American Geophysical Union – Transferred from Springer-Verlag in June 1992. (See AREVA Document No. 51-9226924-000)

**MassGIS 2003**. Massachusetts Office of Geographic Information, date modified March 2003; Available at: http://www.mass.gov/anf/research-and-tech/it-serv-and-support/application-serv/office-of-geographic-information-massgis/datalayers/majbas.html, date accessed August 15 and 19, 2014. (See AREVA Document No. 51-9226924-000)

**MEEA 2014a**. The Massachusetts Executive Office of Energy and Environmental Affairs, South Coastal Watersheds; Available at: http://www.mass.gov/eea/waste-mgnt-recycling/water-resources/preserving-water-resources/mass-watersheds/south-coastal-watersheds.html, date accessed August 19, 2014. (See AREVA Document No. 51-9226924-000)

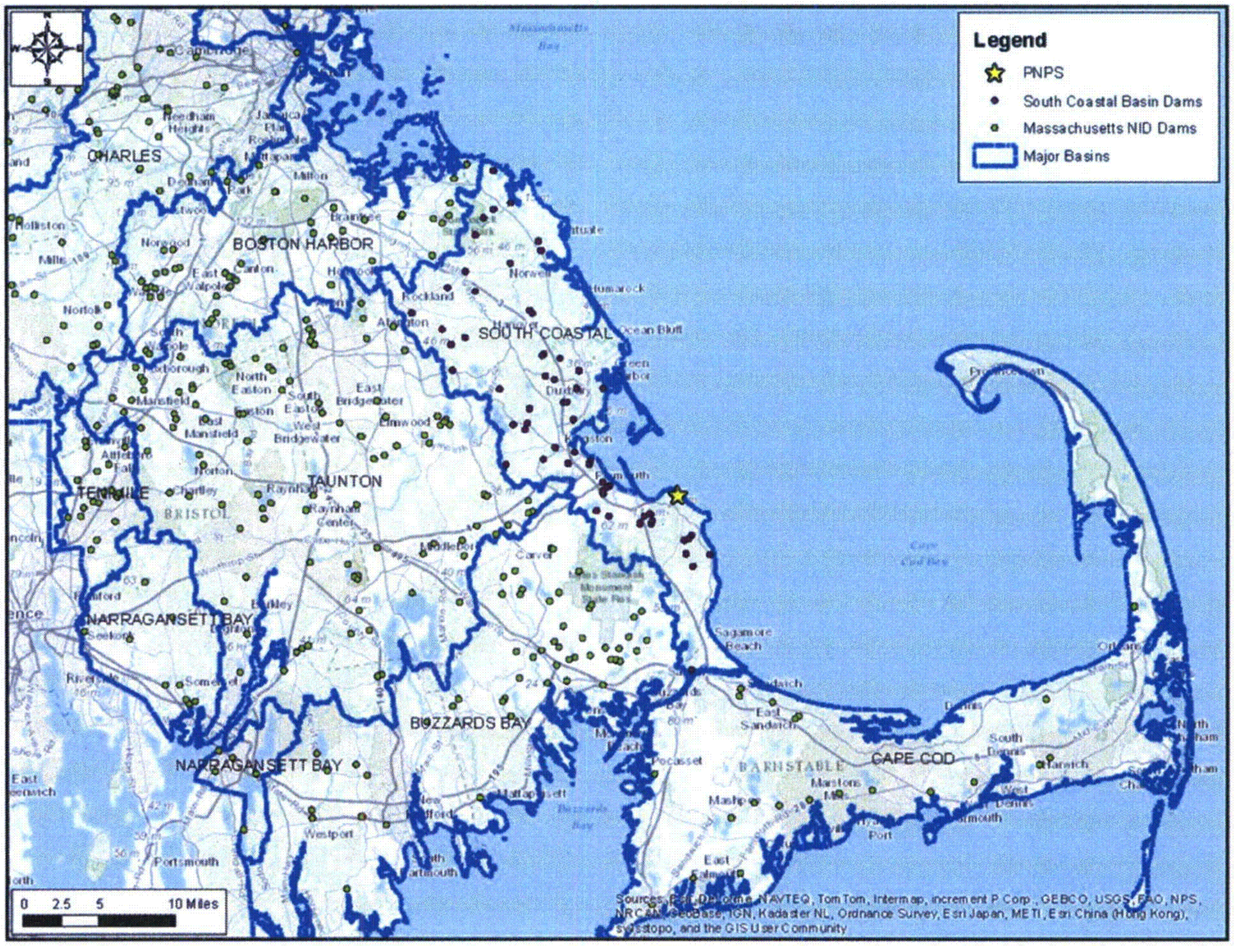
**MEEA 2014b**. The Massachusetts Executive Office of Energy and Environmental Affairs, Cape Cod Watershed; Available at: http://www.mass.gov/eea/waste-mgnt-recycling/water-resources/preserving-water-resources/mass-watersheds/cape-cod-watershed.html, date accessed August 19, 2014. (See AREVA Document No. 51-9226924-000)

**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 2013**. U.S. Army Corps of Engineers National Inventory of Dams, date modified February 2013; Available at: http://geo.usace.army.mil/pgis/f?p=397:1:10956392017256, date accessed August 15 and 19, 2014. (See AREVA Document No. 51-9226924-000)

**Figure 3-10: Dams within the South Coastal and Cape Cod Watershed Basins**

[Source: MassGIS 2003 and USACE 2013]



## 3.4 Storm Surge

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

An evaluation of the Probable Maximum Storm Surge (PMSS) flood hazard at PNPS was performed in a manner consistent with the HHA approach described in NUREG/CR-7046 (NRC 2011, Section 2). This evaluation was performed in two calculations. The AREVA Calculation "Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Probable Maximum Hurricane / Probable Maximum Wind Storm" (AREVA 2015a) assessed the Probable Maximum Hurricane (PMH) and the Probable Maximum Wind Storm (PMWS) at PNPS. Parameters defining the PMH and PMWS, which were ultimately determined based on a site-specific meteorology study, were used as input to the storm surge analysis presented in AREVA Document No. 32-9226919-001, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Probable Maximum Storm Surge (AREVA 2015b), which developed the maximum still water surface elevation representative of the PMSS at PNPS.

The methodology, results and conclusions associated with these calculations are presented below.

### 3.4.1 Methodology

The following sections summarize the methodology used to evaluate the PMH, PMWS and the PMSS elevation at PNPS.

#### 3.4.1.1 Probable Maximum Hurricane and Probable Maximum Wind Storm

A step-wise approach consistent with the HHA approach described in NUREG/CR-7046 (NRC 2011, Section 2) was used to deterministically evaluate the PMH and PMWS at PNPS, as both hurricanes and extra-tropical storms are of concern in the vicinity of PNPS as surge-generating meteorological events. The PMH and PMWS were evaluated independently using the methods summarized below.

The evaluation of the PMH included analyses of National Hurricane Center (NHC) historical, "Best-Track" hurricane data (i.e., HURDAT2) and, to supplement the extremely limited historical data, synthetic hurricane data representative of a large set of synthetic tropical cyclone tracks. The synthetic data were developed for the PNPS region by Dr. Kerry Emanuel of WindRiskTech, LLC (i.e., referred to herein as WRT) using coupled intensity and atmospheric models (refer to AREVA 2015a). The methodology used to develop the synthetic tropical cyclone tracks and storm parameters includes: 1) storm generation; 2) storm track generation; and 3) deterministic modeling of hurricane intensity. Using this methodology, a large number (i.e., 10,013) of synthetic storm tracks (i.e., referred to herein as the WRT storm set) was generated and filtered within a radius of 200 kilometers (km) of Plymouth, Massachusetts (MA) to support the evaluation of the PMH at PNPS.

The WRT storm set was compared to storm characteristics described by the HURDAT2 data set and, where present, variance from the historical hurricane data was identified. Comparisons were performed for parameters reflecting storm intensity, direction, size and speed. Based on these comparisons, the significantly larger WRT data set was ultimately demonstrated to be a conservative reflection of storm characteristics within the PNPS vicinity.

The following steps were then used to evaluate the historical and synthetic data and characterize the PMH at PNPS:

1. Determination of National Weather Service (NWS)-23 PMH parameters: Consistent with guidance presented in NUREG/CR-7046 (NRC 2011), ranges of potential PMH meteorological parameters were initially determined using NWS 23 (NOAA 1979). These parameters included the following: 1) peripheral pressure, 2) central pressure, 3) permissible range for radius of maximum winds, 4) permissible range of forward speeds, 5) permissible range of track direction, and 6) estimated maximum 10-meter, 10-minute over-water wind speed.
2. Site-Specific Meteorology Study: The site-specific meteorology study included a statistical analysis of the HURDAT2 database and the synthetically-developed hurricane parameter data set. The analysis focused on data reflecting storm intensity, direction and physical dimensions in the region of PNPS. Parameter selection was based on data availability within the HURDAT2 database and relevance with respect to comparison to parameter estimates derived from NWS 23. Probability Density Functions (PDFs) were constructed for these parameters from Probability Density Histograms (PDHs) using a non-parametric kernel method. To further refine the analysis of the low probability portion of the 1-minute, 10-meter altitude (1-min, 10-m) average wind speed (mxw) distribution, Extreme Value Analysis (EVA) was used based on the Peak Over Threshold (POT) method and the Generalized Pareto Distribution (GPD). A detailed statistical analysis of the WRT storm data was then performed using techniques determined to be appropriate by expert meteorologist judgment. These techniques included a univariate storm parameter probability analysis, an analysis of storm parameter covariance, and development of a large synthetic storm set extension (i.e., the 3,000,000 or 3M data set). Based on this analysis, a dimensionless scaling function was developed to conservatively reflect the deterministic upper-limit of storm intensity (i.e., maximum wind speed) in the PNPS vicinity in consideration of co-variability with storm direction (i.e., storm bearing reflecting direction of storm travel measured positive in a clockwise direction from north).

The site-specific meteorology study also included an evaluation of the PMWS. As part of this study, a review of available surface weather maps reflecting conditions during historically-significant extra-tropical storms in the PNPS vicinity was performed. Following this review, an up-scaled wind field representative of the PMWS at PNPS was developed in accordance with applicable guidance (e.g., ANS 1992).

The above-described methodology provided input to the storm surge analyses described below.

#### 3.4.1.2 Probable Maximum Storm Surge

The HHA approach described in NUREG/CR-7046 (NRC 2011, Section 2) was also applied in calculating the PMSS at PNPS. Again, the PMH and PMWS were evaluated independently for surge generating potential using the methods summarized below.

In evaluating the PMSS caused by the PMH, a screening-level assessment was first performed using the two-dimensional Sea, Lakes and Overland Surges from Hurricanes (SLOSH) computer model (NOAA 2012a and NOAA 2012b). SLOSH is computationally efficient, allowing many simulations to be performed over a relatively short period of time; however, the SLOSH model has limitations, including its relatively coarse, structured model grid. Therefore, SLOSH was applied in a relative manner to evaluate parameter sensitivity, and in a second phase of modeling, additional simulations were performed using the ADvanced CIRCulation (ADCIRC) model (Luettich et al., 1992). Simulations were also performed using ADCIRC+SWAN (i.e., also known as ADCSWAN), which is a form of ADCIRC that is tightly-coupled with the Simulating Waves Nearshore (SWAN) model (Booij et al., 1999). While ADCIRC and

ADCIRC+SWAN are not hindered by many of the limitations associated with SLOSH, the high-resolution, finite-element mesh and related high computational demand prevent broad applications (i.e., only a limited number of storm simulations is practicable in the context of a given analysis). Therefore, ADCIRC was applied in a targeted fashion to further evaluate and refine surge predictions associated with the storms identified based on the SLOSH sensitivity analysis.

More specifically, the PMSS methodology associated with the PMH included the following steps:

1. Calculation of the Antecedent Water Level: An Antecedent Water Level (AWL) was calculated using data obtained from the Boston, MA National Oceanic and Atmospheric Administration (NOAA) tidal gaging station per applicable regulatory guidelines (ANS 1992 and NRC 2011). In accordance with these guidelines, observed monthly maximum tide data obtained over a continuous 21-year period (i.e., January 1, 1993 through December 31, 2013) were used to calculate the 10 percent exceedance high tide. Cumulative Sea Level Rise (SLR) based on observed rates at the Boston station projected over a 50 year period was then added to obtain the AWL.
2. PMH Parameter Sensitivity Assessment (SLOSH): Simulations were performed using the SLOSH model, the Initial Storm Set, and the AWL to identify: 1) the sensitivity of storm surge at PNPS to different storm parameters (i.e., storm track, radius of maximum winds, etc.) as constrained by the PMH calculation; and 2) the specific combinations of storm parameters and storm tracks that result in the largest predicted storm surges at PNPS, also constrained by the PMH calculation. The simulations assumed steady-state conditions (i.e., storm parameters were not varied from the initial specifications). A set of storms was selected for ADCIRC and ADCIRC+SWAN simulations after processing the results of the SLOSH sensitivity analysis.
3. Refinement-Level Assessment of the PMH (ADCIRC and ADCIRC+SWAN): Refined storm surge simulations were performed using ADCIRC to evaluate the maximum storm surge associated with the PMH. Simulations were performed assuming steady conditions similar to the screening-level assessment for the purpose of comparing ADCIRC to SLOSH. ADCIRC simulations focused on predicting the maximum still water elevation associated with the PMSS; whereas, ADCIRC+SWAN simulations qualitatively evaluated wave setup and generated input required for a combined effects analysis.
4. Assessment of the PMWS (ADCIRC and ADCIRC+SWAN): To evaluate the maximum storm surge caused by the PMWS, ADCIRC and ADCIRC+SWAN were directly applied based on the results of the PMWS evaluation in a manner consistent with guidance presented in ANSI/ANS-2.8-1992, American National Standard for Determining Design Basis Flooding at Nuclear Reactor Sites (ANS 1992) and the HHA approach. This evaluation included utilization of the previously-calculated AWL and the up-scaled PMWS wind and pressure field. Sensitivity of surge to storm path (i.e., track direction) and storm forward speed was evaluated.

Note that the PMSS was determined as the greater of the maximum water surface elevations resulting from either the PMH or the PMWS.

### 3.4.2 Results

The following sections describe the results of the PMH, PMWS and PMSS evaluations at PNPS.

### 3.4.2.1 Probable Maximum Hurricane / Probable Maximum Wind Storm

#### 3.4.2.1.1 Determination of NWS 23 PMH Parameters

The location of PNPS is shown in Figure 3-11. Figure 3-12 also shows the location of PNPS in relation to coastal distance intervals (i.e., mile posts) presented in NWS 23 (NOAA 1979). As indicated on Figure 3-12, PNPS is located in the vicinity of NWS mile post 2800, where coastal distance is measured in nautical miles from the Gulf of Mexico. Based on the location of PNPS, the PMH parameters shown in Table 3-4 were extracted from NWS 23.

The methods of parameter development presented in NWS 23 are generally not consistent with the current state of knowledge for characterizing the PMH affecting the PNPS vicinity. In specific reference to PMH intensity reflected by maximum wind speed, NWS 23 values are recognized as lacking a reflection of the relationship between storm direction and storm magnitude (i.e., co-variability), which is likely to result in overly conservative intensity recommendations for west-of-north tracking (i.e., westerly) storms. Thus, a detailed site-specific meteorology study was performed to develop the hurricane meteorological parameters for analysis of flooding due to combined storm surge and wind-generated waves. The results of this study, which also included an analysis of the PMWS at PNPS, are summarized below.

#### 3.4.2.1.2 Site-Specific Meteorology Study – PMH

Recorded track positions of tropical storms and hurricanes are maintained by the NHC in the annually updated HURDAT database. The official HURDAT database, referred to as HURDAT2, contains information on actual cyclones dating from 1851 through 2013 (NOAA 2014a). This American Standard Code for Information Interchange (ASCII)-formatted database contains six-hourly (00, 06, 12, 18 Coordinated Universal Time, UTC) cyclone center locations (i.e., with differences in locations being representative of storm direction and translational speed) and intensities, with intensity being measured by the maximum, 1-min sustained wind speed. Beginning with the 2012 hurricane season, the HURDAT2 database contains additional storm information including some position and intensity data at nonstandard times and estimates of the radial distances of several wind speed thresholds. The wind data at radial distances are insufficient for estimating the radius of maximum winds, a parameter useful for storm surge modeling but unavailable from the HURDAT2 dataset. The HURDAT2 maximum sustained wind data also have limited precision. Prior to 1885, storm intensity was recorded to the nearest ten knots and to the nearest five knots thereafter. This becomes important in estimating the long return periods of strong hurricanes using extreme value statistics.

Statistical analyses were performed using data extracted from the HURDAT2 database over several sampling domains depicted in Figure 3-13. The statistical analyses of these historical storm data were limited with respect to characterizing the PMH due to the small sample size of hurricanes affecting the PNPS vicinity, particularly with respect to the representation of storms with high intensities. For this reason, synthetic hurricane data, which greatly increase the sample size of intense hurricanes in the vicinity of PNPS, were used to perform a more detailed statistical analysis of storm characteristics within the study area. This analysis, which included qualitative comparisons to the limited historical data, is described below.

The synthetic storm set, referred to herein as the WRT storm set, contains over 10,000 tropical storms and hurricanes characterized by track positions and various storm parameters. These storms are generated by ocean-coupled atmospheric and hurricane intensity models. The storm parameters are available at two-hour intervals, and are pre-screened to impact the Cape Cod Bay (i.e., the source of surge impacts to PNPS).

First, an EVA was performed using maximum wind speed data from the WRT storm set, the results were compared to a similar analysis of the HURDAT2 data. The data, along with functional fits to the data, are shown in Figure 3-14. Based on visual inspection of this figure, the function derived from least-squares fitting appears to be non-conservative for maximum wind speeds exceeding approximately 105 knots (i.e., due to probability-space weighting); whereas, the fit derived from manual parameter selection demonstrates potential bias introduced through fitting to unrealistic data. In recognition of the unsatisfactory results produced by these fitting methods, a kernel method was used to generate a univariate distribution (i.e., CDF) representative of maximum wind speed data in the PNPS subregion (refer to Figure 3-15). The results, shown in CDF form in Figure 3-15 and in terms of log return period in Figure 3-16, are less influenced by the unrealistic maximum wind speed data identified above (i.e., probability determinations derived via the kernel method are not significantly biased by the previously-discussed unrealistic storms). However, as the unrealistic data are not excluded from the analysis, an influence of these data on the kernel method predictions is evident (i.e., rather than plateauing near 120 knots, as suggested by the behavior of the functional fit based on least squares fitting, the kernel method predictions continue to increase as return periods approach 100,000 years).

Figure 3-15 and Figure 3-16 can also be used to evaluate probability determinations derived via the kernel method for the HURDAT2 data within the PNPS subregion (i.e., shown as dashed lines in each figure). The results indicate that the WRT maximum wind speeds produce conservative exceedance probability predictions at higher intensity levels (i.e., above approximately 95 knots) relative to analyses performed using HURDAT2 data, which are very limited with respect to sample size within the PNPS subregion. For these reasons, the kernel method is carried forward to the extended analyses described below.

The univariate distribution of storm bearing within the PNPS subregion, generated by the kernel method, is shown in Figure 3-17 based on WRT data. Tabulated cumulative frequencies and annualized cumulative frequencies are included over the same intervals as reported for the HURDAT2 analysis. For comparison, the CDF curve from the HURDAT2 data is included as a dashed line. The distribution functions for the WRT and HURDAT2 data coincide overall, but the comparison is particularly favorable for west-of-north bearings. The annual cumulative frequencies associated with the WRT data are higher than comparable HURDAT2 values for bearings west of -40° and lower for bearings in the range -30° to +10°; differences which are attributable to the small HURDAT2 sample size and the smoothing effect of the kernel method at the tail of the distribution. In general, the WRT analysis produces more conservative return period estimates for most west-of-north storms.

The distribution of storm forward speed from the WRT dataset is shown in Figure 3-18. HURDAT2 data are shown as a dashed line for comparison. Although the two distributions generally reflect agreement, the WRT dataset produces more frequent storms moving slower than 25 knots and less frequent storms moving at greater speeds compared to the HURDAT2 dataset. As with the storm bearing distribution, differences are likely attributable to the small HURDAT2 sample size and smoothing effect introduced by the kernel method at distribution tails.

The results indicate that use of the WRT representation of the empirical storm data for estimating independent and joint variability of hurricane parameters will contain a conservative bias. While storm intensity is well-represented by the WRT data for major storms, the WRT's bias toward more frequent westerly storms and lower forward speeds is expected, given the sensitivity to storm surge within Cape

Cod Bay relative to westerly storm tracks, to conservatively predict more frequent and larger storm surges near PNPS.

Based on the analyses described above, parameters and parameter ranges representative of the PMH at PNPS were developed in recognition of parameter co-variability (refer to Table 3-5). The considered bearing range includes storms with bearings between -80° and 40° to provide a bounding parameter set (i.e., relative to the anticipated PMH) inclusive of more intense, northerly-bound storms. Maximum storm intensities (i.e., maximum wind speeds) were determined by identifying a dimensionless scaling function that recovered variability of maximum wind speed as a function of storm bearing (Figure 3-19). While the process of identifying this scaling function involved probability calculations for parameter combinations (i.e., storm bearing and maximum wind speed), the resulting parameter combinations represent deterministic PMH limits, as the NWS 23 PMH maximum wind speed is used to establish the probability threshold.

Ranges of the radius to maximum winds and forward speed parameters were also developed in consideration of parameter covariability and a comparison to the NWS 23 recommended PMH parameter ranges. In the case of the radius to maximum winds parameter, an analysis of the synthetic data supported the NWS 23 recommended range; therefore, no change was warranted. For the forward speed parameter, an evaluation of the synthetic data suggested an expanded range (i.e., decrease in the lower bound from 40 to 20 knots) relative to the NWS 23 recommendation was supported and demonstrably conservative.

#### 3.4.2.1.3 Site-Specific Meteorology Study – PMWS

In accordance with the procedures outlined in ANSI/ANS-2.8-1992 (ANS 1992), the February 7, 1978 storm was selected as a representative extra-tropical storm event for use in developing the properties of the PMWS. Surface weather maps for the storm were obtained from the National Climatic Data Center (NCDC) (NOAA 2014b) and used as input into the PMWS model. These maps were used to calculate isobars, distances between isobars, and wind angles affecting the PNPS region and to establish a relationship between the pressure maps and a geographic coordinate system in order to determine storm speed.

The February 7, 1978 extra-tropical storm approached Cape Cod from the south and travelled along the US East Coast in an approximately southwest-to-northeast direction. This path is similar to the tracks associated with many of the historically-significant storms that have affected this region, as shown in Figure 3-20. The translation speed calculated for the February 7, 1978 extra-tropical storm in the PNPS vicinity ranged from 10 to 15 miles per hour (mph) with an average speed of 13 mph. As shown in Table 3-6, the average translation speed ranged from 11 to 37 mph for eight of the top ten surge events at Boston, MA.

To evaluate other potential PMWS paths, the tracks associated with storms responsible for the top 20 storm surges at Boston, MA were digitized from the United States Daily Weather Maps from the NOAA Central Library (NOAA 2014c) and, where available, three-hour surface weather maps from NOAA (NOAA 2014d). The digitized storm tracks are shown in Figure 3-20.

A spatially varying pressure and wind field was developed by dividing the PMWS into twelve radials (refer to Figure 3-21). Additional radials in the western half of the storm were added to more accurately spatially resolve the winds that cause the surge at PNPS. 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 (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 to the maximum wind speed of the entire storm (ANS 1992). In order for the minimum pressure to reach 950 millibars (mb), the minimum pressure of the storm, 991.4 mb, was scaled down to 950 mb using a reduction of 41.4 mb for each pressure value. The resulting PMWS wind and pressure fields are shown in Figure 3-22.

### 3.4.2.2 Probable Maximum Storm Surge

#### 3.4.2.2.1 Development of the Antecedent Water Level

In accordance with NUREG/CR-7046 (NRC 2011), the PMSS is required to be evaluated coincidentally with an AWL equal to the 10 percent exceedance high tide plus long term changes in sea level. The 10 percent exceedance high tide is defined as the high tide level that is equaled or exceeded by 10 percent of the maximum monthly tides over a continuous 21 year period. In accordance with ANSI/ANS-2.8-1992 (ANS 1992), this tide can be determined from recorded tide data or from predicted astronomical tide tables.

In this calculation, the 10 percent exceedance high tide was calculated using recorded monthly maximum tide elevations from the Boston, MA tidal gaging station. Using this approach, a value of 7.34 feet NAVD88 (8.17 feet MSL) was obtained. In consideration of SLR, which was projected over 50 years using the annual rate at the Boston, MA station, the AWL was determined to be 7.80 feet NAVD88 (8.63 feet MSL).

#### 3.4.2.2.2 PMH Parameter Sensitivity Assessment (SLOSH)

In performing the PMH parameter sensitivity analysis using the NOAA SLOSH model, results from 5,005 simulations (i.e., tracks shown in the form of simulated surge elevation time series for each simulation) were extracted at locations including the model cell representing the PNPS shoreline (Figure 3-23). The time series were reduced to peak surge elevations at this location for each simulated storm.

Figure 3-24, Figure 3-25 and Figure 3-26 summarize the results of the PMH sensitivity assessment at PNPS. As indicated by Figure 3-24, SLOSH-simulated maximum still water elevations were found to be positively correlated with radius to maximum winds. The upper limit of the radius to maximum winds range (i.e., based on input derived from the PMH/PMWS calculation, AREVA 2015a) was found to consistently result in the maximum still water elevation for all potential values of potential PMH bearing. As indicated by Figure 3-25, SLOSH-simulated maximum still water elevations were also found to have directionally-dependent (i.e., functionally related to storm bearing) sensitivities to landfall location. In general, maximum still water elevations produced by PMH configurations moving west-of-north were less sensitive to landfall location; whereas, PMH configurations moving east-of-north produced maximum still water elevations that were highly sensitive to landfall location. However, in all cases, sensitivity profiles (i.e., evaluated on a bearing-specific basis) indicated maximum still water elevations were well represented by PMH configurations passing through landfall location 5 (refer to Figure 3-27).

Finally, as indicated by Figure 3-26, SLOSH-simulated maximum still water elevations were found to have directionally-dependent (i.e., functionally related to storm bearing) sensitivities to forward speed. In general, maximum still water elevations produced by PMH configurations moving west-of-north occurred for forward speeds between 30 and 35 knots; PMH configurations moving south-to-north and east-of-north produced maximum still water elevations for forward speeds between 20 and 25 knots.

The above-described process identified 13 PMH configurations to be simulated using ADCIRC and/or ADCIRC+SWAN to produce maximum water surface elevation (WSEL) values at PNPS (i.e., representative of storm-induced surge and wind-driven wave setup). The parameter combinations associated with these PMH configurations, which attempt to maximize surge response for each potential storm bearing within the considered range, are summarized in Table 3-7.

#### 3.4.2.2.3 Refinement Level Assessment of the PMH (ADCIRC and ADCIRC+SWAN)

ADCIRC+SWAN simulations were performed for PMH parameter combinations judged to be representative of conditions responsible for maximizing surge responses at PNPS for each considered storm bearing value based on the results of the sensitivity analysis described above. Results were evaluated at a location within the model mesh representative of the PNPS shoreline (Figure 3-28). Simulations were again performed assuming steady-state storm forcing conditions occurring coincidentally with the AWL. Based on these simulations, the following combination of storm parameters was identified as being responsible for the maximum WSEL caused by a PMH configuration at PNPS:

STORM ID = 3397 (refer to Figure 3-27)

- Track Direction (Θ) = 10 degrees (°);
- Landfall Location = 5 (Latitude 41.739°, Longitude -69.934°);
- Radius of Maximum Winds (Rmax) = 35 nautical miles (nm);
- Forward Speed (Vf) = 20 knots
- Maximum Wind Speed (Vm) = 128 knots; and
- Central Pressure Deficit (CPD) = 112 mb.

After identifying the controlling PMH configuration (i.e., STORM ID responsible for the highest WSEL at PNPS), an additional ADCIRC-only simulation was performed to assess the amount of wave setup contributing to the maximum WSEL at PNPS. The results of this simulation, which are tabulated in Table 3-8, indicate wave setup is a relatively small contributor (i.e., only 0.1 feet) to the total maximum WSEL resulting from STORM ID 3397. This value is likely to be conservative, as breaking wave action and near-shore wave reformation as a result of the influence of the breakwaters is likely to reduce near-shore wave setup. Additional evaluations of flooding potential and wave runup effects relative to plant grade and flood protection elevations for equipment important to safety were performed using high-resolution, near-shore wave modeling as part of the combined flooding effects evaluation (see Section 3.9).

#### 3.4.2.2.4 Assessment of the PMWS (ADCIRC and ADCIRC+SWAN)

As a first step in evaluating the maximum surge caused by the PMWS, a set of base storm tracks was created using three values of potential storm bearing (Figure 3-29). The base track value of 76.5° reflects the generalized track direction associated with the "Blizzard of '78", or the extra-tropical storm that forms the basis for the PMWS wind and pressure fields in the vicinity of PNPS. The two additional values (i.e., 61.5° and 91.5°) represent 15° rotations of the base track. These rotations are used to evaluate alignment of maximum PMWS winds with the primary axis of the water body, in accordance with applicable guidance (ANS 1992).

Each track was then assigned three values of forward speed (i.e., 9, 12 and 15 knots) based on the range of potential PMWS forward speeds identified during the PMWS evaluation. The three tested values represent bounding conditions based on an assessment of the range of forward speeds associated with historically-significant extra-tropical events at the Boston, MA NOAA Center for Operational Oceanographic Products and Services (CO-OPS) station (AREVA 2015a).

Finally, the tracks were paired with the PMWS wind and pressure field developed from the PMWS evaluation to create the PMWS Storm Set. The PMWS Storm Set consists of the following nine parameter combinations:

3 (bearing) x 3 (forward speed) = 9 PMWS Parameter Combinations

Each resulting PMWS configuration was assigned a unique storm identification (STORM ID) number ranging from ET_1 to ET_9. Table 3-9 shows the parameters associated with each PMWS simulation.

ADCIRC-only simulations were first performed for each of the PMWS Storm Set parameter combinations, with the results being representative of maximum still water elevations without contributions from wind-driven wave setup. After assessing the results of the ADCIRC simulations, ADCIRC+SWAN simulations were performed for a subset of the PMWS Storm Set (i.e., bearing = 61.5°, or the bearing associated with the highest maximum still water elevations) to calculate maximum WSELs in consideration of wind-driven wave setup.

ADCIRC-simulated maximum still water elevations and ADCIRC+SWAN-simulated maximum WSELs at the PNPS shoreline location are provided in Table 3-10 for STORM IDs ET_1, ET_2 and ET_3. As indicated by this table, the controlling parameter combination (i.e., STORM ID ET_2) produced the highest maximum WSEL at PNPS of the three PMWS configurations simulated with ADCIRC+SWAN (i.e., bearing = 61.5°). The parameters defining PMWS STORMID ET_2 are provided below:

PMWS STORMID = ET_2

- Track Direction (θ) = 61.5°
- Forward Speed (Vf) = 12 knots
- Maximum wind speed and central pressure defined by PMWS wind and pressure field

Table 3-10 also indicates that wave setup is, again, a relatively small contributor (i.e., only 0.5 feet) to the total maximum WSEL resulting from STORM ID ET_2. As previously noted, this value is likely to be conservative, as breaking wave action and near-shore wave reformation as a result of the influence of the breakwaters is likely to reduce near-shore wave setup. Refer to Section 3.9 for additional evaluations of flooding potential and wave effects relative to plant grade and flood protection elevations for equipment important to safety in which high-resolution, near-shore wave modeling was used to specifically evaluate the effects of these features under combined flooding effect scenarios.

### 3.4.3 Conclusions

Based on the analyses described above, the following conclusions are reached:

- Per ANSI/ANS-2.8-1992 (ANS 1992) and JLD-ISG-2012-06 (NRC 2013), the antecedent water level inclusive of 50 years of projected SLR is 7.8 feet NAVD88.
- The maximum still water elevation generated by a PMH configuration is 14.9 feet NAVD88 (15.7 feet MSL). This elevation is caused by a PMH configuration landfalling along the eastern shore of Cape Cod and traveling slightly east-of-north (i.e., bearing of 10°).
- The maximum still water elevation generated by a PMWS configuration is 14.0 feet NAVD88 (14.8 feet MSL). This elevation is caused by a PMWS configuration passing to the south of Cape Cod and traveling in an east-northeast direction (i.e., bearing of 61.5°).
- In both cases (i.e., the PMH and PMWS), wave setup contributions to total WSELs are determined to be relatively small and generally consistent between the two storm types (i.e., 0.1 feet for the PMH versus 0.5 feet for the PMWS).
- The PMSS at PNPS (i.e., assessed in terms of maximum still water elevation and wave setup) is thus determined to be 15.0 feet NAVD88 (15.8 feet MSL) resulting from a PMH.

Uncertainty and conservatism were considered in the calculation as per Section 5.4 of NUREG/CR-7046 (NRC 2011), as follows. The PMH and 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:

- Conservatism was promoted through the use of synthetic data, which show bias toward more frequent, higher intensity tropical cyclones;
- Conservatism introduced through the assumptions used in developing the antecedent water level, including the use of a 50-year projection window for defining applicable SLR;
- Conservatism introduced by the generally positive bias demonstrated by the ADCIRC/ADCIRC+SWAN model mesh verification with respect to simulated high water levels relative to observed data in the vicinity of PNPS (i.e., Boston, MA);
- 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); and
- Storm tracks were simulated as straight lines, and forward speeds were set and constant (i.e., steady) rates to increase the effect of the pressure gradients and resulting wind speeds.

Thus, the results of this calculation represent a conservative deterministic assessment of the PMSS at PNPS.

### 3.4.4 References

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**AREVA 2015b**. AREVA Document No. 32-9226919-001, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Probable Maximum Storm Surge, 2014.

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**NOAA 1979**. Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Wind Fields, Gulf and East Coast of the United States, National Oceanic and Atmospheric Administration Technical Report NWS 23, September 1979. (See AREVA Document No. 32-9226726-000)

**NOAA 2012a**. SLOSH Model v3.97 National Oceanic and Atmospheric Administration, Evaluation Branch, Meteorological Development Lab, National Weather Service, January 2012. (See AREVA Document No. 32-9226919-001)

**NOAA 2012b**. SLOSH Display Program (1.65b), National Oceanic and Atmospheric Administration, Evaluation Branch, Meteorological Development Lab, National Weather Service, January 2012. (See AREVA Document No. 32-9226919-001)

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**NOAA 2014c**. 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: October, 2014. (See AREVA Document No. 32-9226726-000)

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

**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. (See AREVA Document No. 32-9226726-000)

**Table 3-4: PMH Parameters and Parameter Ranges per NWS 23 for PNPS**

| Parameter | Unit | Lower Bound | Upper Bound |
|---|---|---|---|
| Peripheral Pressure, $P_w$ | in Hg (mb) | 30.12 (1020) | |
| Central Pressure, $P_o$ | in Hg (mb) | 27.23 (922) | |
| Pressure Deficit, $\Delta P$ | in Hg (mb) | 2.89 (98) | |
| Radius to Max. Winds, $R_{max}$ | (nm) | 17 | 34 |
| Forward Speed, T | (kt) | 40 | 50 |
| Track Direction, $\theta$ | (°) | 100 | 155 |
| Storm Bearing, Fdir | (°) | -80 | -25 |
| Maximum 1-minute wind speed, $V_{max}$ | (kt) | 132.8 | 138.8 |

Note: mb = millibars; kt = knots; nm = nautical miles; ° = degrees.

**Table 3-5: Recommended PMH Parameters for PNPS Vicinity Based on Site-Specific Meteorology Study**

| Storm Bearing θ | Maximum Wind Speed, Vm | Forward Speed, $V_f$ | Radius of Maximum Winds, $R_{max}$ |
|---|---|---|---|
| -80° | 88 kt | 20 to 50 kt (applies to all bearings) | 17 to 34 nm (applies to all bearings) |
| -70° | 91 kt | | |
| -60° | 95 kt | | |
| -50° | 101 kt | | |
| -40° | 108 kt | | |
| -30° | 114 kt | | |
| -20° | 119 kt | | |
| -10° | 123 kt | | |
| 0° | 125 kt | | |
| 10° | 128 kt | | |
| 20° | 129 kt | | |
| 30° | 130 kt | | |
| 40° | 128 kt | | |

Note: Maximum wind speed reflected as 1-minute average, 10-meter altitude value.

**Table 3-6: Forward (i.e., translational) Speeds for Eight of the Top Ten Extra-Tropical Storms in the PNPS Vicinity**

| Storm Event | Forward Speed (mph) | Forward Speed (kt) |
|---|---|---|
| Blizzard of '78 | 13.0 | 11.3 |
| January 1987 | 16.4 | 14.3 |
| Perfect Storm | 20.0 | 17.4 |
| January 1979 | 19.6 | 17.0 |
| December 1992 | 10.6 | 9.2 |
| December 1959 | 36.9 | 32.0 |
| December 1972 | 21.3 | 18.5 |
| December 2010 | 15.0 | 13.0 |

**Table 3-7: PMH Configurations Used in Refinement Simulations with ADCIRC and ADCIRC+SWAN**

| STORM ID | CPD (mb) | θ (deg from N) | Vf (kt) | Rmax (nm) | Landfall Location | Vm (kt) |
|---|---|---|---|---|---|---|
| 2737 | 112 | 40 | 20 | 35 | 5 | 128 |
| 2957 | 115 | 30 | 20 | 35 | 5 | 130 |
| 3177 | 113 | 20 | 20 | 35 | 5 | 129 |
| 3397 | 112 | 10 | 20 | 35 | 5 | 128 |
| 3618 | 100 | 0 | 25 | 35 | 5 | 124 |
| 3838 | 98 | -10 | 25 | 35 | 5 | 123 |
| 4058 | 92 | -20 | 25 | 35 | 5 | 119 |
| 4279 | 80 | -30 | 30 | 35 | 5 | 114 |
| 4499 | 72 | -40 | 30 | 35 | 5 | 108 |
| 4719 | 61 | -50 | 30 | 35 | 5 | 101 |
| 4939 | 53 | -60 | 30 | 35 | 5 | 95 |
| 5160 | 45 | -70 | 35 | 35 | 5 | 91 |
| 5380 | 43 | -80 | 35 | 35 | 5 | 88 |

Note: Maximum wind speed (Vm) reflected as 1-minute average, 10-meter altitude value

**Table 3-8: Simulated Maximum Still Water and Total Water Surface Elevations for STORM ID 3397**

| STORM ID | ADCIRC (feet NAVD88) | ADCIRC (feet MSL) | ADCIRC+SWAN (feet NAVD88) | ADCIRC+SWAN (feet MSL) | Wave Setup (feet) |
|---|---|---|---|---|---|
| 3397 | 14.9 | 15.7 | 15.0 | 15.8 | 0.1 |

Note: Tabulated values reflect rounding to one tenth of a foot precision.

**Table 3-9: Ranges of Evaluated PMWS Bearing and Forward Speed**

| STORM ID | Storm Bearing (degrees from north) | Forward Speed (kt) |
|---|---|---|
| ET_1 | 61.5 | 9 |
| ET_2 | 61.5 | 12 |
| ET_3 | 61.5 | 15 |
| ET_4 | 76.5 | 9 |
| ET_5 | 76.5 | 12 |
| ET_6 | 76.5 | 15 |
| ET_7 | 91.5 | 9 |
| ET_8 | 91.5 | 12 |
| ET_9 | 91.5 | 15 |

**Table 3-10: Simulated Maximum Still Water and Total Water Surface Elevations for STORM IDs ET_1, ET_2 and ET_3**

| | STORM ID | | |
|---|---|---|---|
| | ET_1 | ET_2 | ET_3 |
| Without Wave Setup (feet NAVD88) | 14.0 | 14.0 | 13.7 |
| With Wave Setup (feet NAVD88) | 14.4 | 14.5 | 14.1 |
| Without Wave Setup (feet MSL) | 14.8 | 14.8 | 14.5 |
| With Wave Setup (feet MSL) | 15.2 | 15.3 | 14.9 |
| Wave Setup (feet) | 0.5 | 0.5 | 0.4 |

**Figure 3-11: Site Setting**

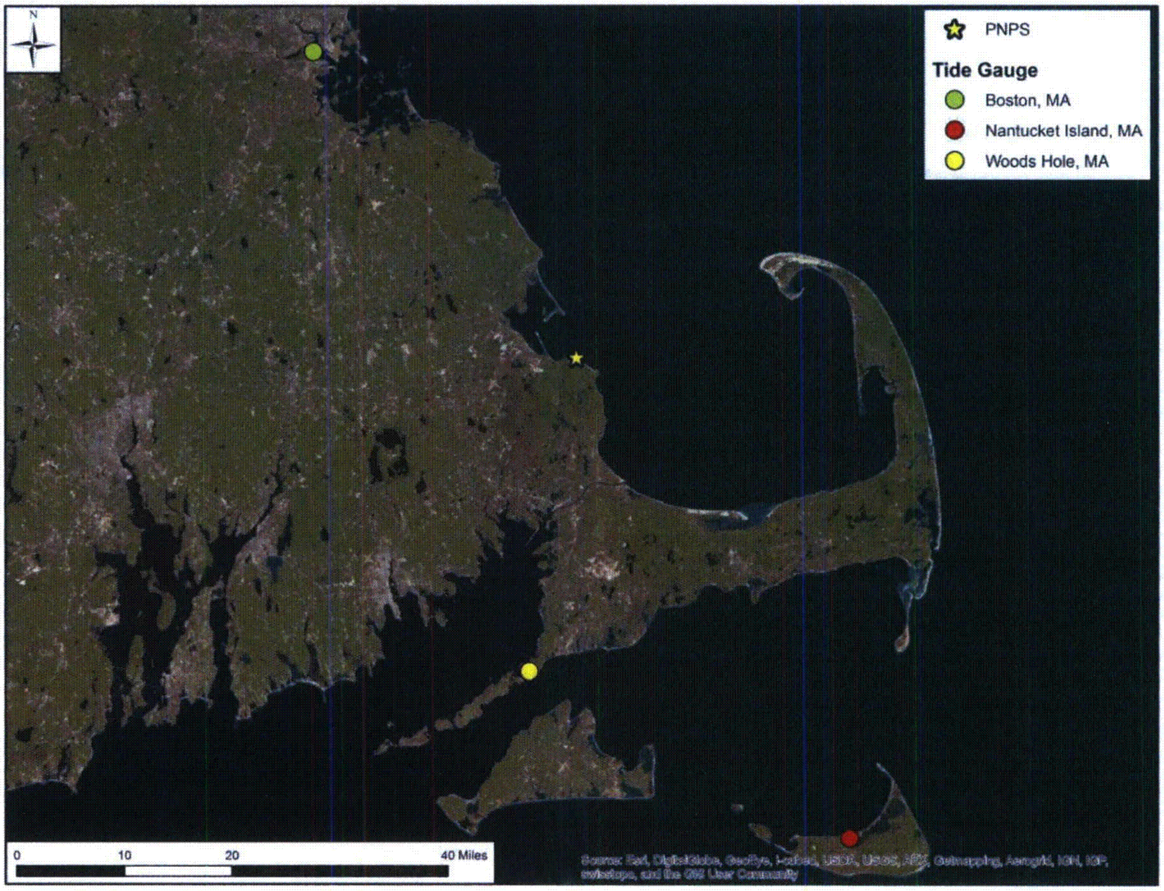
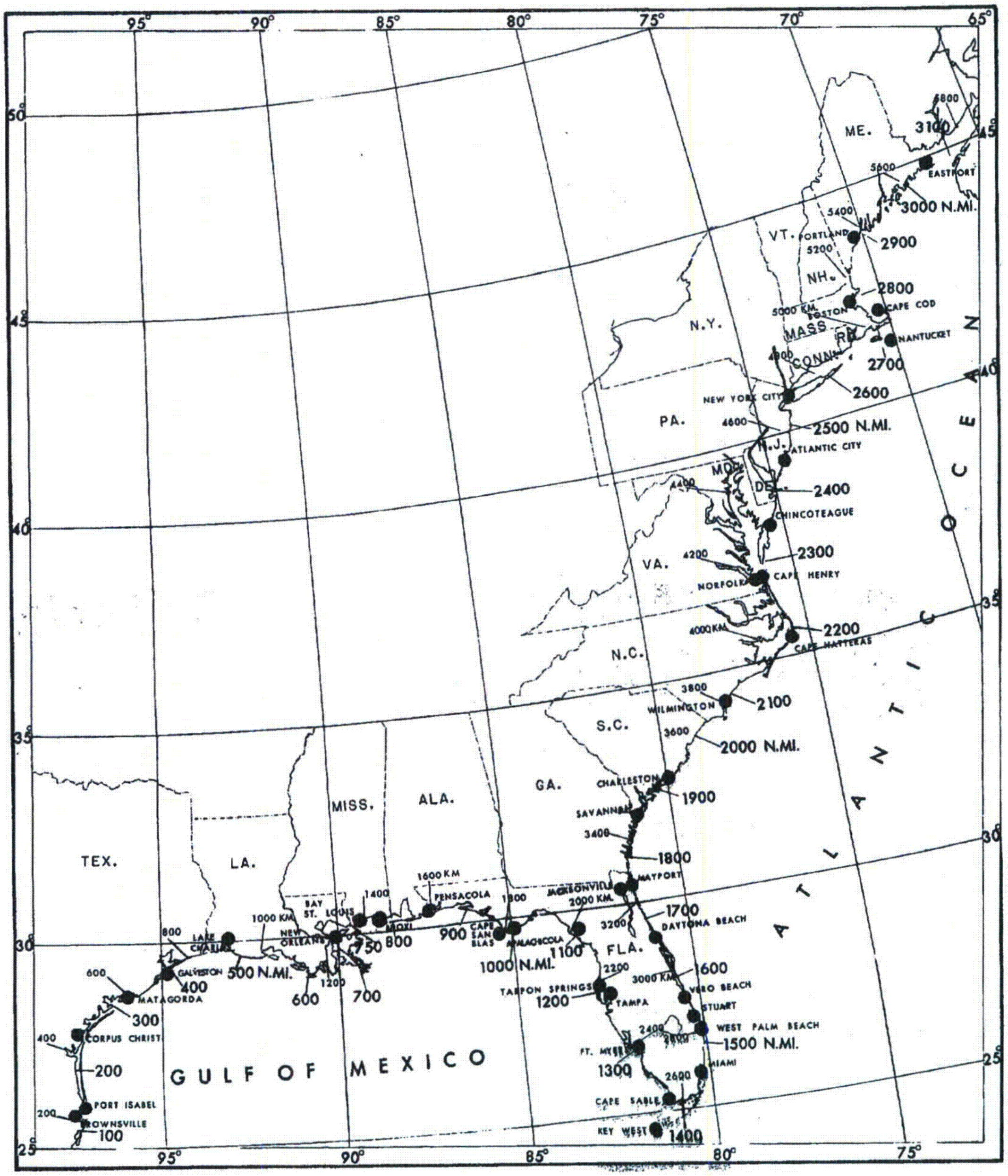


**Figure 3-12: NWS 23 Locator Map with PNPS Mile Post Identified**

[Source: NOAA 1979]




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

**Figure 3-13: HURDAT2 Storm Tracks and Six-Hour Positions**

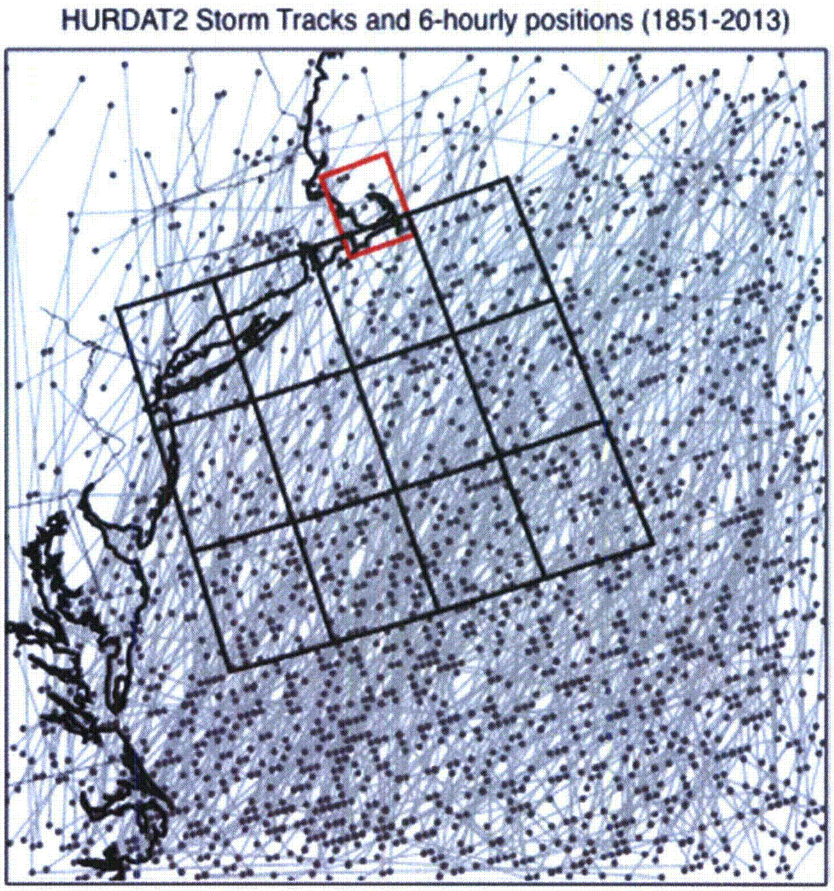
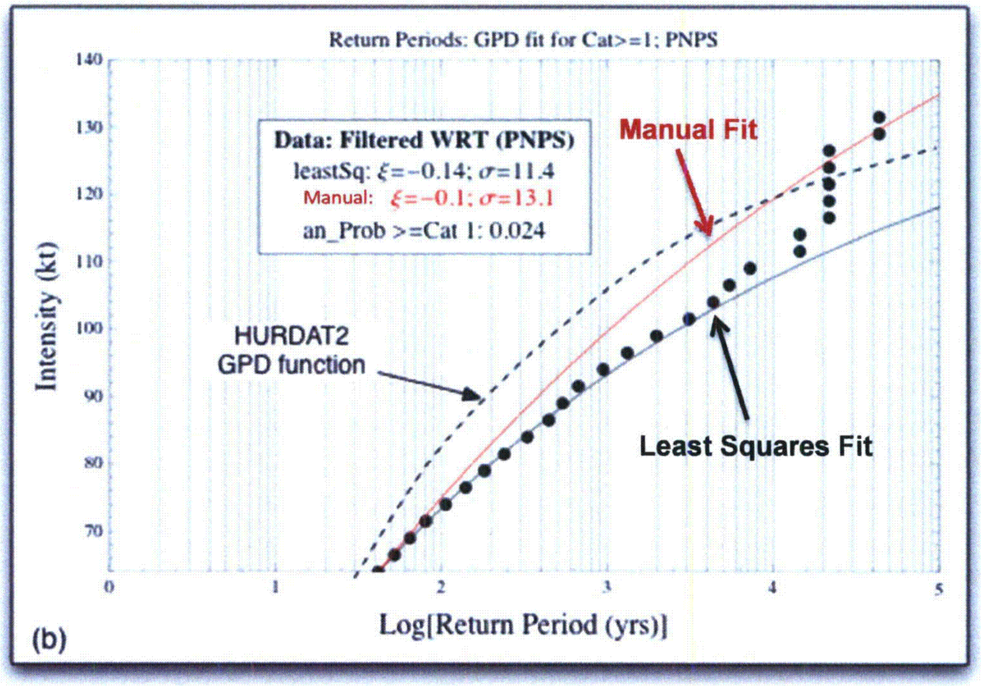
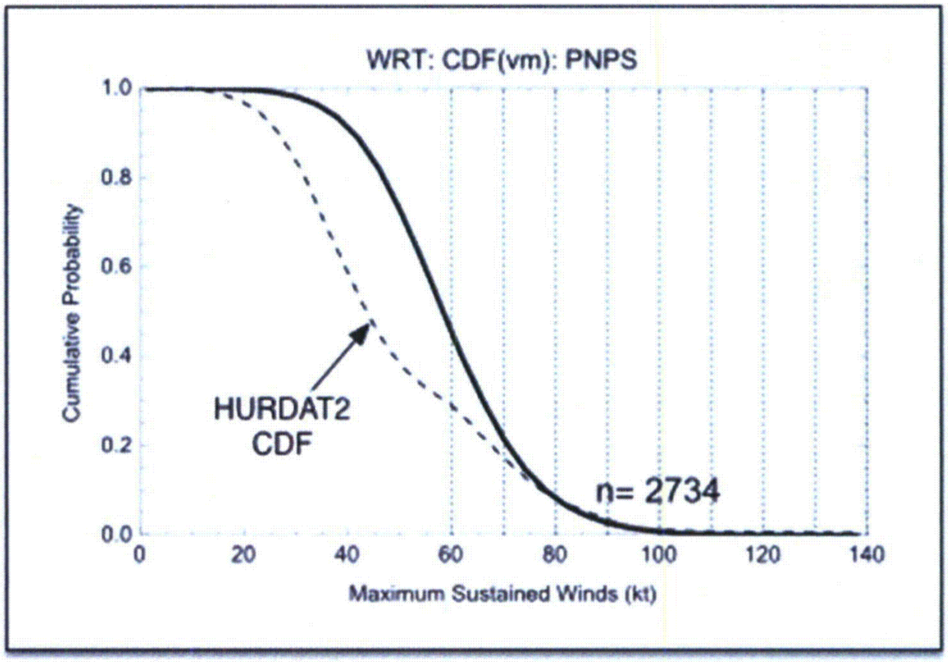


**Figure 3-14: Peak-Over-Threshold and Generalized Pareto Distribution Functional Fit to WRT Maximum Wind Speed Data within the PNPS Subregion**




Note: "Stacks" indicate empty bins (i.e., no data) below the top circle.

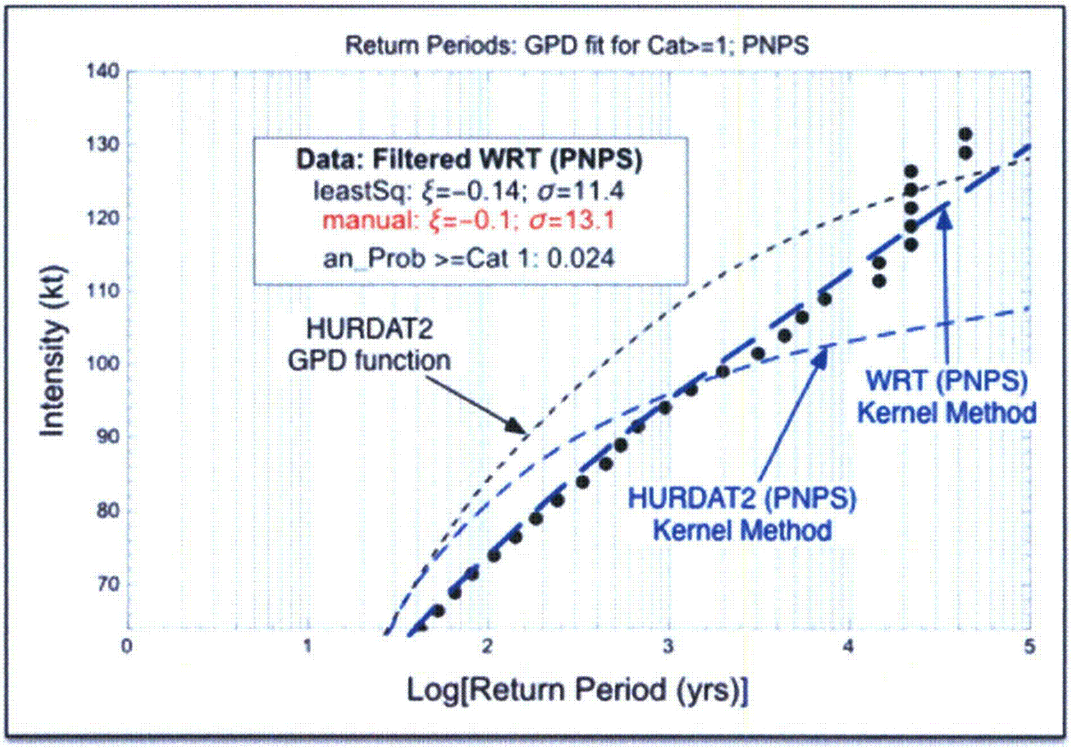
**Figure 3-15: Distribution of Maximum Wind Speed (vm) from the WRT Data within the PNPS Subregion**




Tabulated Wind Speed Exceedance Frequencies: WRT PNPS region

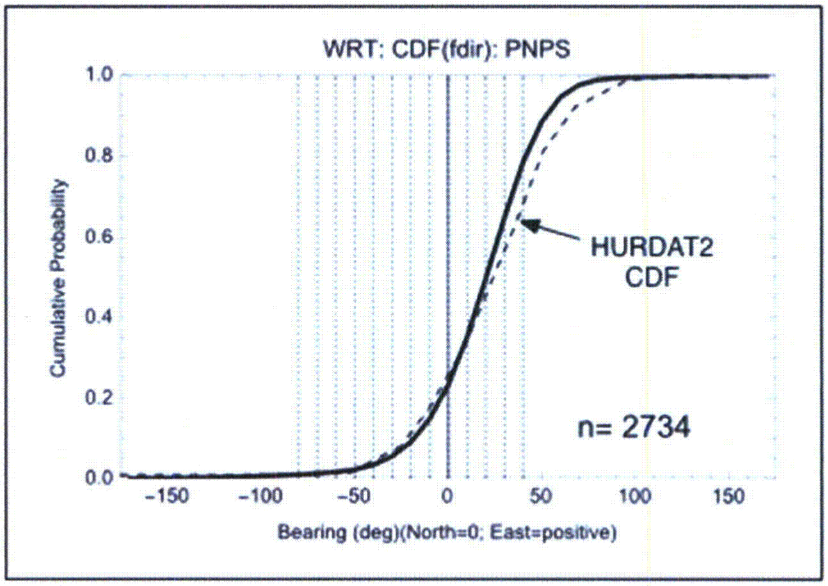
| vm | 60 | 70 | 80 | 90 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|
| Pr(vm) | 0.452371 | 0.214223 | 0.081133 | 0.027149 | 0.007962 | 0.002014 | 0.000689 | 0.000226 | -0.000016 |
| An_Freq(vm) | 0.028373 | 0.013436 | 0.005089 | 0.001703 | 0.000499 | 0.000126 | 0.000043 | 0.000014 | 0.000001 |

**Figure 3-16: Comparison of Maximum Wind Speed from the WRT Data to the Resulting Distribution Derived via the Kernel Method**




Note: “Stacks” indicate empty bins (i.e., no data) below the top circle.

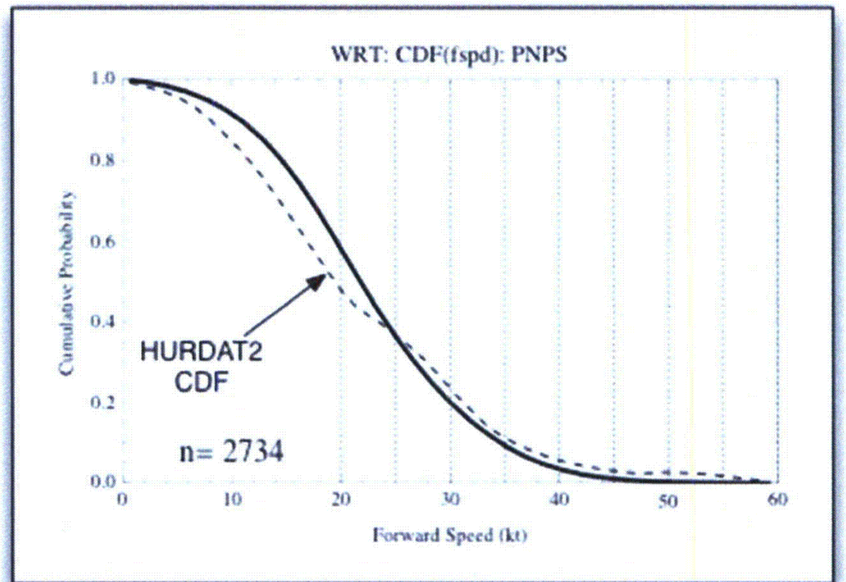
**Figure 3-17: Distribution of Storm Bearing (fdir) from the WRT Data within the PNPS Subregion**




Tabulated Track Bearing Frequencies: WRT PNPS region

| fdir | -80 | -70 | -60 | -50 | -40 | -30 | -20 | -10 | 0 | 10 | 20 | 30 | 40 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pr(fdir) | 0.0101 | 0.0126 | 0.0162 | 0.0225 | 0.0341 | 0.0552 | 0.0912 | 0.1486 | 0.2343 | 0.3521 | 0.4966 | 0.6495 | 0.7855 |
| An_Freq(fdir) | 0.0006 | 0.0008 | 0.001 | 0.0014 | 0.0021 | 0.0035 | 0.0057 | 0.0093 | 0.0147 | 0.0221 | 0.0311 | 0.0407 | 0.0493 |

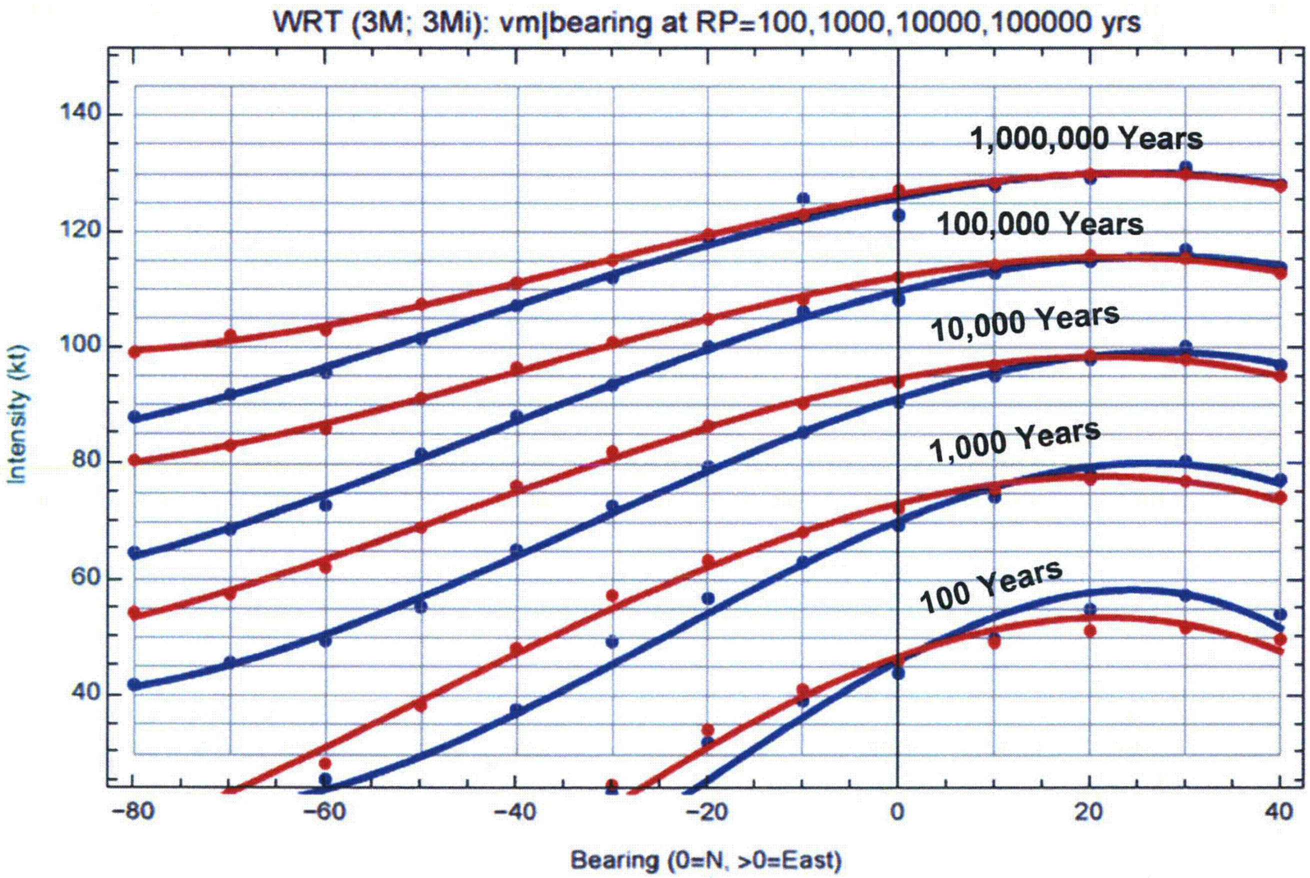
**Figure 3-18: Distribution of Forward Speed (fspd) from the WRT Data within the PNPS Subregion**




Tabulated Storm Forward Speed Frequencies: WRT PNPS region

| fspd | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 |
|---|---|---|---|---|---|---|---|---|---|
| Pr(fspd) | 0.5817 | 0.3653 | 0.2002 | 0.0928 | 0.0344 | 0.0103 | 0.0028 | 0.0007 | −0.0001 |
| An_Freq(fspd) | 0.0365 | 0.0229 | 0.0126 | 0.0058 | 0.0022 | 0.0006 | 0.0002 | 0. | 0. |

**Figure 3-19: Maximum Wind Speed as a Function of Storm Bearing at Fixed Return Periods of 100, 1,000, 10,000, 100,000 and 1,000,000 years.**

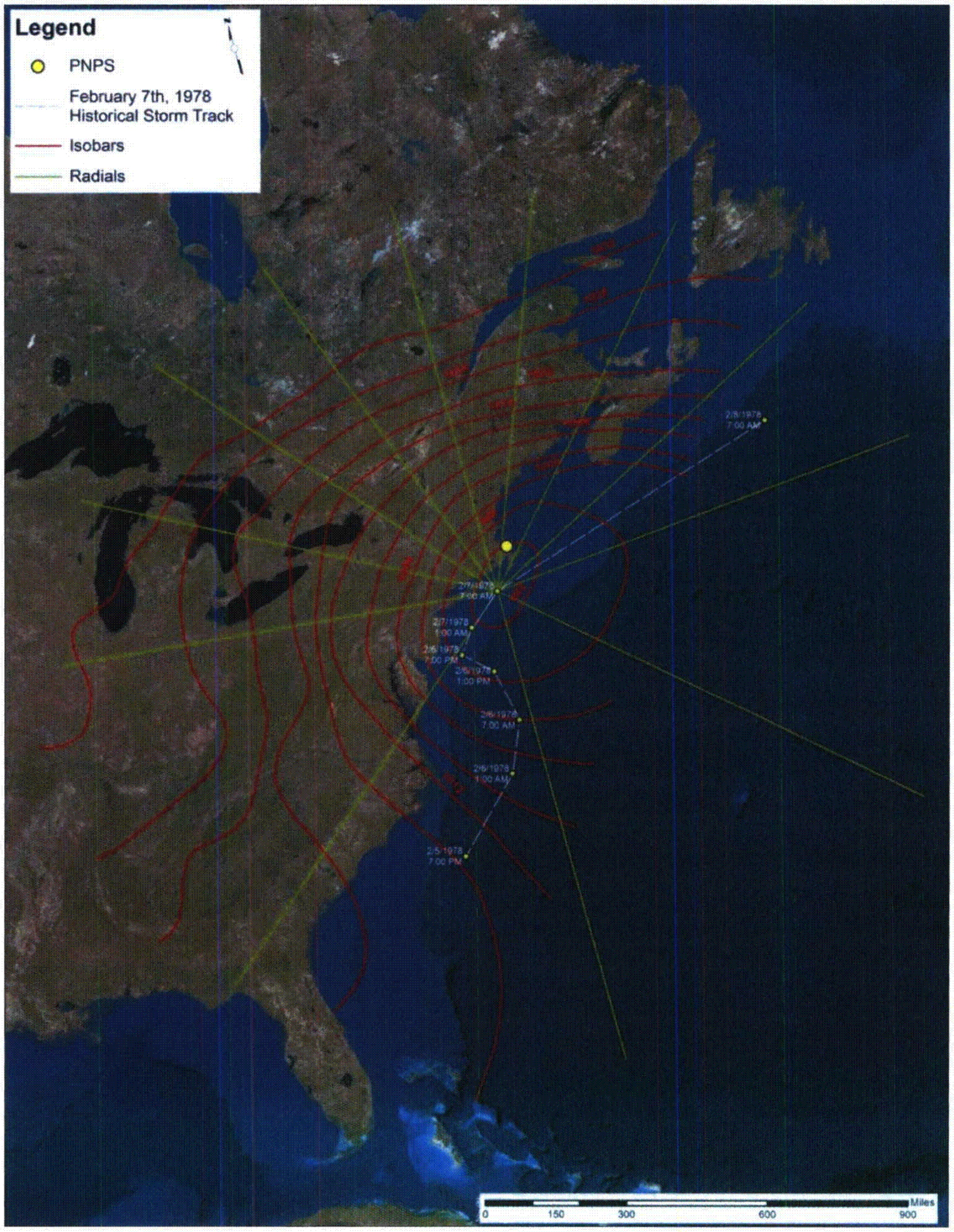



Note: Joint parameter variability is reflected in blue and independent parameter variability is reflected in red.

**Figure 3-20: Storm Tracks Associated with Historically-Significant Extra-Tropical Events in the Vicinity of PNPS**

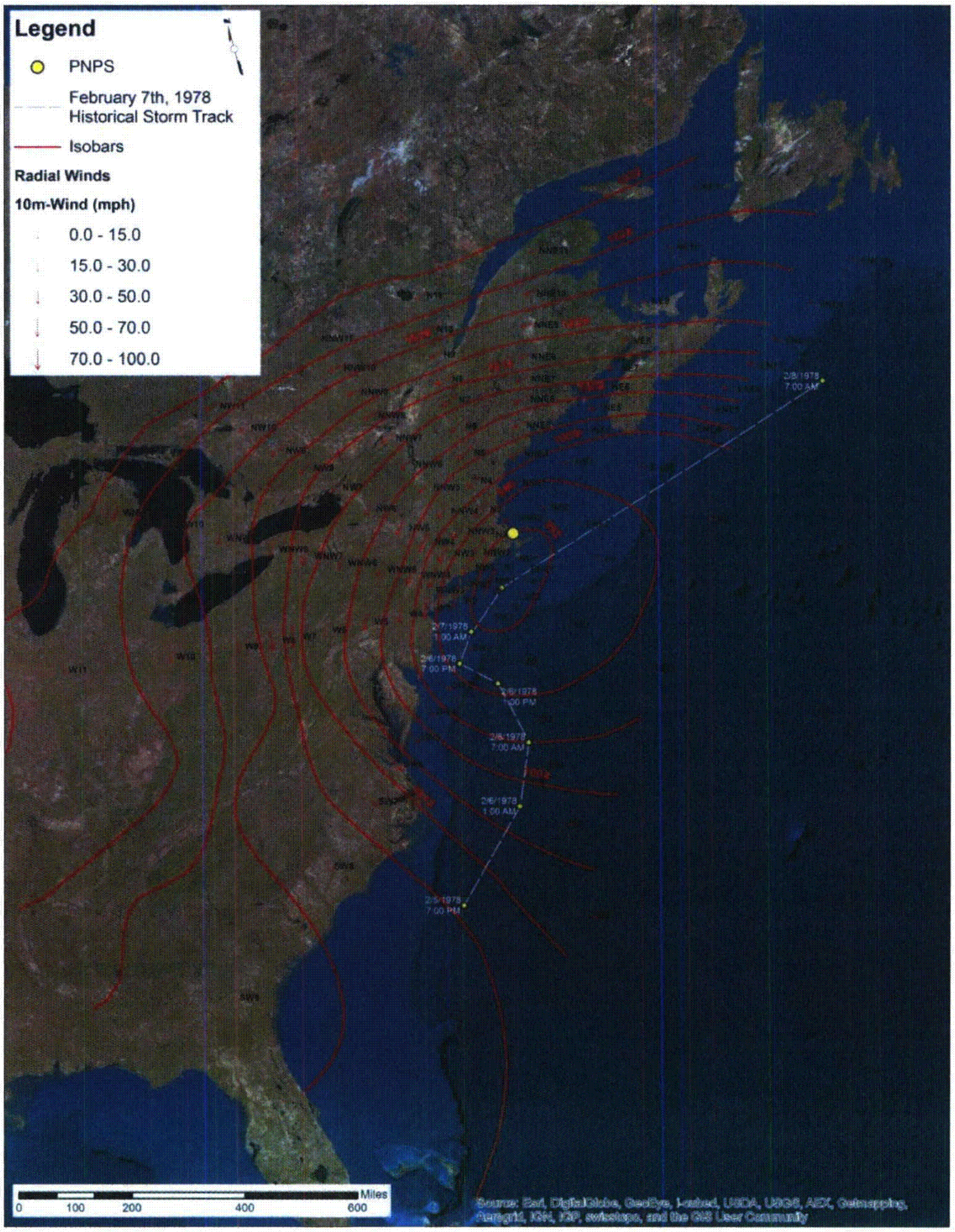

Note: Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-21: PMWS Radial Selection**




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

**Figure 3-22: PMWS Wind and Pressure Fields**




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

**Figure 3-23: SLOSH Model Basin – Output Cell Location**


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

**Figure 3-24: Sensitivity of SLOSH-simulated Storm Surge to PMH Radius to Maximum Winds and Storm Bearing**

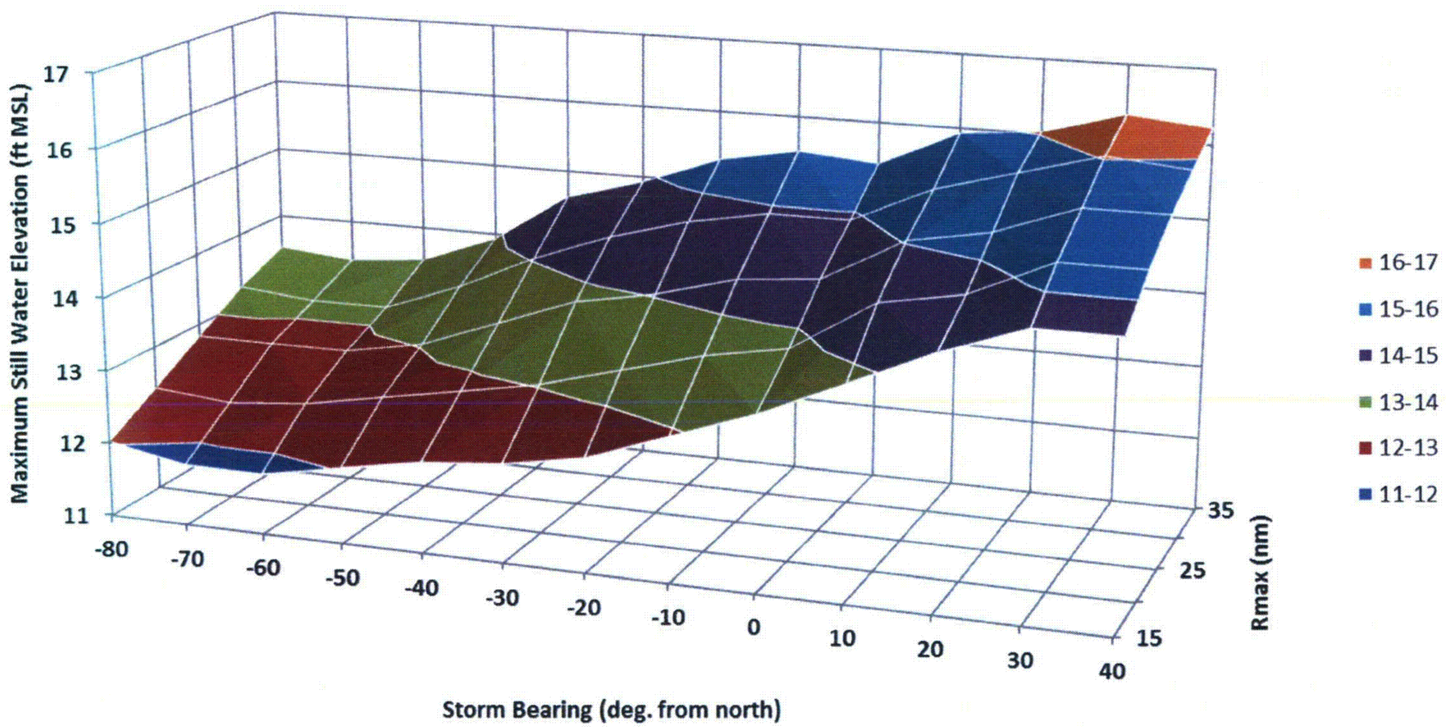


**Figure 3-25: Sensitivity of SLOSH-simulated Storm Surge to PMH Landfall Location and Storm Bearing**

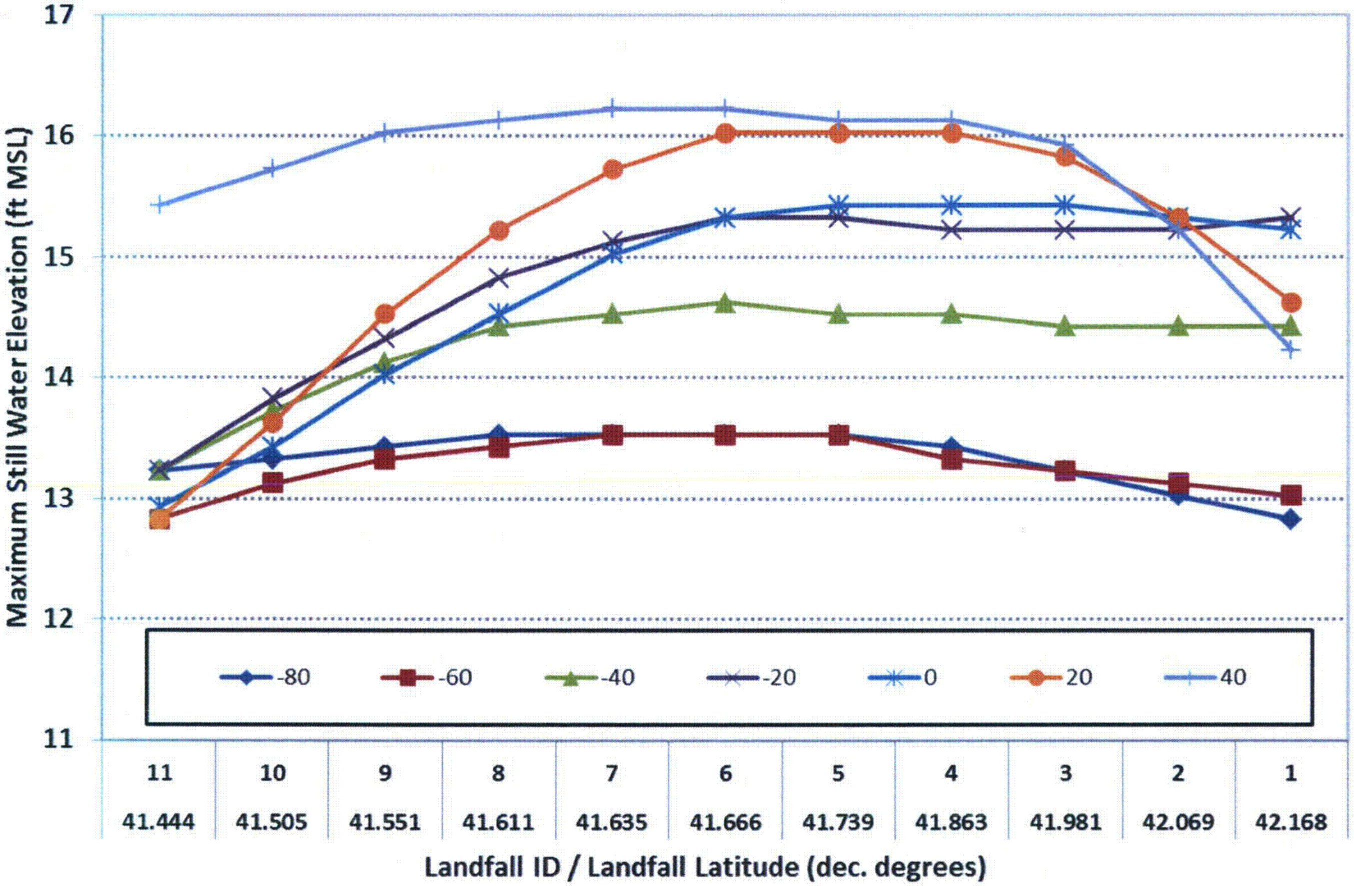


**Figure 3-26: Sensitivity of SLOSH-simulated Storm Surge to PMH Forward Speed and Storm Bearing**

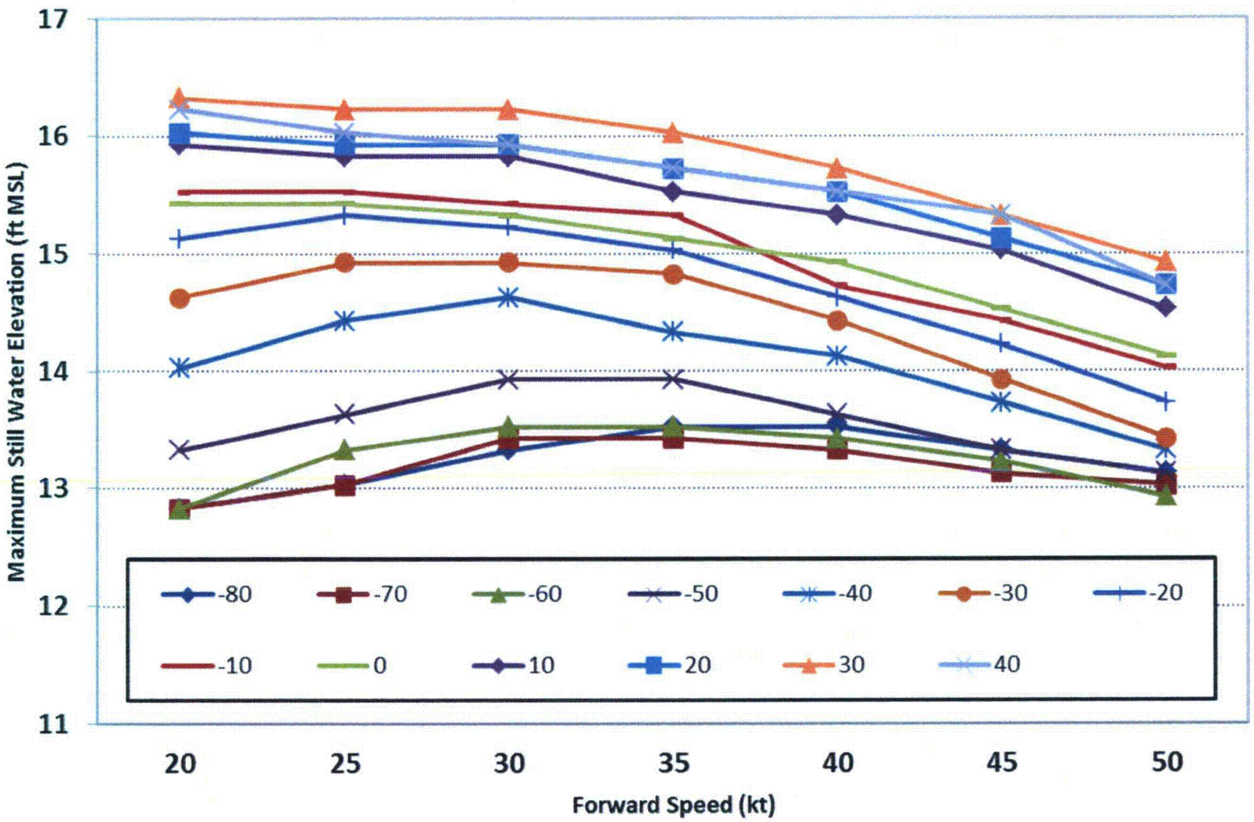
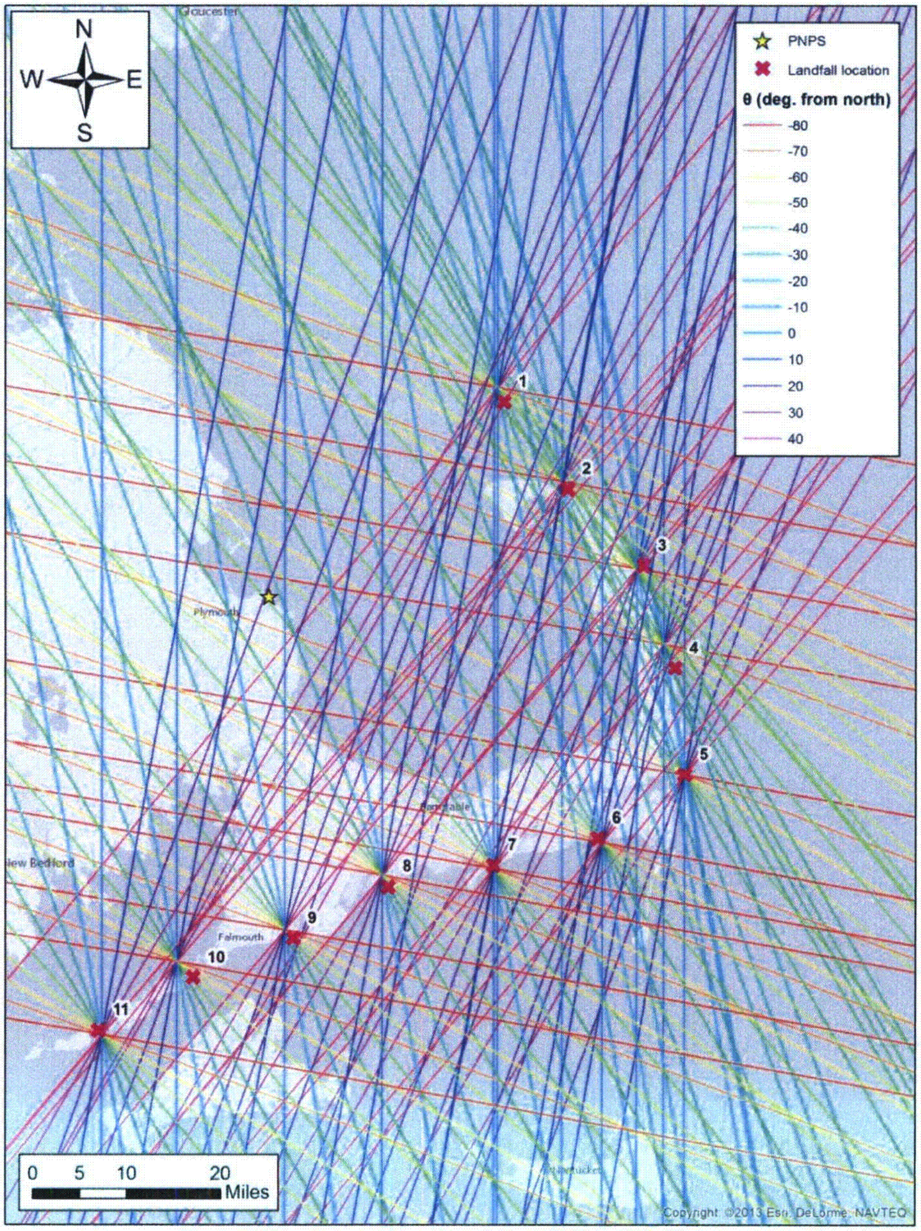


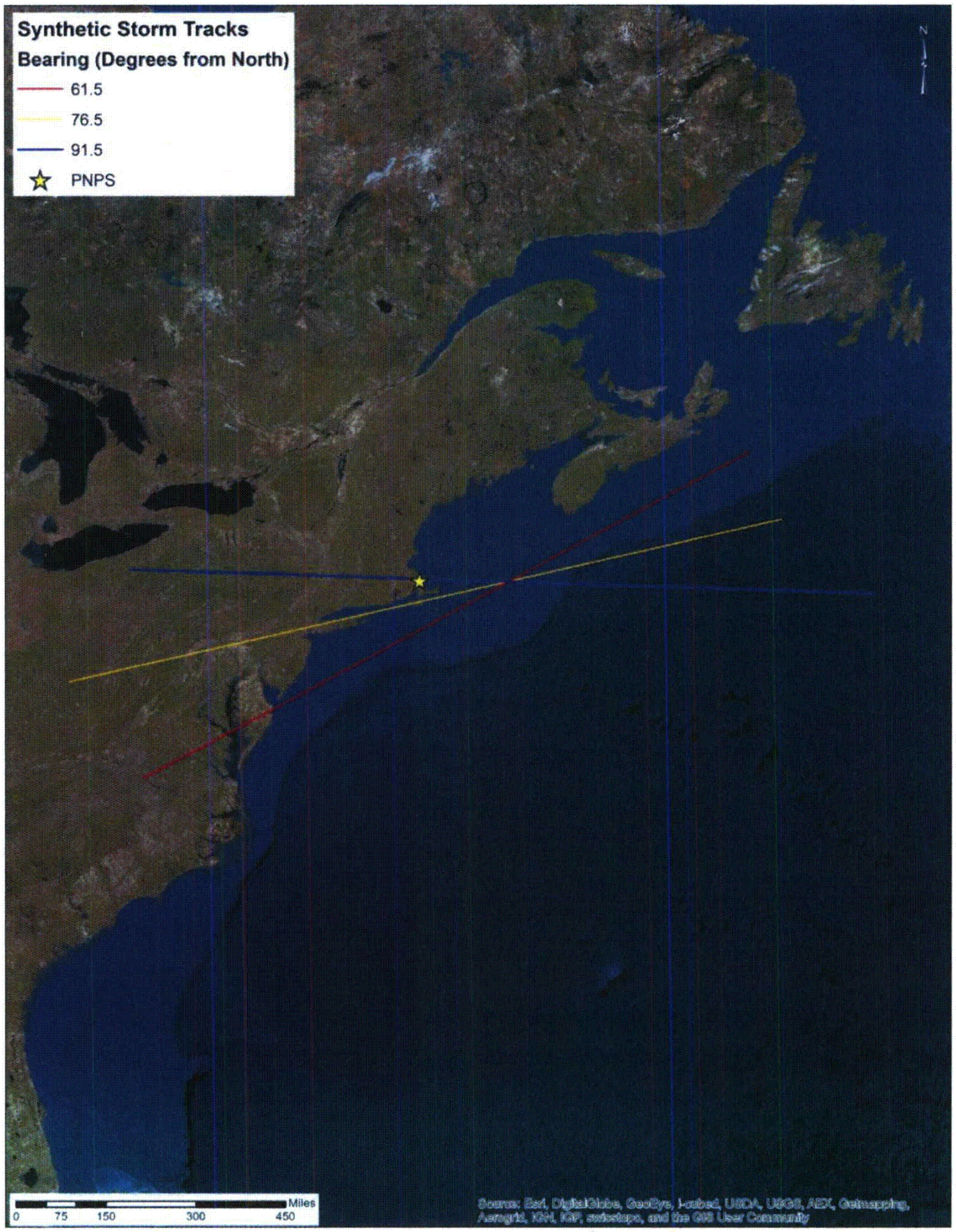
**Figure 3-27: PMH Storm Tracks Evaluated During Sensitivity Assessment with SLOSH**




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

**Figure 3-28: ADCIRC/ADCIRC+SWAN Finite Element Mesh for PNPS – PNPS Vicinity**

**Figure 3-29: PMWS Storm Tracks**



## 3.5 Seiche

This section addresses the potential for flooding at PNPS due to a seiche. Seiches are standing waves or oscillations of the free surface of a water body in an enclosed or semi-enclosed basin (Scheffner 2008). Seiches are initiated by external forcing mechanisms.

This section summarizes the seiche hazard assessment performed for the PNPS site. For further details, refer to AREVA Document No. 51-9226926-000 (AREVA, 2015).

### 3.5.1 Methodology

The HHA approach described in NUREG/CR-7046 (NRC 2011) was used to determine whether a seiche in Cape Cod Bay can result in significant flooding at PNPS. This approach initially involves the determination of the natural period of the bay, evaluation of the natural oscillation periods of the external forces and comparison of the periods to determine if resonance is possible. The intake and discharge channels were also evaluated for a seiche hazard.

### 3.5.2 Results

#### 3.5.2.1 Natural Periods of Cape Cod Bay

Cape Cod Bay, at the southernmost end of the Gulf of Maine, is a generally rectangular embayment (Davis 1992). The mouth of the bay is 17.5 miles in width, extending from Race Point (located on the bay's east side) westward to Bartlet Rock off the entrance to Green Harbor (located on the bay's west side) in the Town of Marshfield, Massachusetts, north of the Town of Plymouth. At its widest point near its south end, the bay is 24 nautical miles wide (east-west direction). In the north-south direction, the bay is approximately 20 nautical miles. The average depth of the bay is 82 feet. (PNPS 2013, Section 2.4.3.1.1 and Davis 1992)

Referring to Figure 3-30, Cape Cod Bay is an open mouthed water body (PNPS 2013, Section 2.4.3.1.1), with its north end open and its south end closed. As such, the bay was evaluated as an open-ended/semi-enclosed basin in the north-south direction, and as an enclosed basin in the east-west direction.

Using Merian's modified formula for a semi-enclosed basin (see Equation 1 below) which is based on the quarter wavelength theory, the bay's natural period in the north-south direction was estimated to be approximately 2.6 hours. Using Merian's formula for an enclosed basin (see Equation 2 below), the bay's natural period in the east-west direction was estimated to range between 1.1 hours (i.e., at the mouth of the bay) and 1.6 hours (i.e., at the south end of the bay).

$$T = 4L / (1 + 2n)(gh)^{0.5}$$ [Equation 1] (Scheffner 2008)

Where:

T is the period
L is the length of the basin
g is the acceleration due to gravity
h is the average depth of the basin
$(gh)^{0.5}$ is the shallow water wave speed
n = the number of nodes along the axis of the basin (i.e., '0' is the primary mode for a semi-enclosed basin)

$$T = 2L / n(gh)^{0.5}$$ [Equation 2] (Scheffner 2008)

Where:

T is the period
L is the length of the basin
g is the acceleration due to gravity
h is the average depth of the basin
$(gh)^{0.5}$ is the shallow water wave speed
n = the number of nodes along the axis of the basin (i.e., '1' is the primary mode in an enclosed basin)

#### 3.5.2.2 External Forcing Mechanisms in Cape Cod Bay

Referring to Section 3.5.2.1 above, external forcing, generally seismic, astronomical and meteorological in nature, must have a period of approximately one to three hours to cause resonance within Cape Cod Bay. Therefore, the periods of external forcing mechanisms were evaluated to determine if resonance with the periods of the bay is likely.

Based on the PNPS operating basis earthquake and safe shutdown earthquake response spectrums, peak seismic forcing would not exceed 10 seconds (PNPS 2013, Figures 2.5-5 and 2.5-6); therefore, resonance within Cape Cod Bay will not occur due to the large difference between the primary seiche modes of the bay and the period of seismic motions. Similarly, considering that astronomical tides are the primary forcing mechanism for flow within the bay and are dominated by semidiurnal tides (USGS 1992), the bay is not resonant near the semidiurnal frequency of 12 hours for astronomical forcing. Meteorological forcing also does not have sufficient energy to drive a seiche in Cape Cod Bay. Wind generated waves in the bay have periods ranging from eight to eighteen seconds (PNPS 2013, Section 2.4.4.2), which are too short to force a seiche in the bay. Additionally, local convection drives wind gusts with a period of about one minute, and diurnal heating and cooling also drive weak periodic motions. For periods longer than a day, wind energy typically reaches a maximum fluctuation at a period of a few days (i.e., three to seven days) due to the large synoptic scale variability of the atmosphere (Wells 1997).

#### 3.5.2.3 Natural Periods of the Intake and Discharge Channels

Referring to Figure 3-31, the PNPS intake channel is situated between the main breakwater (i.e., the northwest breakwater) and the east breakwater (i.e., the breakwater on the southeast side). The east breakwater is approximately 700 feet long, and the widest distance between the main and east breakwaters is approximately 2,000 feet (PNPS 2013, Section 2.4.4.1 and PNPS 2008). Based on bathymetric data for the northwestern portion of the intake channel, the channel has an average depth of about 12.5 feet (PNPS 2013a, Attachment G). Using Merian's formula, the intake channel's natural periods as a semi-enclosed basin and enclosed basin are estimated to be 2.3 minutes (in the northeast-southwest direction) and 3.3 minutes (in the northwest-southeast direction), respectively.

The PNPS discharge channel is located north/northwest of the intake structure (see Figure 3-31) and is protected by rock-fill jetties (PNPS 2013, Section 2.4.4.1). It is about 870 feet long (PNPS 2013, Section 2.4.4.1) and about 100 feet wide (PNPS 1970, Figure 6). Based on the mean spring tidal range depth of 10.6 feet (PNPS 2013, Section 2.4.4.2), the discharge channel's natural periods are estimated to be 3.1 minutes in the longitudinal direction and 11 seconds in the transverse direction.

#### 3.5.2.4 External Forcing Mechanisms in the Intake and Discharge Channels

Referring to Section 3.5.2.2 above, the periods for peak earthquake frequency and wind generated waves are less than the natural periods of the intake channel, whereas the tidal flow period is several orders of magnitude larger.

However, the period of one minute for wind gusts is close to the intake channel's natural periods. If a seiche were to occur in the intake channel, the height of the seiche in the northeast-southwest direction and in the northwest-southeast direction would be limited to the elevation of the breakwaters at 11.2 feet MSL. As such, it would not overtop the land side of the intake channel which has a minimum elevation of 20 feet MSL (PNPS 2013, Sections 2.4.1.2 and 2.4.4.1).

The period of the primary mode of the discharge channel in the transverse direction is near the range of earthquake frequency. However, the height of a seiche in the transverse direction would be limited by the elevation of the jetties at 16 feet mean low water (MLW). Considering that station grade is at 20 feet MSL, which is equivalent to 24.78 feet MLW (PNPS 2013, Section 2.4.4.2), the seiche would not overtop the landward end of the channel. For similar reasons, although the period of one minute for wind gusts is close to that of the discharge channel in the longitudinal direction, it would not result in flooding. Although astronomical tides in Cape Cod Bay have periods that are several orders of magnitude larger than the longitudinal period of the discharge channel and will not cause resonance within the channel, wind generated waves could occur with periods in the range of the discharge channel's transverse period; however, the channel's geometry does not allow waves to enter in the transverse direction.

### 3.5.3 Conclusions

Cape Cod Bay was identified as being susceptible to seiches. However, the large difference between the natural periods of the bay and the periods of seismic motions and semidiurnal tides precludes seismic-induced or astronomical-induced seiches. Additionally, although meteorological forcing has a broad energy spectrum, it typically does not have sufficient energy to drive a seiche in Cape Cod Bay at the estimated seiche modes. The PNPS intake and discharge channels were also identified as susceptible to seiches; however, potential seiches in the channels would not result in flooding. Therefore, no further analysis or modeling is required for the seiche flooding hazard.

### 3.5.4 References

**AREVA 2015**. AREVA Document No. 51-9226926-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Screening for Seiche, January 2015.

**Davis 1992**. Western Cape Cod Bay: Hydrographic, Geological, Ecological, and Meteorological Backgrounds for Environmental Studies by J. D. Davis, American Geophysical Union – Transferred from Springer-Verlag in June 1992. (AREVA Document No. 51-9226926-000)

**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.

**PNPS 1970**. Pilgrim Nuclear Power Station Model Wave Study, Prepared by Alden Research Laboratories, dated February 1970. (AREVA Doc. No. 38-9226908-000)

**PNPS 2008**. Pilgrim Nuclear Power Station Site Plan, EC #5000072228, dated September 22, 2008. (AREVA Doc. No. 38-9226908-000)

**PNPS 2013**. Pilgrim Nuclear Power Station Final Safety Analysis Report (FSAR), Revision 29, October 2013. (AREVA Doc. No. 38-9226908-000)

**PNPS 2013a**. Pilgrim Nuclear Power Station Document CR-PNP-2013-5246, HT - Apparent Cause Evaluation, Technical Specification Limit Exceeded for UHS Temperature, Rev. 1, dated September 3, 2013. (AREVA Doc. No. 38-9226908-000)

**Scheffner 2008**. Water Levels and Long Waves, In: Demirbilek, Z., Coastal Engineering Manual, Part II, Coastal Hydrodynamics, Chapter 5, Engineering Manual 1110-2-1100, U.S. Army corps of Engineers, Washington , D.C. (See AREVA Document No. 51-9226926-000)

**USGS 1992**. Woods Hole Branch of the U.S. Geological Survey, WHOI-92-35, Technical Report Tides of Massachusetts and Cape Cod Bays by J.D. Irish and R.P. Signell, September 1992; Available at: http://www.dtic.mil/dtic/tr/fulltext/u2/a264790.pdf; Accessed August 12, 2014. Abstract available at: http://www.researchgate.net/publication/33547852_Tides_of_Massachusetts_and_Cape_Cod_Bays, Accessed August 1, 2014. (See AREVA Document No. 51-9226926-000)

**Wells 1997**. The Atmosphere and Ocean, A Physical Introduction, by N. Wells, 2nd Edition, John Wiley & Sons Ltd., 1997. (See AREVA Document No. 51-9226926-000)

## Figure 3-30: Cape Cod Bay

[Source: Davis 1992]

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ECOLOGY AND BIOLOGY OF WESTERN CAPE COD BAY
Vol. 11

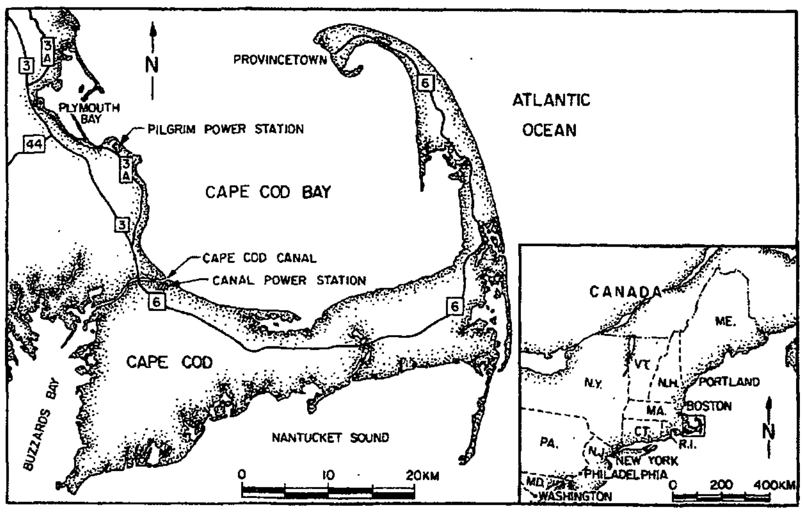
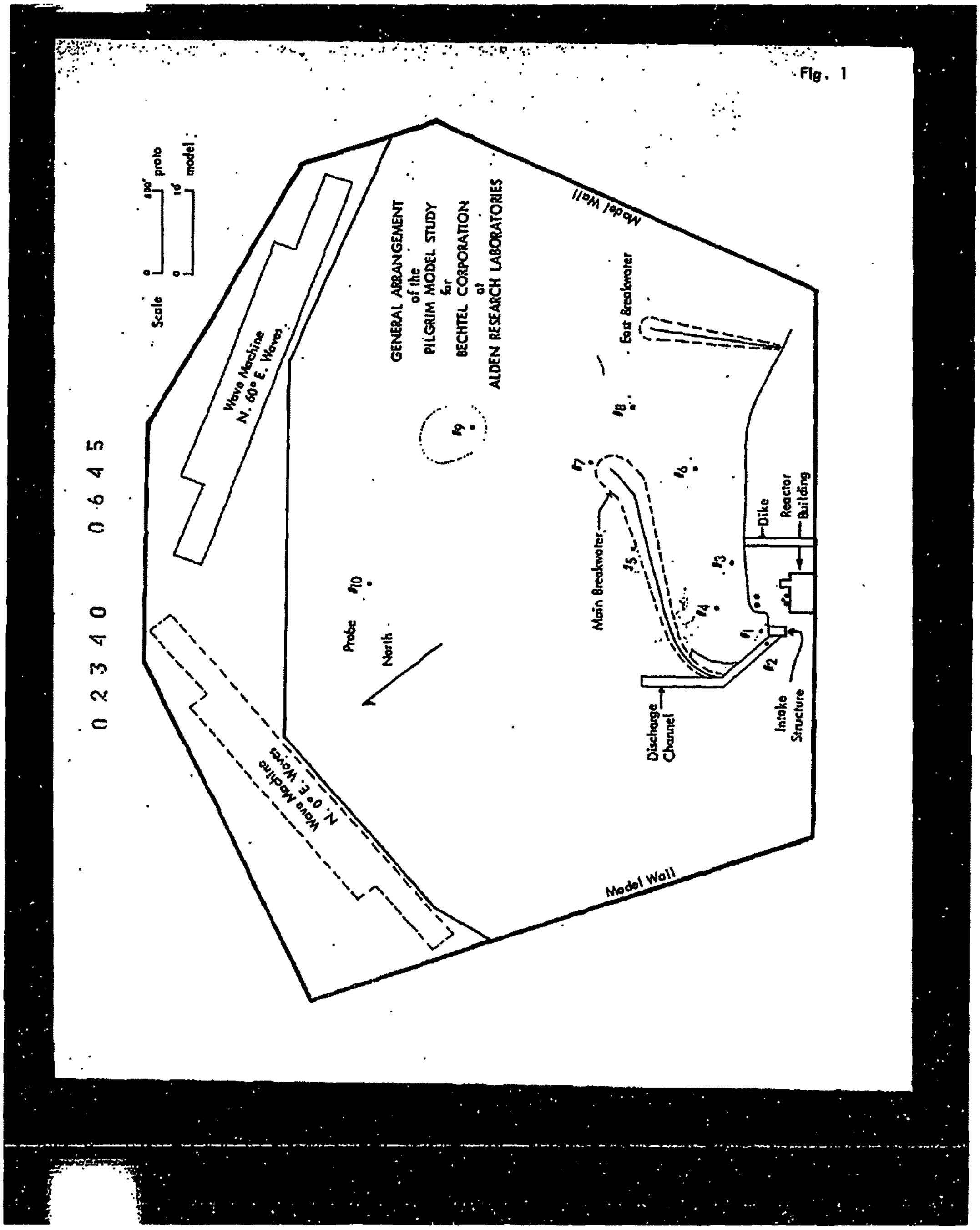



Figure 1. Cape Cod Bay, Cape Cod, Buzzards Bay, and adjacent land and waters. Locations of Pilgrim Nuclear Power Station and Canal Power Station indicated.

**Figure 3-31: PNPS Intake and Discharge Channels**

[Source: PNPS 1970]



## 3.6 Tsunami

This section addresses the potential for flooding at PNPS due to the Probable Maximum Tsunami (PMT). PNPS is considered a "coastal" nuclear power plant site; therefore, an evaluation of the potential impact of oceanic tsunamis was required. Tsunami or tsunamis are waves generated by a vertical displacement of a water column. The waves propagate radially from a subsurface point of origin, which is commonly referred to as the tsunamigenic source (NRC 2009). According to Grilli et al. 2011, examples of potential tsunamigenic sources include seismic activity (e.g., tectonic displacement) and indirect or secondary effects of seismic (i.e., co-seismic tsunamis) or volcanic activity (e.g., submarine mass failure (SMF) or volcanic flank collapse stemming from a submarine eruption). The PMT is defined as that tsunami for which the impact at the site is derived from the use of the best available scientific information to arrive at the set of scenarios reasonably expected to affect the nuclear power plant site, taking into account: 1) appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity and period of time in which the historical data have been accumulated; 2) appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena; and 3) the importance of safety functions to be performed (NRC 2009).

This section summarizes the PMT assessment performed in AREVA Document No. 51-9226934-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Screening for Tsunami (AREVA 2014).

### 3.6.1 Methodology

The HHA screening approach described in NUREG/CR-6966, Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America (NRC 2009) and Interim Staff Guidance (ISG) JLD-ISG-2012-06, Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment (NRC 2013) was used to determine if tsunami-induced inundation represents the controlling flooding mechanism at PNPS. Consistent with this approach, regional and site screening tests were performed.

The regional screening test assessed near-field and far-field tsunamigenic sources based on a review of industry-standard technical literature and available scientific data on tsunami hazards in the Atlantic Ocean. The reviewed information included: 1) the NOAA NGDC tsunami and earthquake databases (NGDC 2014); 2) a comprehensive study of the tsunami hazard in the Atlantic Ocean published by the Atlantic and Gulf of Mexico Tsunami Hazard Assessment Group (AGMTHAG 2008); and 3) a detailed literature review performed by leading researchers focusing on tsunamigenic sources associated with tsunami hazards along the Atlantic Coast (Grilli et al., 2011). The review of this literature was used identify potential tsunamigenic sources and determine the source posing the greatest potential threat to PNPS (i.e., the source associated with the PMT).

The site screening test compared the location and elevation of PNPS relative to areas affected by tsunamis in the region. Consideration was given to local characteristics, including plant grade relative to the water surface elevation and distance to the shoreline. This test was used to assess the risk of site flooding from the PMT relative to other potential flood-causing mechanisms (e.g., the Probable Maximum Storm Surge) as defined in NUREG/CR-7046 (NRC 2011).

Per the screening protocols defined by NUREG/CR-6966 (NRC 2009), more detailed analyses, which may include detailed numerical modeling of tsunami genesis and propagation, are required if projected PMT runup effects suggest risk relative to flood-protected elevations and site SSCs important to safety.

### 3.6.2 Results

Based on a review of available information, the regional screening test identified a near-field tsumanigenic source (e.g., an SMF along the continental shelf potentially triggered by seismic activity) as being the source responsible for the PMT (Figure 3-32). Three historic tsunamis have been observed resulting from either earthquakes or submarine landslides caused by earthquakes along the Nova Scotia Margin and in the Labrador Sea off the coast of Newfoundland (NGDC 2014). However, several physical characteristics that would mitigate the tsunami risk in the vicinity of PNPS were identified. These characteristics include: 1) the shallow, generally-continuous bank/shelf of Georges Bank, the Scotian Shelf, and Nova Scotia, which enclose and protect the Gulf of Maine; 2) the southern and eastern outer banks of Cape Cod, which protect Cape Cod Bay to the south and east; 3) the shallow banks (e.g., Stellwagen Bank), which extend from the northern tip of Cape Cod toward Cape Ann to the north; 4) the significant distance (i.e., over 200 miles) between the outer edge of the continental shelf (i.e., primary potential SMF tsunami source) and PNPS; and 5) the orientation of the continental shelf and slope (i.e., and potential SMFs along the slope) relative to PNPS (Figure 3-33).

The site-screening test identified shielding resulting from the location of PNPS relative to the hooked peninsula of Cape Cod and northwest-trending shallows, which include Stellwagen Bank. Due to this shielding and in recognition of the orientation of the continental shelf, flooding effects at PNPS due to tsunami events were determined to be bounded by other potential flood-causing mechanisms, specifically the PMSS.

### 3.6.3 Conclusions

While tsunamis may be generated from far-field and near-field sources, the regional screening test suggested that a near-field source, such as an SMF along the continental shelf, represents the most significant tsunami hazard risk in the vicinity of PNPS. However, review of the NOAA NGDC tsunami database (NGDC 2014) did not identify any documented, historic tsunami events along the U.S. East Coast that resulted in significant historical tsunami impacts (i.e., runup heights greater than two to three feet). Furthermore, local and regional physical characteristics, such as the protection provided by the orientation of Cape Cod, Georges Bank, the Scotian Shelf, and Nova Scotia, greatly reduce the potential impact of a tsunami generated by a near-field source in the PNPS vicinity.

The results of the site screening test suggested that a tsunami would not be a significant contributor to flooding potential at PNPS in consideration of 1) the site's location relative to Cape Cod Bay; 2) the relatively low level of exposure to the open ocean; and 3) the complex geography and bathymetry of the region (i.e., Cape Cod and Georges Bank). Therefore, a tsunami was not considered to be the controlling flood hazard at PNPS.

No further analysis or modeling is required due to the results of the screening analysis, as potential tsunami events will not cause the controlling flooding event at PNPS and will not impact SSCs important to safety.

### 3.6.4 References

**AGMTHAG 2008**. Atlantic and Gulf of Mexico Tsunami Hazard Assessment Group, Evaluation of Tsunami Sources with the Potential to Impact the U.S. Atlantic and Gulf Coasts - An Updated Report to the Nuclear Regulatory Commission, U.S. Geological Survey, 2008. (See AREVA Document No. 51-9226934-000)

**AREVA 2014**. AREVA Document No. 51-9226934-000, Pilgrim Nuclear Power Station Flooding Hazard Re-evaluation – Screening for Tsunami, 2014.

**Grilli et al., 2011**. Grilli, S. T., Harris, J. C., & Tajalli Bakhsh, T., Literature Review of Tsunami Sources Affecting Tsunami Hazard Along the U. S. East Coast, Center for Applied Coastal Research, University of Delaware/University of Rhode Island, 2011. (See AREVA Document No. 51-9226934-000)

**NGDC 2014**. National Geophysical Data Center / World Data Service: Global Historical Tsunami Database. National Geophysical Data Center, NOAA, doi:10.7289/V5PN93H7; Accessed: July 28, 2014. (See AREVA Document 51-9226934-000)

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

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

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

**Figure 3-32: Tsunamigenic Source Locations**

[Source: Grilli et al., 2011]

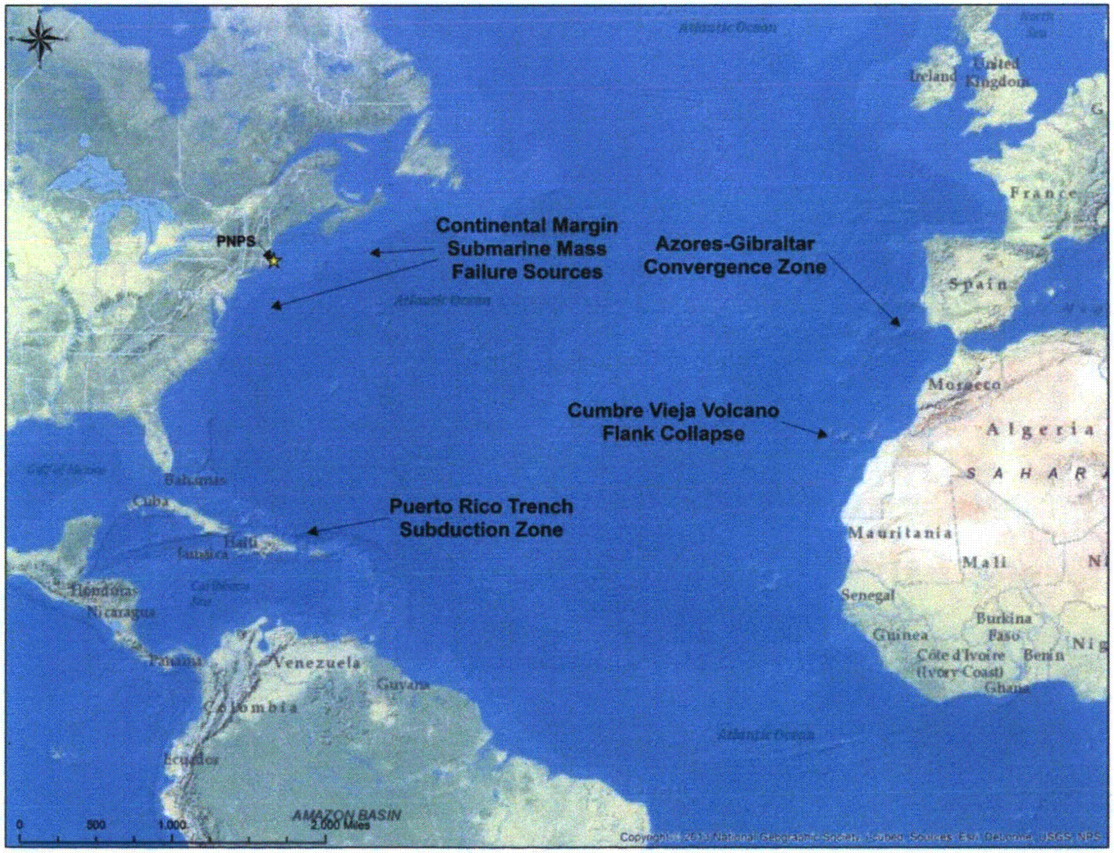
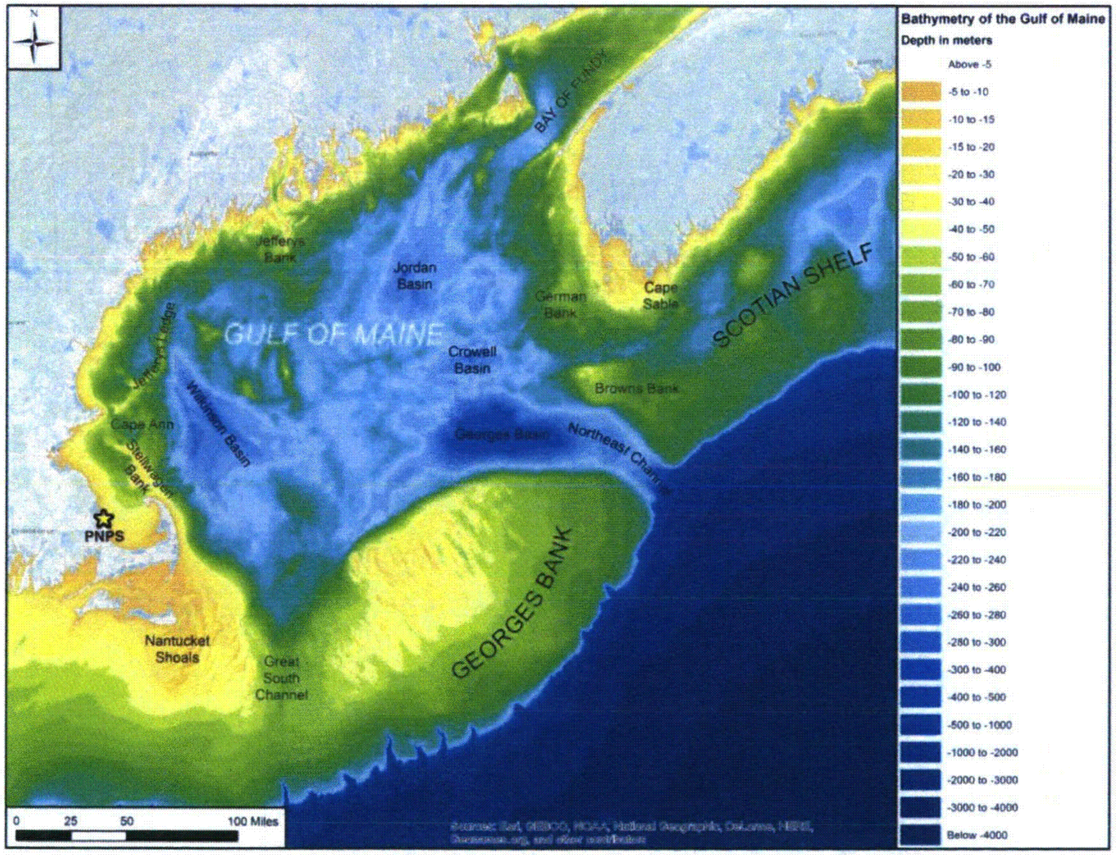


**Figure 3-33: Gulf of Maine Bathymetry**



## 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 PNPS 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 was performed and revealed that there have been no ice jams on or near Cape Cod Bay (USACE 2014). Air temperature data summaries for the nearest National Climatic Data Center (NCDC) meteorological stations to PNPS, Plymouth-Kingston and Plymouth Municipal Airport stations, were also reviewed and indicated that for the years of 1981 to 2010, the annual/seasonal normal, winter minimum temperature for both stations was near 18 °F. In addition, the normal minimum temperature for the month of January for the Plymouth-Kingston Station was 17.9 °F and that for the Plymouth Municipal Airport was 18.6 °F (NCDC 2014). Therefore, although temperatures along the Cape Cod Bay shoreline are typically tempered due to relatively warm water in the winter compared to temperatures for inland locations (PNPS 2013, Section 2.3.1), since conditions can potentially exist along the bay's shoreline for frazil ice to form, a site assessment was subsequently performed.

#### 3.7.2.2 Site Findings

As noted in the PNPS FSAR, ice glaze formation typically develops a few times each winter during favorable weather and past storms have deposited ice glaze of 0.25 inches or more in thickness in the site area, although the coastal location of the PNPS site makes it less susceptible to ice glaze formation than nearby inland locations (PNPS 2013, Section 2.3.6). Additionally, although the mean temperature of bay water is about 35 °F in the winter, the water temperature can dip below 30 °F (PNPS 2013, Figure 2.4-2). However, it is likely that the warmer water associated with service water and circulating water discharges back to the bay via the discharge channel (PNPS 2013, Sections 10.7.5 and 11.6.3) aid in suppressing the development of frazil ice.

There are no rivers adjacent to the PNPS site. Discharge for Bartlett Pond/Beaver Dam Brook and the mouth of the Eel River are approximately one and three-quarter, and two and one-quarter miles from PNPS, respectively. Similarly, there are no streams adjacent to the PNPS site. PNPS is located in a small, isolated drainage area on the northeast side of Pine Hill. Since flooding at PNPS due to ice jams or ice dams on rivers or streams is not likely, a hypothetical ice jam within the intake and discharge channels 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 the Plymouth Municipal Airport, the winter of 2013-2014 was particularly cold and resulted in a maximum AFDD value of about 410 for March 2014 (CRREL 2014). 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:

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

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 410, gives an estimated ice thickness of 16 inches. This estimate is considered conservative in regards to seawater ice thickness because it assumes a freshwater freezing point of 32 °F. The freezing point of water in Cape Cod Bay will be depressed due to its salinity content, which will mitigate the formation of surface ice. Assuming that the estimated ice thickness of 16 inches melts and becomes impounded within the intake and discharge channels when the tide is at its historical high elevation of 10.5 feet MSL (i.e., which occurred at Boston in February 1723 and is the highest still water tide level ever recorded in the site area (PNPS 2013, Section 2.4.4.2)), water within the channels would rise to elevation 11.83 feet MSL; however, the rise in water level would be below PNPS station grade at 20 feet MSL (PNPS 2013, Section 2.4.1.2). In the event that the estimated ice thickness of 16 inches was to develop in the service water pump bays when the tide is at the predicted minimum low water level elevation of (-)10.1 feet MSL (PNPS 2013, Section 2.4.4.2), the available water level would be at (-)11.43 feet MSL. Considering that the Salt Service Water System, which provides a heat sink for the Reactor Building Closed Cooling Water System, is not anticipated to be adversely impacted unless the seawater level is 13.75 feet MSL (PNPS 2013, Sections 2.4.4.2 and 10.7.1), a margin of 2.32 feet would remain for flow passage.

### 3.7.3 Conclusions

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

- There are no ice jams on record for Cape Cod Bay or on waterways in the site vicinity.
- There are no streams adjacent to PNPS in which ice jams or ice dams could form and impact PNPS. Site drainage is independent of other area drainage basins.
- Although PNPS is potentially susceptible to frazil ice formation, frazil ice would not directly result in flooding.
- A potential rise in water level within the intake and discharge channels would be below station grade for conservatively assumed ice melt conditions.
- Ice formation within the service water pump bays would still allow for sufficient service water flow.

- Ice formation on Cape Cod Bay is not likely to result in flooding at PNPS.

### 3.7.4 References

**AREVA 2014.** AREVA Document No. 51-9226932-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Screening for Ice Induced, October 2014.

**CRREL 2014.** USA Cold Regions Research & Engineering Laboratory, Accumulated Freezing Degree-Days for Massachusetts, Available at: https://webcam.crrel.usace.army.ml/AFDD/ and https://webcam.crrel.usace.army.mil/AFDD/ma_plymouth_muni.gif, dated accessed August 21 and 22, 2014. (See AREVA Document No. 51-9226932-000)

**NCDC 2014.** National Climatic Data Center, Data Tools: 1981-2010 Normals, Available at: http://www.ncdc.noaa.gov/cdo-web/datatools/normals, date accessed August 25, 2014. (See AREVA Document No. 51-9226932-000)

**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.

**PNPS 2013.** Pilgrim Nuclear Power Station Final Safety Analysis Report (FSAR), Revision 29, October 2013. (AREVA Doc. No. 38-9226908-000)

**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 August 21, 2014. (See AREVA Document No. 51-9226932-000)

## 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. 51-9226930-000 (AREVA 2014).

### 3.8.1 Methodology

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

Since there are no channels, rivers or streams adjacent to the PNPS site or nearby (i.e., Bartlett Pond and Beaver Dam Brook, which flows into Bartlett Pond, are about one and three-quarter miles to the southeast and the Eel River is about two and one-quarter miles to the west (USGS 1977 and USGS 2012)), channel diversion/shoreline erosion of Cape Cod Bay was evaluated.

#### 3.8.2.1 Regional Findings

The shoreline of Cape Cod Bay is at risk of erosion due to high winds, waves and storm surge flooding as a result of tropical and extratropical storms (hurricanes and nor'easters). As such, Plymouth County on Cape Cod Bay was selected for the Federal Emergency Management Agency (FEMA) erosion hazard mapping study (O'Connell 1999) since its shoreline ranges from low-lying beaches to high, unconsolidated sedimentary cliffs, which is representative of the coastal environments in Massachusetts. Based on 147 years of available data, FEMA's study found that although there are areas with recession rates as high as four feet per year, Plymouth County has a fairly low rate of erosion. Approximately, one third of the county was determined to have a relatively stable coastline and about 40 percent of the shoreline erosion rate was found to be less than 1.5 feet per year, with an average annual erosion rate of about 0.5 feet per year (O'Connell 1999). Based on the 1977 and 2012 topographic maps, there are no readily apparent signs of shoreline erosion on Cape Cod Bay in the vicinity of the PNPS site (USGS 1977 and USGS 2012). Referring to Figure 3-34 and Figure 3-35, the absence of significant shoreline erosion is shown on the 1:25,000 (1977) and 1:24,000 (2012) scale topographic maps by the close similarity of shoreline configuration maintained during the intervening 35 years. However, since the Cape Cod Bay shoreline is prone to erosion which may divert bay water inland, a site assessment was subsequently performed.

#### 3.8.2.2 Site Findings

The PNPS shoreline is stabilized against erosion from wind, currents, water fluctuations and storm conditions by a series of breakwaters and discharge channel jetties constructed of heavy rock (PNPS 2013, Section 2.4.4.1).

The main breakwater is 1,400 feet long and parallel to the shoreline; it separates the intake channel from Cape Cod Bay. The east breakwater is 700 feet long and perpendicular to the shoreline at the southeast shorefront. The top of both breakwaters is at elevation 11.2 feet MSL (BEC 1993, PNPS 2013, Section 2.4.4.1 and PNPS 2014). In addition to preventing rapid siltation of the dredged intake channel, the breakwaters protect the intake structure and revetments from excessive wave action and overtopping so that storm flooding at PNPS is limited. The on-shore, stone revetments on either side of the intake structure also provide shoreline stabilization and prevent flooding during severe storms (PNPS 2013, Sections 1.6.1.1.8 and 2.4.4.1).

Similarly, the 870 foot long discharge channel jetties, with a nominal elevation of 11.2 feet MSL, protect the discharge and intake structures from wave action (PNPS 2013, Section 2.4.4.1).

The breakwaters were damaged during the winters of 1977-1978 and 1978-1979 and were subsequently repaired to their original configuration. In addition to repairing the breakwaters, resolution also included a commitment to the NRC by PNPS to monitor the breakwaters on an annual basis and after major storms to ensure their integrity. (BEC 1993, Attachments 6, 9 and 10, and PNPS 2013, Section 2.4.4.1). Hence, it is unlikely that the shoreline at PNPS will experience changes due to shoreline erosion processes that would divert bay water towards the PNPS site and impact safety-related components.

### 3.8.3 Conclusions

The historical, topographic and geologic data in the region indicate that there is limited potential for diversion/erosion of the Cape Cod Bay shoreline at the PNPS site. Although the bay's shore in Plymouth County is prone to erosion, there is no apparent evidence of such for the plant's shoreline. The shoreline protection system at the plant, consisting of breakwaters, jetties and revetments, has been effective in stabilizing the site's shorefront since construction of the breakwaters in 1970 (PNPS 2013, Section 2.4.4.2).

### 3.8.4 Findings

**AREVA 2014**. AREVA Document No. 51-9226930-000, Pilgrim Nuclear Power Station Flooding Hazard Re-Evaluation – Screening for Channel Diversion, October 2014.

**BEC 1993.** IPEEE – External Flooding Analysis (Local Intense Precipitation), BEC-039, Boston Edison Company, 1993. (See AREVA Document No. 38-9226908-000)

**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.

**O'Connell 1999**. Coastal Erosion Hazards and Mapping Along the Massachusetts Shore by O'Connell, J.F. and Leatherman, S.P., Journal of Coastal Research, SI(28), 27-33, Royal Palm Beach (Florida), Spring 1999. (See AREVA Document No. 51-9226930-000)

**PNPS 2013**. Pilgrim Nuclear Power Station Final Safety Analysis Report (FSAR), Revision 29, October 2013. (AREVA Doc. No. 38-9226908-000)

**PNPS 2014**. Pilgrim Drawing C2, Rev. 10, "Site Plan". (AREVA Doc. No. 38-9226908-000)

**USGS 1977**. Manomet Topographic Quadrangle Map 1977, Massachusetts-Plymouth County, 7.5 Minute Series, Scale 1: 25 000, U.S. Geological Survey. (See AREVA Document No. 51-9226930-000)

**USGS 2012**. Manomet Topographic Quadrangle Map 2012, Massachusetts-Plymouth County, 7.5 Minute Series, Scale 1: 24 000, U.S. Geological Survey. (See AREVA Document No. 51-9226930-000)

**Figure 3-34: PNPS Shoreline in 1977**

[Source: USGS 1977]

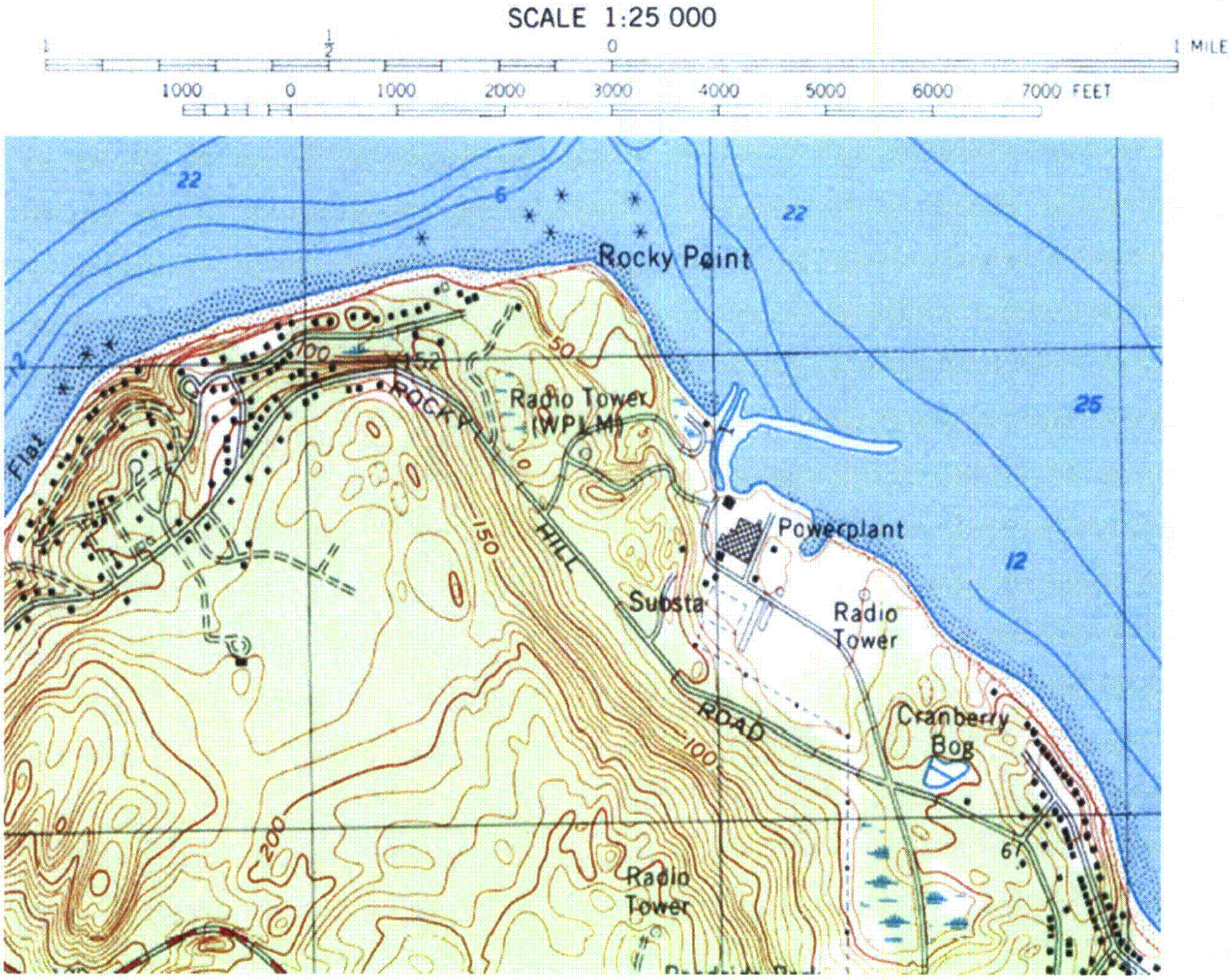
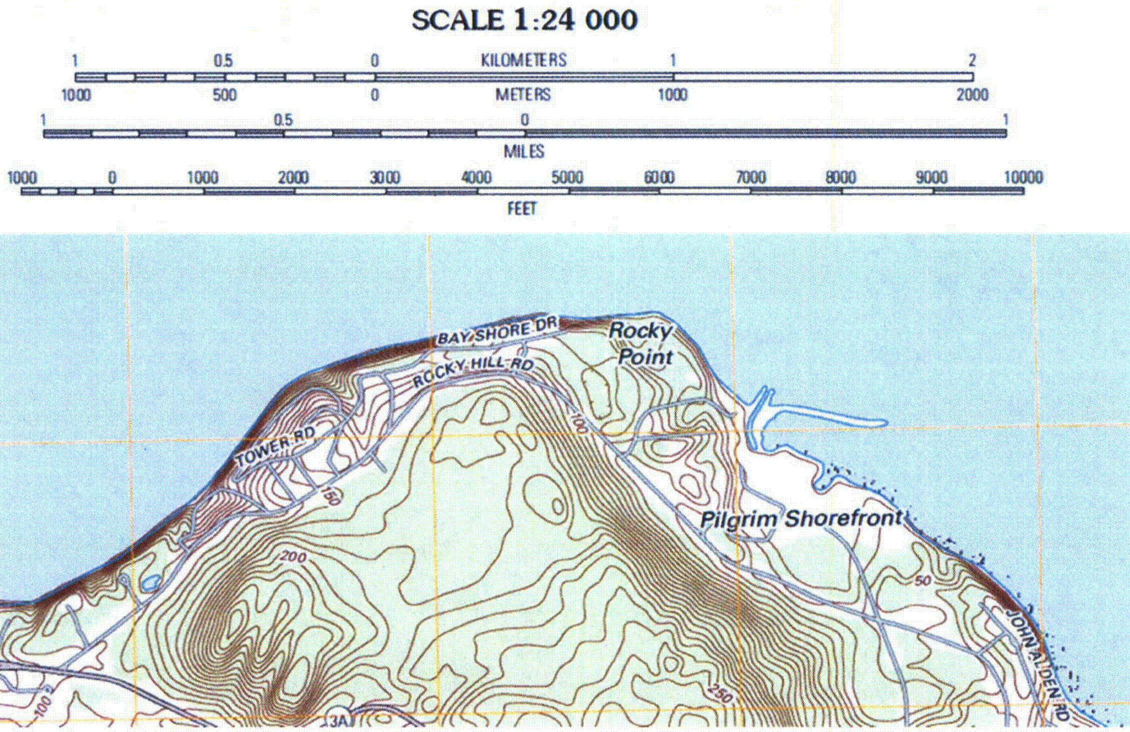


**Figure 3-35: PNPS Shoreline in 2012**

[Source: USGS 2012]



## 3.9 Combined Effect Flood

This section addresses combined events flooding at PNPS. This evaluation includes consideration of the impacts of 1) the Probable Maximum Storm Surge (PMSS) on Cape Cod Bay, which includes the design antecedent water level; and 2) wave effects associated with the Probable Maximum Hurricane (PMH) and Probable Maximum Wind Storm (PMWS). These wave effects include wave runup. Other combined events flood scenarios were assessed and screened out as not applicable at PNPS.

This section summarizes the evaluation of combined flooding events performed in the AREVA Calculation, Pilgrim Nuclear Power Station 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 PNPS: floods along shores of semi-enclosed water bodies (Scenario H.3). Other combined effect flood scenarios described in NUREG/CR-7046 were screened out as not being applicable to PNPS. The flooding impact of the Scenario H.3 combined events flood mechanism was assessed, as described below.

Scenario H.3: In consideration of the site location on the western shore of Cape Cod Bay, the H.3 combined flood event sub-scenario that is applicable to the site is:

Shore Location:

- Probable maximum surge and seiche with wind-wave activity.
- Antecedent 10 percent exceedance high tide.

The "streamside location" sub-scenario of H.3 does not apply because there are no significant rivers or streams that would contribute to a combined effects flooding event.

The methodology used to evaluate the H.3 - Shoreside combined flood event at PNPS consisted of the following steps:

1. Review of the USACE Wave Information Studies (WIS) for comparison to the simulated offshore, deep-water wave heights and periods. Historic wave data from three WIS stations near PNPS, Station 63057, Station 63060, and Station 63061 were compiled to provide a comparison to simulated deep wave heights and periods (see Figure 3-36).
2. Development of the deep-water waves resulting during the PMSS using the ADvanced CIRCulation (ADCIRC) model coupled with the Delft University of Technology's (DUT) Simulating WAves Nearshore (SWAN) model Version 41.01 of Cape Cod Bay developed in the PMSS Calculation (refer to Section 3.4). The coupling of ADCIRC and SWAN involves an integrated modeling process that is illustrated in Figure 3-37. 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 (i.e., also referred to as ADCSWAN) 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 hydrograph was used as input to the nearshore/shallow water SWAN model (described below) as it accounts for wave setup. Deep-water wave spectra outputted at the nearshore model boundary at four points were also used as the incoming parametric spectra for the nearshore model. The ADCIRC model mesh elevations are shown in Figure 3-38.

3. Development of the nearshore and shallow-water waves near PNPS resulting from the PMH or PMWS using the SWAN Version 41.01 model. A local SWAN grid was used to provide greater resolution in the vicinity of PNPS, including nearshore features such as the breakwaters.
4. Wind-wave effects, including runup, were calculated using SWAN output reflecting wave characteristics for the PMH and PMSS and by FEMA (FEMA 2007) and ASCE-7 (ASCE 2010) methodology.

The methodology was applied to the PMH and PMWS in parallel until it was determined which forcing event would produce the controlling combined events flooding.

### 3.9.2 Results

#### 3.9.2.1 Potential Shoreside Location on Semi-Enclosed Waterbody Combined Event

##### 3.9.2.1.1 Review of Historical Wave Data

Hindcasts of deep-water significant wave heights resulting from historical storms range from 23.7 to 29.1 feet and the range of peak periods is 12.6 to 17.1 seconds for the top ten wave events reported at the WIS stations (USACE 2010), see Table 3-11. The WIS stations provide a good indication of deep-water wave conditions offshore of PNPS. 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

Deep-water waves offshore of PNPS were simulated using the coupled ADCIRC and SWAN model under both PMH and PMWS conditions.

PMH

Peak significant wave heights and periods for the coupled ADCIRC+SWAN model output locations (see Figure 3-39 for output locations) are shown in Table 3-12. At the peak of the PMH, the significant deep-water wave height varies from 18.4 to 29.7 feet across the seven boundary output locations. The peak spectral wave period associated with the significant wave height range from 9.9 to 15.7 seconds at the boundary output points.

Simulated maximum significant wave heights and wave periods were compared to published data to determine the conservativeness of the model. The 100-year significant wave height at WIS station 63061 was 34.8 feet (USACE 2010). The coupled ADCIRC+SWAN output at that location (longitude -69.92, latitude 42.17) was 62.0 feet, which is 27.2 feet higher than the maximum WIS hindcast data. The large difference is the result of the extreme intensity associated with the PMH and PMWS; therefore, the coupled ADCIRC+SWAN results are considered consistent and conservative.

PMWS

Peak significant wave heights and periods for the coupled ADCIRC+SWAN model output locations (see Figure 3-40 for output locations) are shown in Table 3-13. At the peak of the PMWS, the significant deep-water wave height varies from 16.8 to 34.5 feet across the seventeen boundary output locations. The peak spectral wave period associated with the significant wave height range from 11.5 to 16.4 seconds at the output points.

Simulated maximum 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 63061 was 34.8 feet (USACE 2010). The coupled ADCIRC+SWAN output at that location (longitude -69.92, latitude 42.17) was 56.7 feet, which is 21.9 feet higher than the maximum WIS hindcast data. The large difference is the result of the extreme intensity associated with the PMWS, and therefore the coupled ADCIRC+SWAN simulation is considered conservative.

#### 3.9.2.1.3 Nearshore Wave Results

PMH

Nearshore and shallow waves in the vicinity of PNPS during the PMH were simulated using the SWAN model. The nearshore/shallow-water SWAN grid extent and elevations are shown in Figure 3-41. Figure 3-42 shows the time-varying water level that was generated by the deep-water coupled ADCIRC+SWAN model and used as input to the nearshore/shallow-water SWAN simulation. Figure 3-43 shows the time-varying wind speed and wave direction results used as input to the SWAN simulation. Output of wave characteristics was specified at nine output nodes representative of important locations and structures at PNPS (Figure 3-48 and Table 3-14). Peak significant wave heights and periods for the output locations ranged from 0.9 to 7.3 feet and 1.8 to 9.6 seconds, respectively, as shown in Table 3-14. Figure 3-46 shows the peak significant wave height and vector results within the breakwaters compared to outside of the breakwaters.

The fraction of breaking waves due to depth-induced breaking generated by the nearshore SWAN model is shown in Figure 3-50 for the PMH. Large deep-water waves break along the breakwaters before reaching the site. Shoreward structures are well beyond the breakwater structure and are therefore protected from the larger offshore waves.

PMWS

Nearshore and shallow waves in the vicinity of PNPS during the PMWS were simulated using the SWAN model. The nearshore/shallow-water SWAN grid extent and elevations are shown in Figure 3-41. Figure 3-44 shows the time-varying water level which was output from the deep-water coupled ADCIRC+SWAN model and was used as input to the nearshore/shallow-water SWAN simulation. Figure 3-45 shows the time-varying wind speed and wave direction which was used as input to the SWAN simulation. Output of wave characteristics was specified at nine output nodes which were representative of important locations and structures at PNPS shown on Figure 3-48 and Table 3-15. Peak significant wave heights and periods for the output locations ranged from 0.6 to 7.1 feet and up to 12.7 seconds, respectively, as shown in Table 3-15. Figure 3-47 shows the peak significant wave height and vector results within the breakwaters compared to outside of the breakwaters.

The fraction of breaking waves due to depth-induced breaking was output from SWAN and is shown in Figure 3-51 for the PMWS. Large deep-water waves break along the breakwater before reaching the site. Shoreward structures are well beyond the breakwater structure and are therefore protected from the larger offshore waves.

#### 3.9.2.1.4 Incident Wave Characteristics

PMH

Peak significant wave height, peak wave period and wave crest elevations of the peak significant waves for important locations are presented in Table 3-14. At the peak of the PMH, the significant wave heights at these locations vary from 0.9 to 7.3 feet. The corresponding maximum wave heights and depth-limited heights at these locations were calculated as per NRC guidance (NRC 2013). Maximum wave heights and depth-limited heights are also reported in Table 3-14. Maximum wave heights ranged from 1.5 to 12.2 feet based on the significant wave heights calculated by SWAN, and depth-limited wave heights ranged from 11.5 to 30.4 feet in the site area. Wave crest elevations for maximum waves at the peak of the PMH ranged from 16.6 to 21.9 feet MSL in the site area. The lesser of the maximum wave or breaking wave is considered in establishing the controlling combined events flood elevations, as per guidance in NUREG (NRC 2013).

PMWS

Peak significant wave height, peak wave period and wave crest elevations of the peak significant waves for important locations are presented in Table 3-15. At the peak of the PMWS, the significant wave heights at the locations vary from 0.6 to 7.1 feet. The corresponding maximum wave heights and depth-limited heights at these locations were calculated as per NRC guidance (NRC 2013). Maximum wave heights and depth-limited heights are also reported in Table 3-15. Maximum wave heights ranged from 1.0 to 11.9 feet based on the significant wave heights calculated by SWAN, and depth-limited heights ranged from 11.2 to 30.1 feet in the site area. Wave crest elevations for maximum waves at the peak of the PMWS ranged from 15.8 to 21.2 feet MSL in the site area. The lesser of the maximum wave or breaking wave is considered in establishing the controlling combined events flood elevations, as per guidance in NUREG (NRC 2013).

Because simulated wave conditions generated by the PMWS are equal to or less than those generated by the PMH, and because the maximum water surface elevation of 15.3 feet MSL resulting from the PMWS is approximately 0.5 feet lower than the maximum water surface elevation of 15.8 feet MSL resulting from the PMH, the PMH was determined to be the controlling storm event for combined effects flooding. Therefore, wave effects were calculated based on the PMSS resulting from the PMH and wind-wave effect generated by the PMH. It is noted that while the wave effects generated by the PMWS are not greater than those generated by the PMH, the duration of high intensity wave action ranges from 50 to 60 hours for the PMWS compared to 10 to 15 hours for the PMH.

#### 3.9.2.1.5 Standing Wave Height at Vertical Structures

Wave effects at the PNPS Intake Structure headwall were calculated using the Sainflou formulas for fully head-on non-breaking waves (USACE 2006). The maximum incident significant wave height near the intake was calculated in SWAN to be approximately 3.0 feet with a wavelength of 30.4 feet. This wave results in a reflected wave crest height of approximately 4.0 feet (see Table 3-16) and an elevation of 19.8 feet MSL. This elevation is approximately 1.7 feet below the top of the Intake Structure (PNPS 2005). The maximum wave height calculated at the intake headwall is approximately 5.0 feet. The maximum wave crest elevation is 20.8 feet MSL, which may result in infrequent runup wedge overtopping of the intake headwall (at 21.5 feet MSL). Overtopping due to the maximum height wave will be cycled back into the intake via the grating on the deck of the Intake Structure.

The Sainflou formulas for the reflected wave crest are for fully head-on regular waves (i.e., perpendicular to the intake headwall) and the conservatism built into the reflected wave crest elevations at the PNPS intake headwall are considered appropriate. Figure 3-46 shows the waves approaching in a general head on direction for the PMH.

#### 3.9.2.1.6 Wave Runup onto a Plateau above a Low Bluff

Wave runup in the yard area at PNPS was determined using empirical equations for runup on a rock armored slope (USACE 2006). Significant, two percent, and maximum runup from PMH waves was computed using significant wave heights as input. Initial runup estimates were adjusted using FEMA methodology for special conditions where runup may appreciably exceed the top elevation of a revetment. Wave heights ranging from approximately 0.9 feet to 7.3 feet will occur for a duration of approximately ten to fifteen hours during the PMH controlling event.

The maximum runup elevation at PNPS caused by the PMH was 22.1 feet MSL at Transect 3. This coincides with a significant wave height of 7.3 feet and mean period of 7.7 seconds. Maximum runup elevation at Transect 2 was calculated to be 21.9 feet MSL (see Figure 3-49 for transect locations and Table 3-17 for runup values.).

#### 3.9.2.1.7 Combined Events Water Elevations at PNPS

The maximum combined events water surface elevation at PNPS was determined to be 22.1 feet MSL due to runup from a fully head-on wave on the revetment slightly east of the Reactor Building portion of the plant. This results in shallow flooding of the shoreline area of the site due to overtopping flow from wave action at the revetment.

#### 3.9.2.1.8 Structure Loading and Associated Effects

Flood loading at the Intake Structure was considered. The hydrostatic loading on the Intake Structure was determined to be approximately 61,400 pounds per linear foot from the reflected wave crest elevation loading at 19.8 feet MSL. This conservatively assumes a uniform structure and does not take into account the intake opening and potential water surface elevation within the Intake Structure. The flow velocity was determined to be approximately 28.0 feet per second based on a reflected wave crest elevation of 19.8 feet MSL. This assumes an average channel invert at the reflected wave crest elevation. The hydrodynamic load was determined to be approximately 39,000 pounds per linear foot based on the reflected wave crest elevation of 19.8 feet MSL and the velocity within the channel. The debris impact load was determined to be 45,000 pounds based on a debris weight of 2,000 pounds.

A dedicated analysis of groundwater elevations was not performed as part of the combined events calculation. In general, effects of storm surge on groundwater elevations are expected to be limited to those areas currently observing tidal influence on groundwater elevations. Additionally, as stated in the Individual Plant Examination for External Events (IPEEE), the minimum entry level for all safety related structures is 23 feet MSL (IPEEE 1994, Section 5.2.1). NRC guidance states that the impact of scour, sediment transport and deposition should be considered when storm surge flood levels impinge on flood protection, safety-related SSCs and foundation materials (NRC 2013). NRC guidance states that detailed analyses should be conducted to evaluate the effects of sediment and erosion (NRC 2013).

### 3.9.3 Conclusions

The following summarizes the results and conclusions relative to combined events flooding at PNPS:

- The maximum combined events flood elevation, including wave action, near the reactor building in the site yard area between the plant buildings and the shore revetment is 22.1 feet MSL.
- The maximum combined events flood elevation at the upstream face of the Intake Structure is 19.8 feet MSL.
- Because wind-wave activity during the PMSS impacts the Intake Structure, it will be subject to hydrostatic, hydrodynamic, and wave loads.

### 3.9.4 References

**AREVA 2015**. Pilgrim Nuclear Power Station Flood Hazard Re-evaluation – Combined Events, AREVA Document No. 32-9226937-000, 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, Issue2, August 2012. (See AREVA Document No. 32-9226937-000)

**FEMA 2007**. Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update, Federal Emergency Management Agency, February 2007.

**IPEEE 1994**. Pilgrim Nuclear Power Station – Individual Plant Examination for External Events (GL 88-20), Rev. 0, July 1994. (See AREVA Document No. 38-9226908-000)

**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.

**PNPS 2005**. Intake Structure Sections, Drawing No. C46, Pilgrim Nuclear Power Station, Date Revised: December 2005. (See AREVA Document No. 38-9226908-000)

**USACE 2006**. Coastal Engineering Manual – Part VI, Chapter 5, Fundamentals of Design, EM 1110-2-1100, U.S. Army Corps of Engineers, June 2006.

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

### Table 3-11: Summary of Extreme Wave Conditions at WIS Stations near PNPS

| Rank | WIS Station 63057 | | WIS Station 63060 | | WIS Station 63061 | |
|---|---|---|---|---|---|---|
| | Significant Wave Height (feet) | Peak Wave Period (seconds) | Significant Wave Height (feet) | Peak Wave Period (seconds) | Significant Wave Height (feet) | Peak Wave Period (seconds) |
| 1 | 24.3 | 17.1 | 23.7 | 16.6 | 29.1 | 12.6 |
| 2 | 23.0 | 12.3 | 23.1 | 12.7 | 28.3 | 17.1 |
| 3 | 22.9 | 12.8 | 22.4 | 13.1 | 27.1 | 13.3 |
| 4 | 22.5 | 13.3 | 22.2 | 11.8 | 26.9 | 12.8 |
| 6 | 19.9 | 11.3 | 19.8 | 11.1 | 24.0 | 12.4 |
| 7 | 19.9 | 12.3 | 19.5 | 11.0 | 22.9 | 12.2 |
| 8 | 19.1 | 11.8 | 18.7 | 11.8 | 22.8 | 11.4 |
| 9 | 18.9 | 11.2 | 18.7 | 11.0 | 22.0 | 11.6 |
| 10 | 18.5 | 11.0 | 18.5 | 10.9 | 21.8 | 13.2 |

### Table 3-12: Coupled ADCIRC+SWAN Simulation Results – PMH

| Output Location | Peak Wave Height (feet) | Wave Period (seconds) |
|---|---|---|
| B1 | 28.0 | 10.2 |
| B2 | 28.1 | 10.1 |
| B3 | 29.7 | 10.1 |
| B4 | 27.6 | 10.1 |
| B5 | 27.3 | 9.9 |
| B6 | 22.0 | 10.6 |
| B7 | 25.0 | 9.9 |
| V1 | 22.6 | 15.7 |
| V2 | 18.4 | 10.9 |

## Table 3-13: Coupled ADCIRC+SWAN Simulation Results - PMWS

| Output Location | Peak Wave Height (feet) | Wave Period (seconds) |
|---|---|---|
| B1 | 29.5 | 16.4 |
| B2 | 29.9 | 15.9 |
| B3 | 33.3 | 15.7 |
| B4 | 33.8 | 15.6 |
| B5 | 34.5 | 15.7 |
| B6 | 22.8 | 16.2 |
| B7 | 29.1 | 15.6 |
| B8 | 34.0 | 15.7 |
| B9 | 31.0 | 15.7 |
| B10 | 26.6 | 15.5 |
| B11 | 16.8 | 15.5 |
| B12 | 27.2 | 16.3 |
| B13 | 24.4 | 16.3 |
| B14 | 21.2 | 16.0 |
| B15 | 20.7 | 11.5 |
| B16 | 19.8 | 15.5 |
| B17 | 20.1 | 15.0 |
| V1 | 22.6 | 15.7 |
| V2 | 20.0 | 15.7 |

### Table 3-14: Nearshore/Shallow Water SWAN Simulation Results – PMH

| ID Number | Description | Longitude | Latitude | Depth (feet) | Peak Significant Wave Height (feet) | Wave Period (seconds) | Peak Significant Wave Crest Elevation (feet, MSL) | Maximum Wave Height (feet) | Maximum Wave Crest Elevation (feet, MSL) | Depth-limited Wave Height (feet) | Depth-limited Wave Elevation (feet, MSL) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 12 | Intake | -70.57919583 | 41.94565794 | 39.0 | 3.0 | 9.5 | 17.3 | 5.0 | 18.3 | 30.4 | 37.1 |
| 13 | Revetment | -70.57901323 | 41.94567853 | 28.6 | 3.6 | 9.4 | 17.6 | 6.0 | 18.8 | 22.3 | 31.4 |
| 14 | Revetment | -70.5786544 | 41.94586094 | 23.0 | 5.1 | 9.6 | 18.4 | 8.5 | 20.1 | 17.9 | 28.4 |
| 15 | Revetment | -70.57785935 | 41.94570467 | 24.0 | 6.2 | 9.6 | 18.9 | 10.4 | 21.0 | 18.7 | 28.9 |
| 16 | Revetment | -70.57732438 | 41.94550411 | 22.4 | 6.8 | 9.5 | 19.2 | 11.4 | 21.5 | 17.5 | 28.0 |
| 17 | Discharge | -70.57978726 | 41.94573224 | 20.3 | 0.9 | 1.8 | 16.3 | 1.5 | 16.6 | 15.8 | 26.9 |
| 18 | Discharge | -70.57974751 | 41.94569664 | 20.2 | 0.9 | 1.8 | 16.3 | 1.5 | 16.6 | 15.8 | 26.8 |
| 19 | Discharge | -70.57979678 | 41.9458015 | 20.3 | 0.9 | 1.8 | 16.3 | 1.5 | 16.6 | 15.8 | 26.9 |
| 20 | Boat Ramp | -70.57679734 | 41.9450345 | 14.7 | 7.3 | 9.5 | 19.5 | 12.2 | 21.9 | 11.5 | 23.8 |

Note: See Figure 3-48 for locations of points.

## Table 3-15: Nearshore/Shallow Water SWAN Simulation Results – PMWS

| ID Number | Description | Longitude | Latitude | Depth (feet) | Peak Significant Wave Height (feet) | Wave Period (seconds) | Peak Significant Wave Crest Elevation (feet, MSL) | Maximum Wave Height (feet) | Maximum Wave Crest Elevation (feet, MSL) | Depth-limited Wave Height (feet) | Depth-limited Wave Elevation (feet, MSL) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 12 | Intake | -70.57919583 | 41.94565794 | 38.6 | 2.5 | 10.6 | 16.6 | 4.2 | 17.4 | 30.1 | 36.4 |
| 13 | Revetment | -70.57901323 | 41.94567853 | 28.3 | 3.0 | 10.3 | 16.8 | 5.0 | 17.8 | 22.1 | 30.8 |
| 14 | Revetment | -70.5786544 | 41.94586094 | 22.6 | 4.5 | 10.7 | 17.6 | 7.5 | 19.1 | 17.6 | 27.6 |
| 15 | Revetment | -70.57785935 | 41.94570467 | 17.8 | 5.6 | 10.6 | 18.1 | 9.4 | 20.0 | 13.9 | 25.0 |
| 16 | Revetment | -70.57732438 | 41.94550411 | 22.0 | 6.4 | 10.5 | 18.5 | 10.7 | 20.6 | 17.2 | 27.3 |
| 17 | Discharge | -70.57978726 | 41.94573224 | 19.9 | 0.6 | N/A | 15.6 | 1.0 | 15.8 | 15.5 | 26.2 |
| 18 | Discharge | -70.57974751 | 41.94569664 | 19.9 | 0.6 | N/A | 15.6 | 1.0 | 15.8 | 15.5 | 26.2 |
| 19 | Discharge | -70.57979678 | 41.9458015 | 19.9 | 0.6 | N/A | 15.6 | 1.0 | 15.8 | 15.5 | 26.2 |
| 20 | Boat Ramp | -70.57679734 | 41.9450345 | 14.3 | 7.1 | 12.7 | 18.9 | 11.9 | 21.2 | 11.2 | 23.1 |

Note: See Figure 3-48 for locations of points.

**Table 3-16: PNPS Intake Wave Effects**

| Parameter | Transect 1 | Parameter Description |
|---|---|---|
| h | 22 | Depth at intake, feet |
| T | 9.5 | Peak period from SWAN, seconds |
| L | 30.4 | Wavelength from SWAN, feet |
| Hs | 3.02 | Hs from SWAN, feet |
| Vertical Shift | 0.94 | feet |
| Wave Crest | 3.96 | feet |
| Setup | 0.10 | feet |
| Initial SWEL | 15.70 | PMSS still water SWEL, feet MSL |
| Total WSE | 19.8 | Combined Event Elevation, feet MSL |
| Height of Intake | 21.5 | feet MSL |

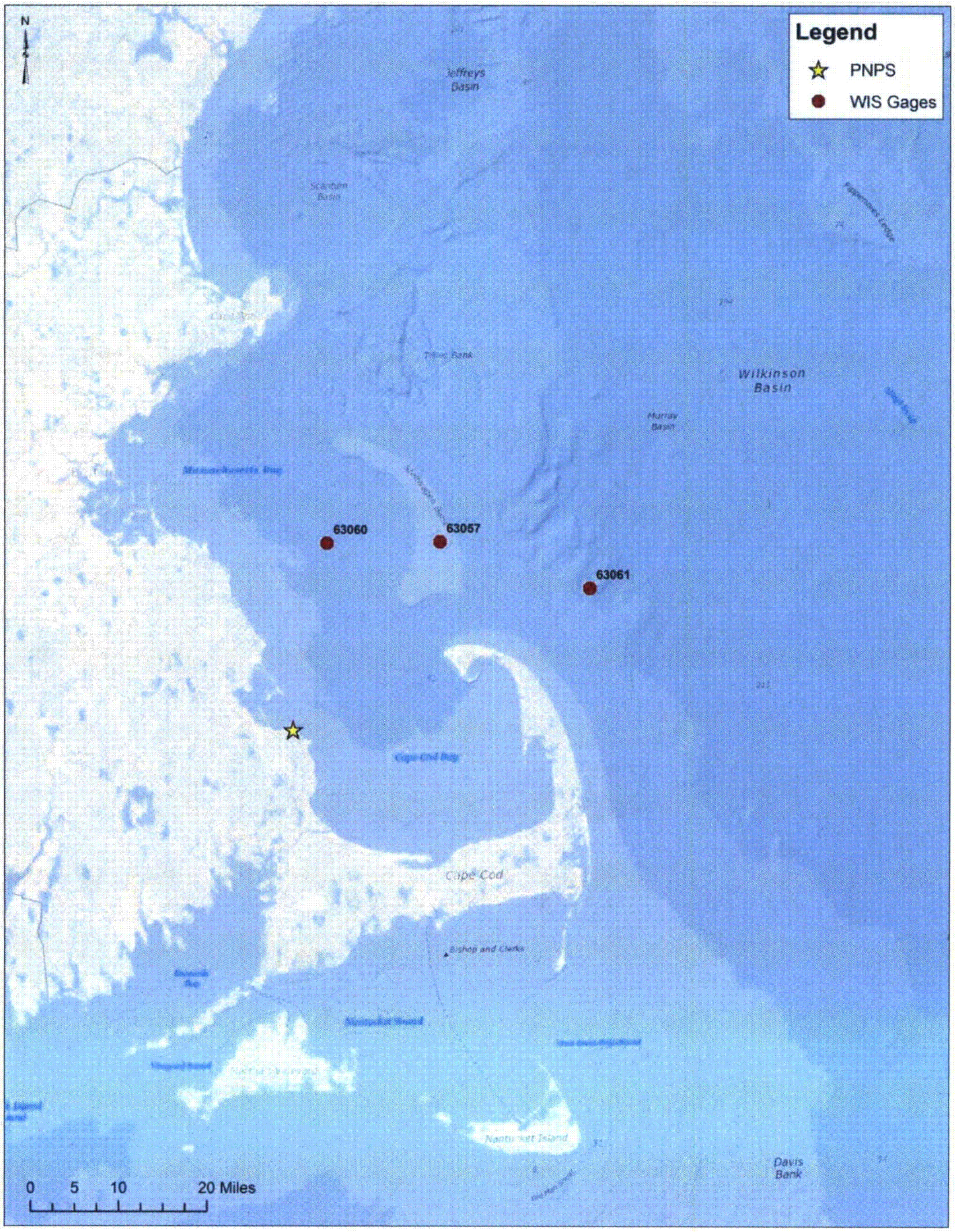
Note: See Figure 3-49 for locations of transect.

**Table 3-17: PNPS PMH Runup**

| Runup (Percent Exceedance) | Transect 2 (feet, MSL) | Transect 3 (feet, MSL) |
|---|---|---|
| **% 0.1** | 21.9 | 22.1 |
| **% 2.0** | 21.5 | 21.6 |
| **% 5.0** | 20.9 | 21.1 |

Note: See Figure 3-49 for locations of transect.

**Figure 3-36: WIS Wave Gage Locations**




Any illegible text or features in this figure are not pertinent to the technical purposes of this document.

**Figure 3-37: Coupled ADCIRC+SWAN Computation**

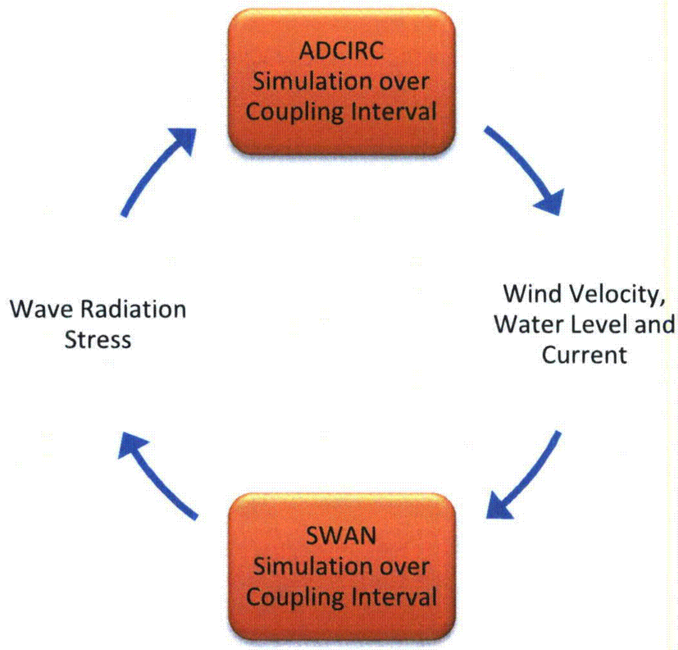


**Figure 3-38: ADCIRC Model Mesh Elevations**

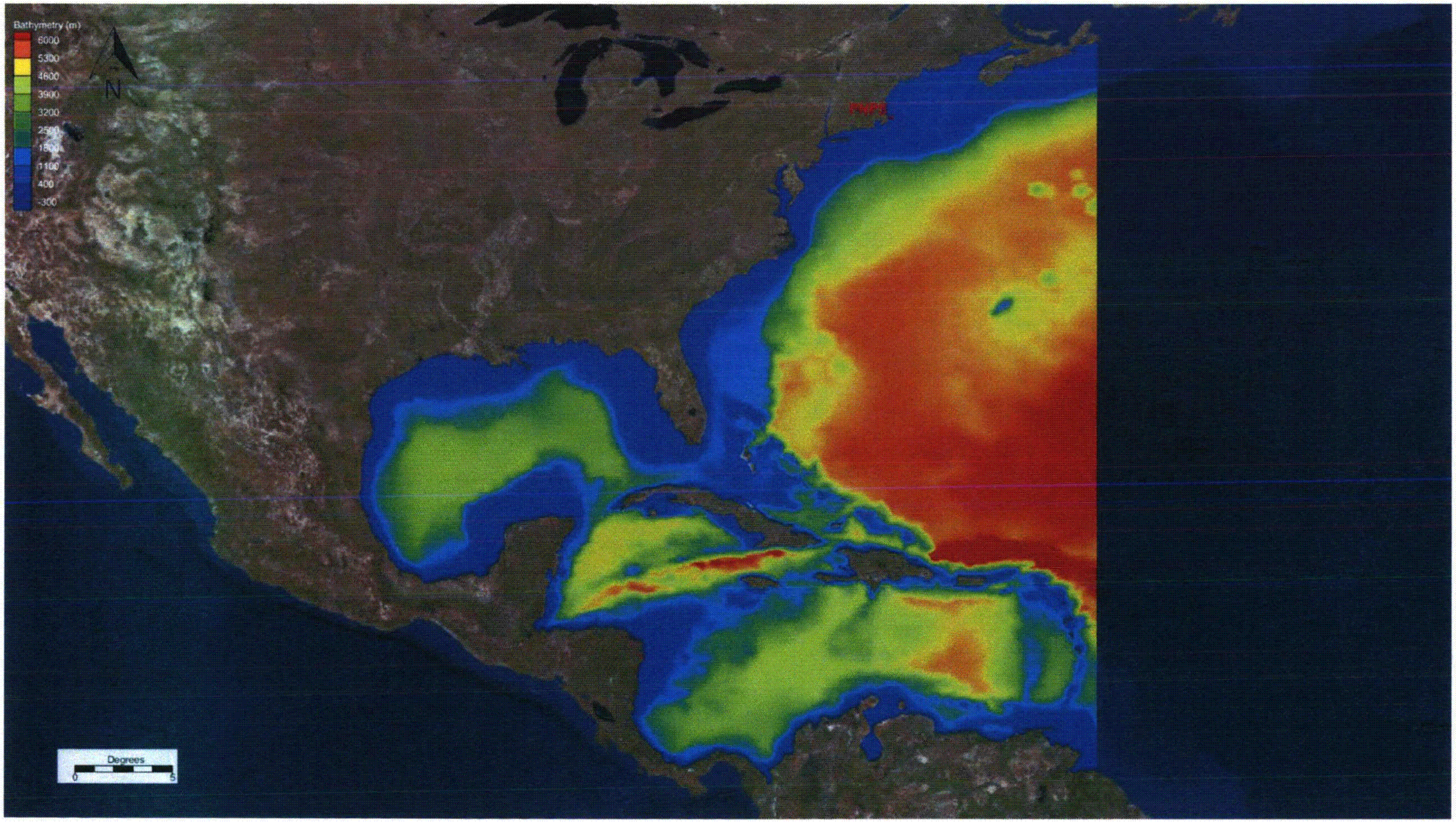


**Figure 3-39: Coupled ADCIRC+SWAN Simulation Output Locations – PMH**

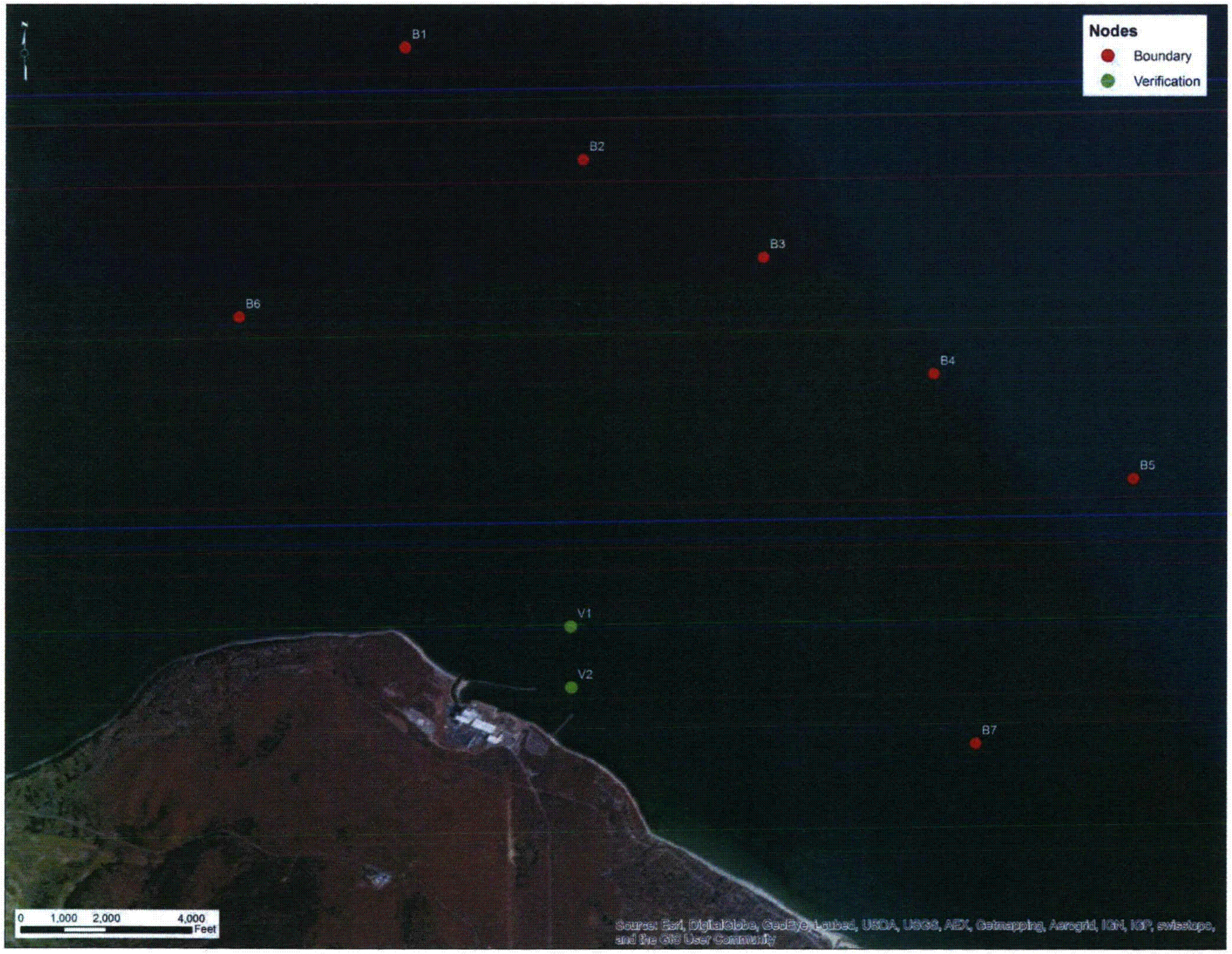


**Figure 3-40: Coupled ADCIRC+SWAN Simulation Output Locations – PMWS**

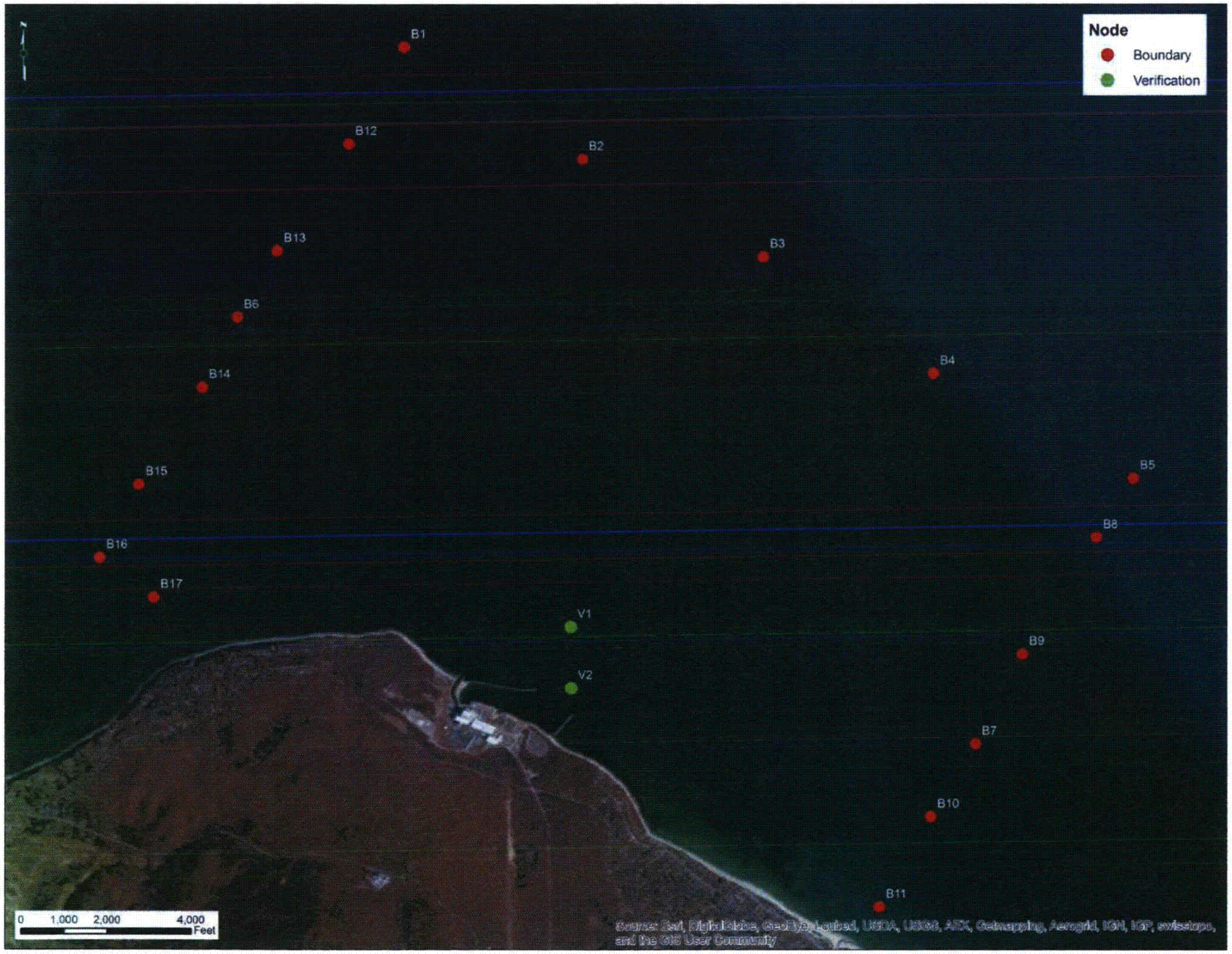


**Figure 3-41: Nearshore/Shallow Water SWAN Simulation Model Elevations**

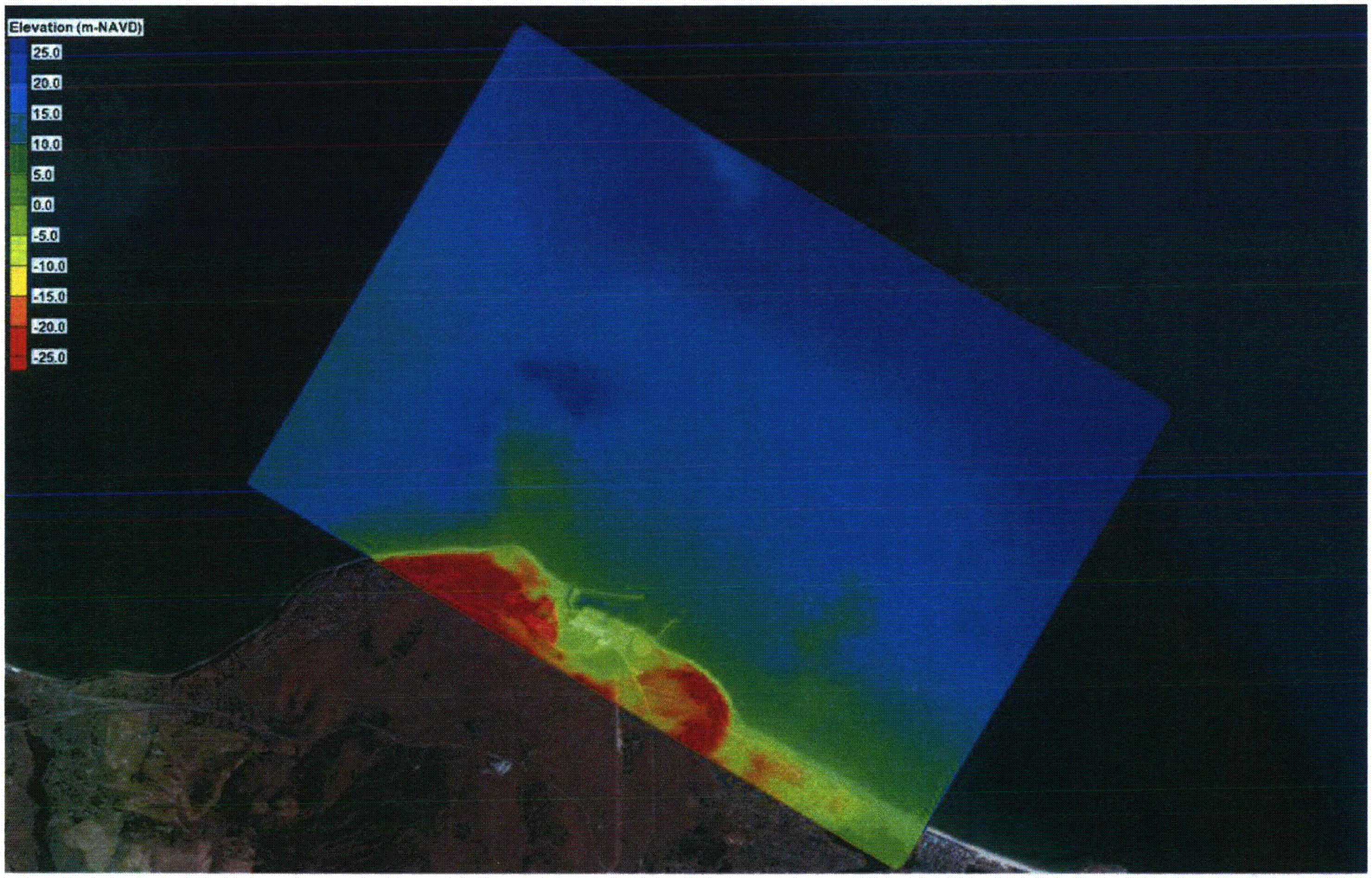
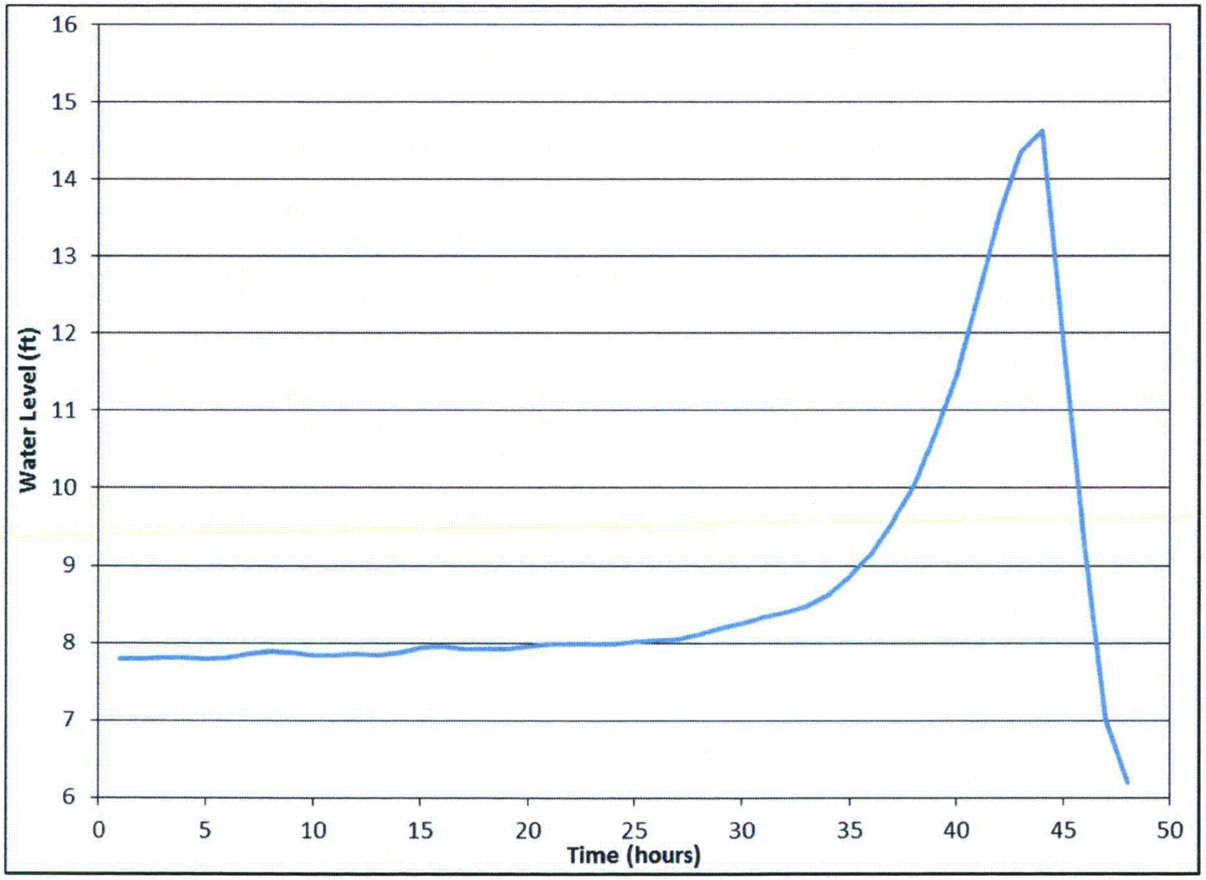
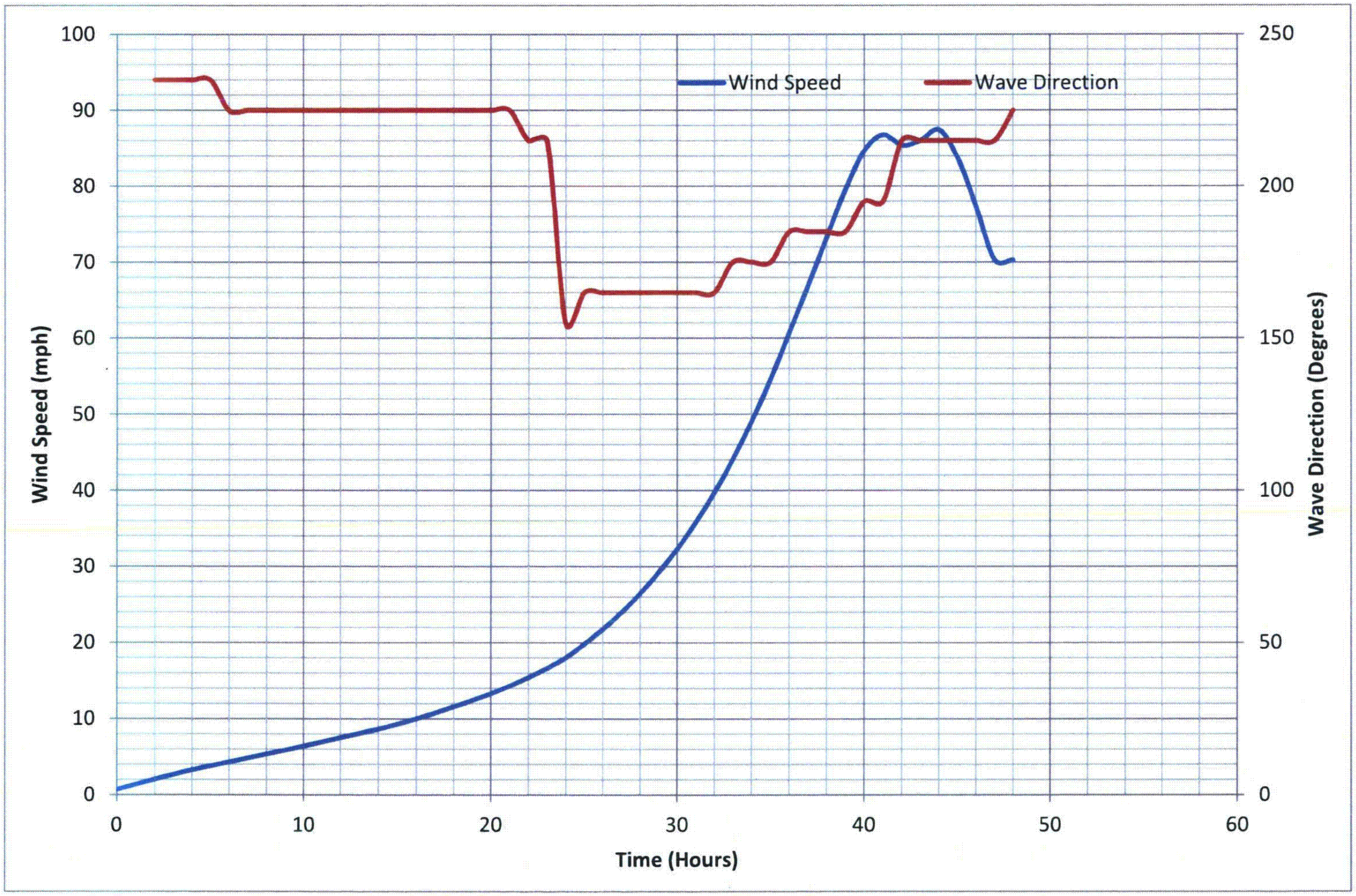


**Figure 3-42: Nearshore/Shallow-Water SWAN Simulation Input Water Level – PMH**



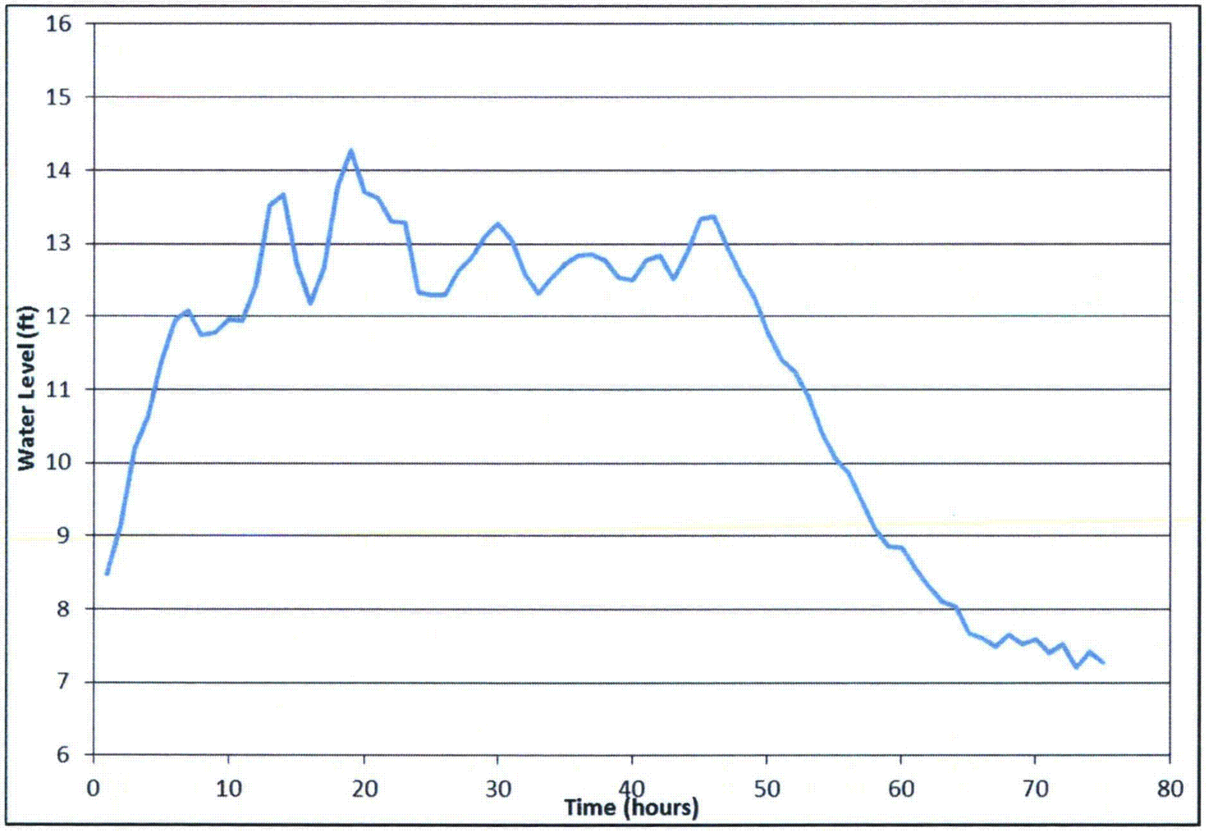

Note: Results from coupled ADCIRC+SWAN simulation at representative output location V2: -70.5722412556 longitude, 41.9465878842 latitude; see Figure 3-39 for location.

**Figure 3-43: Nearshore/Shallow-Water SWAN Input Wind Speed and Wind Direction - PMH**



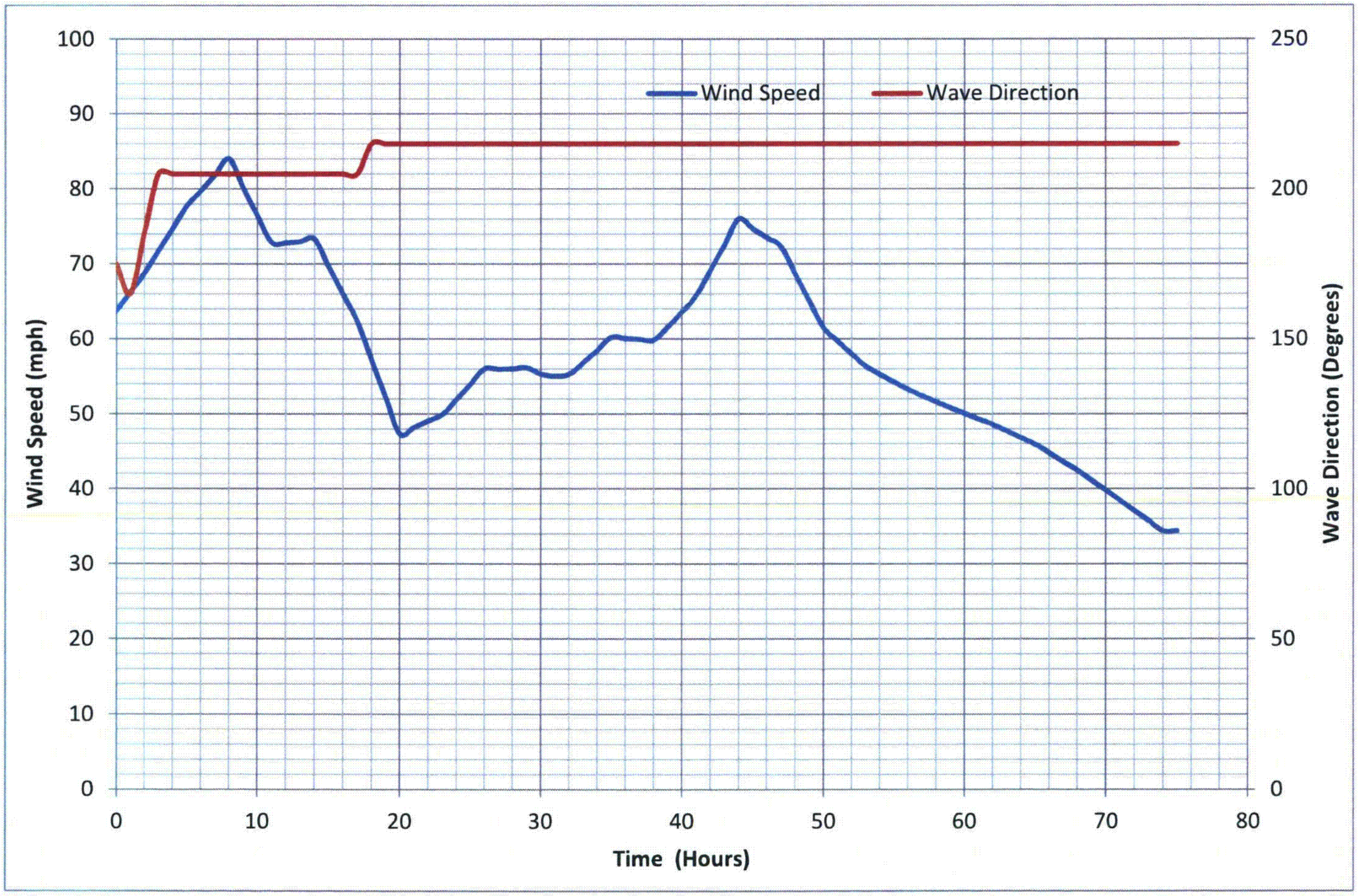

Note: Results from coupled ADCIRC+SWAN simulation at representative output location V2: -70.5722412556 longitude, 41.9465878842 latitude; see Figure 3-39 for location.

**Figure 3-44: Nearshore/Shallow-Water SWAN Simulation Input Water Level – PMWS**




Note: Results from coupled ADCIRC+SWAN simulation at representative output location V2: -70.5722412556 longitude, 41.9465878842 latitude; see Figure 3-40 for location.

**Figure 3-45: Nearshore/Shallow-Water SWAN Input Wind Speed and Wave Direction – PMWS**




Note: Results from coupled ADCIRC+SWAN simulation at representative output location V2: -70.5722412556 longitude, 41.9465878842 latitude; see Figure 3-40 for location.

**Figure 3-46: Peak Significant Wave Height – PMH**

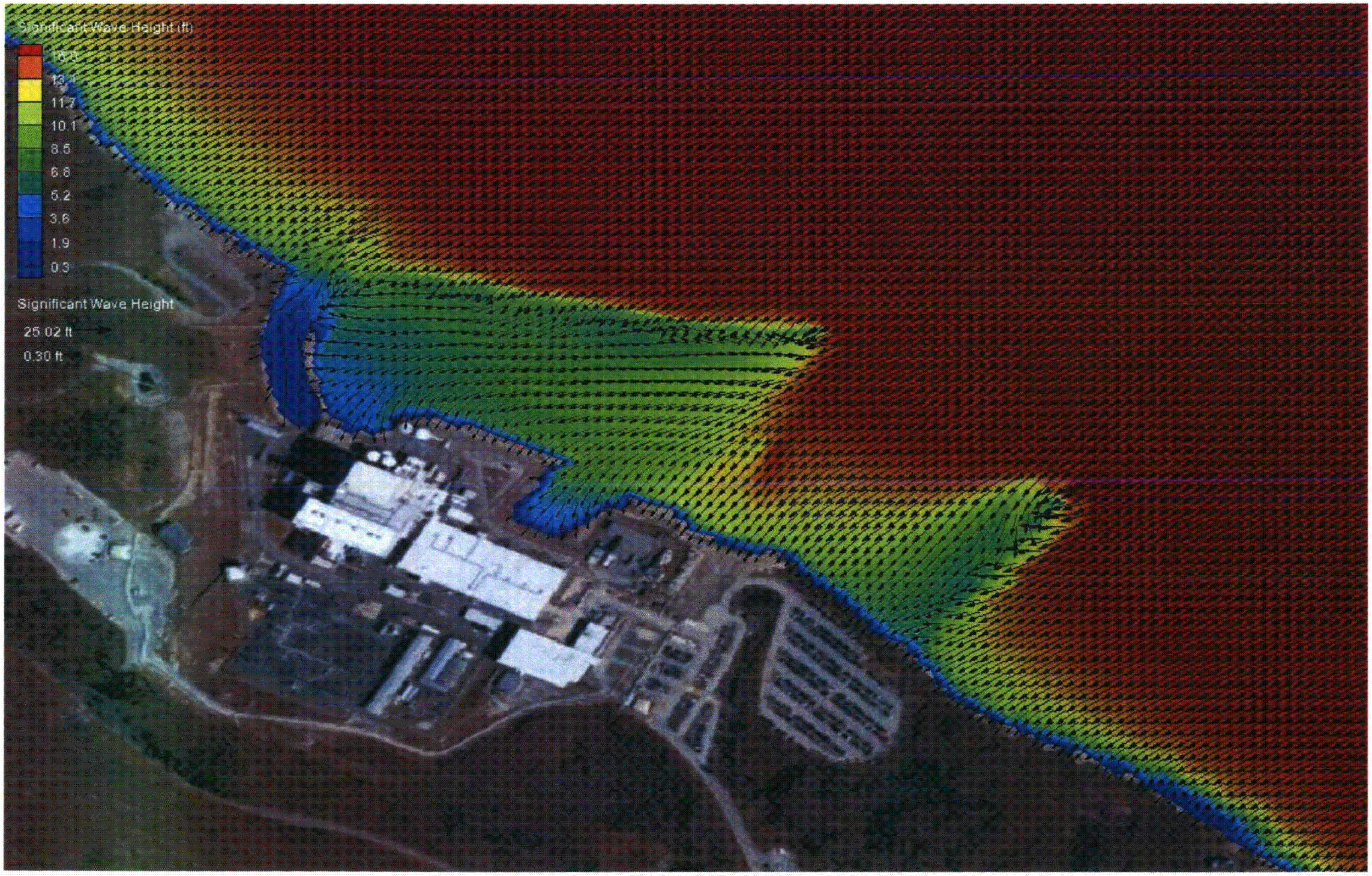


**Figure 3-47: Peak Significant Wave Height – PMWS**

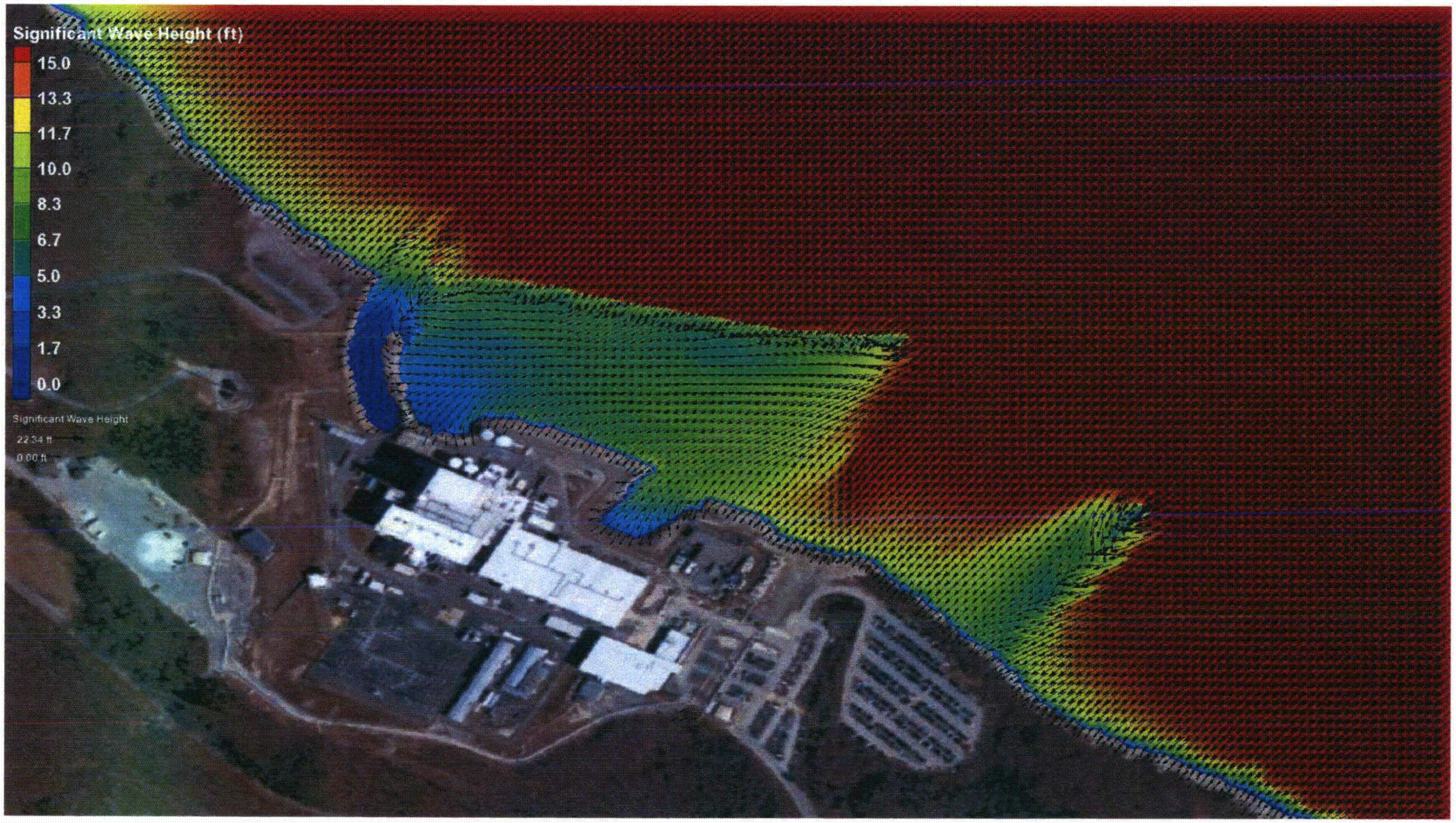


**Figure 3-48: Nearshore/Shallow-Water SWAN Simulation Output Locations – PMH & PMWS**

**Figure 3-49: Transect Locations**

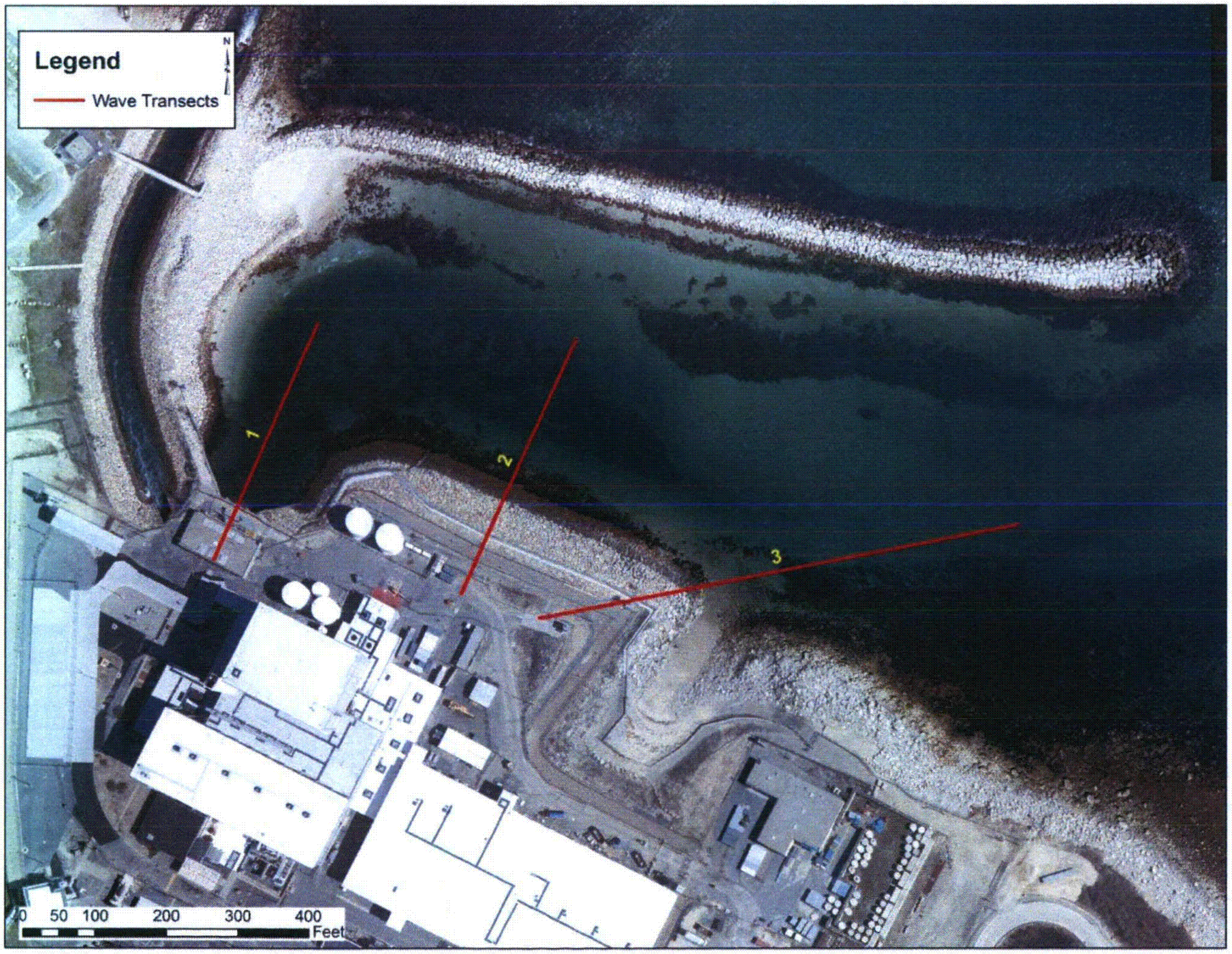


**Figure 3-50: Nearshore/Shallow-Water SWAN Simulation Wave Breaking Zone – PMH**

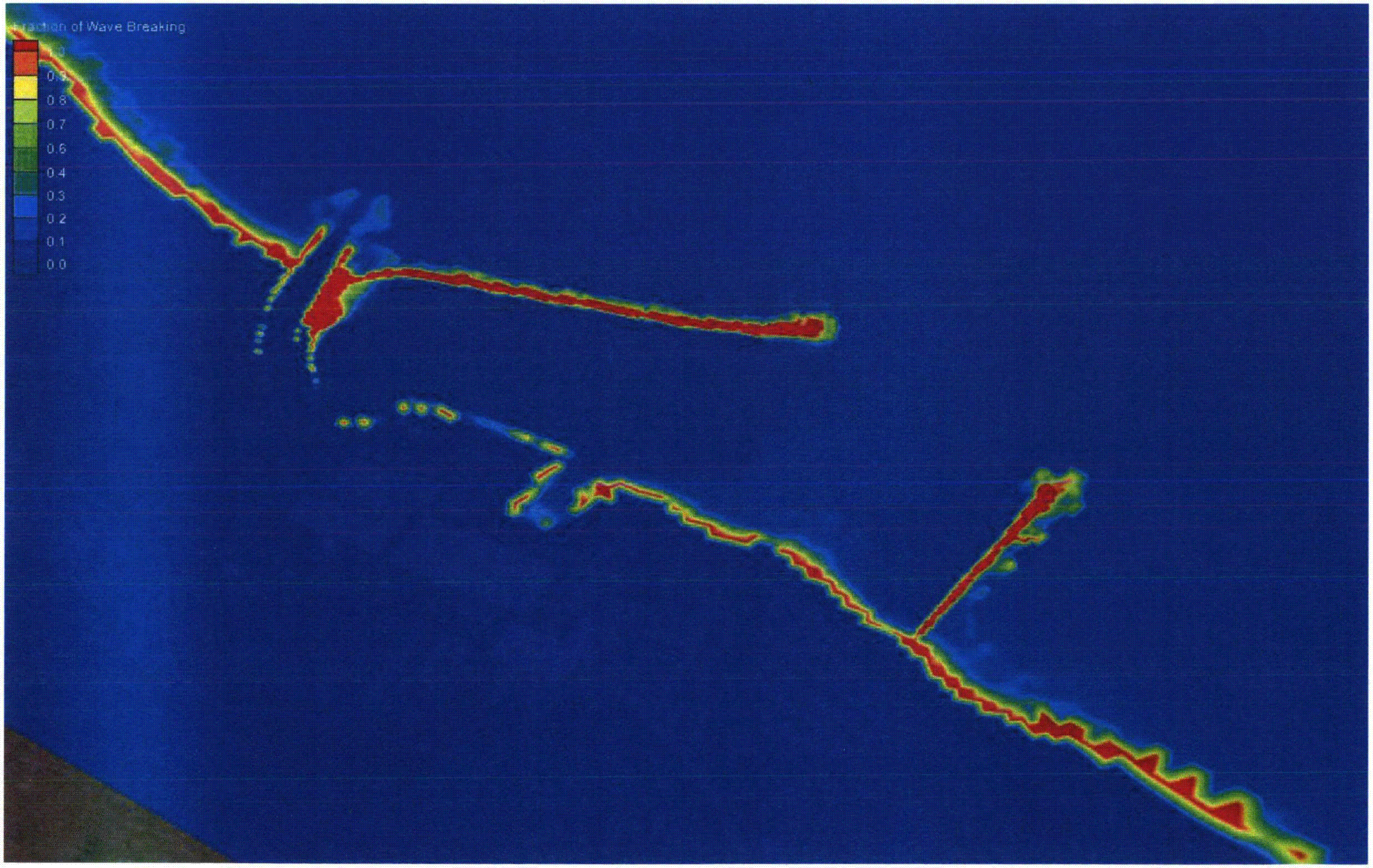
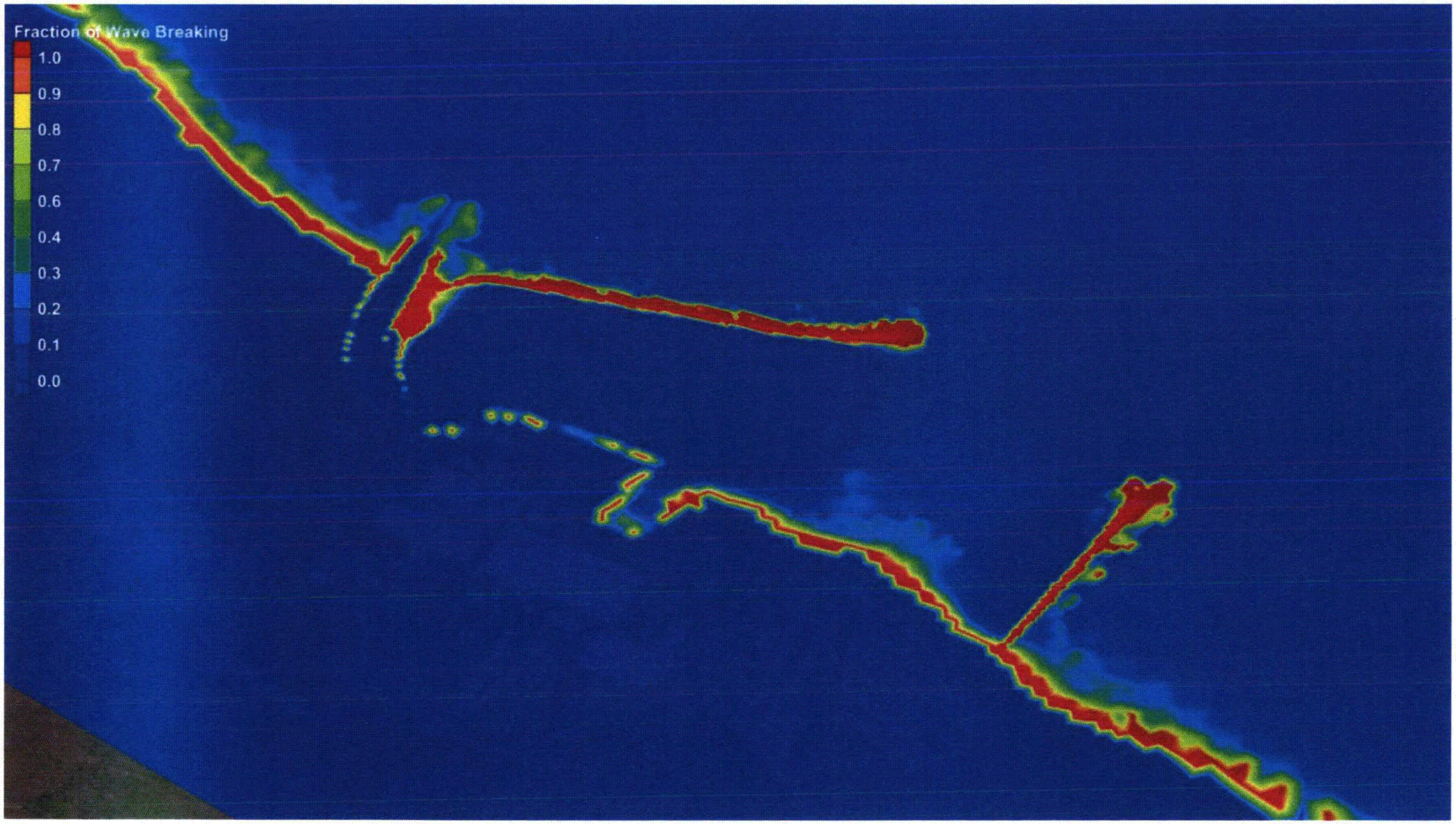


**Figure 3-51: Nearshore/Shallow-Water SWAN Simulation Wave Breaking Zone – PMWS**



## 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 PNPS.

- 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. Still water elevation;
    b. Wind waves and runup 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).

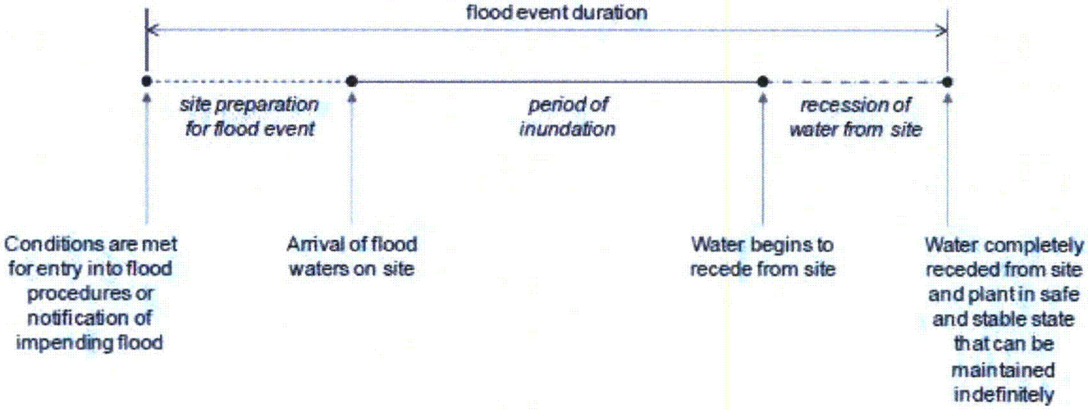



*Illustration of Flood Event Duration* (Figure 6 of JLD-ISG-2012-05 (NRC 2012b))

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 PNPS, 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

PNPS was considered potentially exposed to the flood hazards listed below. In some instances, an individual flood-causing mechanism (e.g., storm surge) was also addressed in the combined effect flood scenario.

- Local Intense Precipitation;

- Probable Maximum Storm Surge due to the Probable Maximum Hurricane or the Probable Maximum Wind Storm, and;
- Combined Effect Flood scenario consisting of the Probable Maximum Storm Surge and wave effects.

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 not to be applicable to the flooding hazard for PNPS.

Flooding due to LIP or the combined effect flood are the only flood mechanisms that could result in inundation in the vicinity of plant SSCs important to safety. Potential 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 not currently addressed in the CLB; however, the PMP event was evaluated as part of the IPEEE. The PMP produces water depths of 24.5 feet MSL at buildings on the south side of the plant and 22.5 feet MSL at buildings on the north side of the plant. It also results in ponding on building roofs of about 0.5 feet.

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 17.1 inches in one hour and 25.5 inches in six hours. The maximum flood depths range from 0.6 feet to locally as high as 2.6 feet above grade near important plant locations, with the highest LIP water surface elevations occurring on the south side of the plant. The maximum LIP flood elevation at an important location examined is 25.2 feet MSL.

Inundation durations at the important 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 important plant locations are summarized in Table 4-3.

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

### 4.1.2 Flooding in Streams and Rivers

The 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 PNPS.

### 4.1.3 Dam Breaches and Failures

The 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 PNPS.

### 4.1.4 Storm Surge

The only flood hazard addressed in the CLB is an extreme storm tide level of 13.5 feet MSL resulting from either the peak storm surge from a nor'easter and an astronomical high tide, or from a maximum hurricane produced storm surge.

Based on the evaluation summarized in Section 3.4, the maximum still water elevation for the PMSS resulting from a PMH was determined to be 15.7 feet MSL, and the maximum water surface elevation, which includes wave setup, was determined to be 15.8 feet MSL. As noted in Section 2.0, station grade is at 20 feet MSL and as noted in Section 3.9, the minimum entry level for all safety related structures is 23 feet MSL.

### 4.1.5 Seiche

The flood hazard due to seiche 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 PNPS.

### 4.1.6 Tsunami

The 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 at PNPS.

### 4.1.7 Ice Induced Flooding

The flood hazard due to ice 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 PNPS.

### 4.1.8 Channel Migration or Diversion

The 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 PNPS.

### 4.1.9 Combined Effect

The flood hazard due to combined effect was not specifically addressed as part of the CLB. However, as noted in Section 2.0, a series of wave action model studies were performed to assist in the design of PNPS waterfront structures.

The combined effect flooding mechanism from the flood hazard re-evaluation is the combination of the PMSS which includes the design antecedent water level, and wave effects which include wave runup. The maximum combined effect flood elevation is 22.1 feet MSL which occurs near the Reactor Building in the yard area between plant buildings and the shore revetment. The maximum combined effect flood elevation at the upstream face of the Intake Structure is 19.8 feet MSL and the Intake Structure will be subject to hydrostatic, hydrodynamic and wave loads due to impact from wind-wave activity during the PMSS. As noted in Section 2.0, station grade is at 20 feet MSL.

A comparison of the re-evaluated combined effect flood hazard to the CLB is provided in Table 4-4.

Impacts 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 | 22.5 feet MSL along north side of plant buildings<br><br>24.5 feet MSL along south side of plant buildings<br><br>Roof ponding of approx. 0.5 feet based on all roof drains being 100% effective.<br><br>[Note: PMP was evaluated as part of the IPEEE.] | 23.3 to 23.5 feet MSL (at important locations on north and west sides of plant)<br><br>25.2 feet MSL (at important locations on south side of plant)<br><br>Not Applicable | +0.8 to +1.0 feet MSL<br><br>+0.7 feet MSL<br><br>Not Applicable |
| PMF in Rivers and Streams | Not Evaluated | Screened | Not Applicable |
| Dam Breaches and Failures | Not Evaluated | Screened | Not Applicable |
| Storm Surge | 13.5 feet MSL | 15.8 feet MSL<br><br>[max. water surface elevation (i.e., still water plus wave setup)]<br><br>[Note: Station grade is at 20 feet MSL.] | +2.3 feet |
| Seiche | Not Evaluated | 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 | 22.1 feet MSL (near Reactor Building in site yard between buildings and shore revetment)<br><br>19.8 feet MSL (upstream face of Intake Structure) [Note: Station grade is at 20 feet MSL.] | Not Applicable |

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. Still Water Elevation | Not identified in the CLB. 24.5 feet MSL per the IPEEE PMP event. | 25.2 feet MSL | NB (see Section 5.0) |
| | Max. Wave Runup Elevation | Still water elevation of 14.7 feet MSL based on wave model studies (see Section 2.2) | 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. | No unreasonably high velocities were indicated; the maximum flow velocity occurs at the drop off between site grade and the discharge channel and is reasonable given the difference between the channel's surface water elevation and ground surface elevation at the head of the channel. No erosion is expected within the power block or discharge channel. | 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 | The CLB indicates that the Reactor, Turbine and Radwaste Buildings have a waterproofing membrane designed to prevent or minimize groundwater in leakage. | 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 | Not identified in the CLB. For the IPEEE PMP event, flooding starts at zero height, increases to PMP levels and then recedes back to zero height for the one hour duration of maximum precipitation. | 0.1 to 1.5 hours at important locations. | NB (see Section 5.0) |
| | Period of Recession | Not identified in the CLB. For the IPEEE PMP event, flooding starts at zero height, increases to PMP levels and then recedes back to zero height for the one hour duration of maximum precipitation. | Over ten hours at some important locations, specifically on the south side of the plant. | 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 | Maximum Flood Elevation (feet, MSL) | Maximum Flood Depth (feet) | Time to Maximum Flood Elevation (hours) |
|---|---|---|---|---|
| 8 | Emergency Diesel Generator Building Door – North Side | 23.5 | 0.6 | 0.1 |
| 9 | Reactor Building Truck Lock Door – West Side | 23.3 | 0.7 | 1.5 |
| 10 | Water Treatment Area Ground Level Door – West Side | 23.3 | 0.8 | 1.5 |
| 11 | Turbine Building Truck Rollup Door – South Side | 25.2 | 2.5 | 1.4 |
| 12 | O&M Building Ground Level Door - South Side | 25.2 | 2.6 | 1.4 |
| 13 | Hatch A – Turbine Building – South Side | 25.2 | 1.1 | 1.4 |
| 14 | Hatch B – Turbine Building – South Side | 25.2 | 1.1 | 1.4 |
| 15 | Air Vent –Redline Building – South Side | 25.2 | 2.2 | 1.4 |

**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. Still Water Elevation | Not identified in the CLB. | 22.1 feet MSL (near Reactor Building in site yard between buildings and shore revetment)<br><br>19.8 feet MSL (upstream face of Intake Structure)<br>[Note: Station grade is at 20 feet MSL.] | NB (see Section 5.0) |
| | Max. Wave Runup Elevation | Still water elevation of 14.7 feet MSL based on wave model studies (see Section 2.2) | 22.1 feet MSL due to runup from a fully head-on wave on the revetment slightly east of the Reactor Building portion of the plant. | NB (see Section 5.0) |
| | Max. Hydrodynamic/Debris Loading | Not identified in the CLB. | Hydrostatic, hydrodynamic and debris impact loads were determined at the 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. | The PMSS includes the design antecedent water level. | NB (see Section 5.0) |
| | Effects on Groundwater | The CLB indicates that the Reactor, Turbine and Radwaste Buildings have a waterproofing membrane designed to prevent or minimize groundwater in leakage. | Not evaluated; effects of storm surge on groundwater elevations are expected to be limited to those areas currently observing tidal influence on groundwater elevations. | 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 | 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. | | | | |

## 4.2 References

**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.

## 5.0 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED

Flooding due to LIP or the combined effect flood are the only flood mechanisms which could cause inundation of the PNPS site in the vicinity of SSCs important to safety.

### 5.1 Impacts of Re-Evaluated Flood Elevations

In response to the re-evaluated flood elevations resulting from the LIP and the combined effect flood which consists of wind-generated waves in conjunction with the PMSS, 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 this evaluation indicate that there are no impacts to equipment important to safety as a result of the re-evaluated flood elevations, as discussed further below.

#### 5.1.1 LIP Affected Locations

Referring to Table 4-1, the re-evaluated LIP maximum flood elevation of 23.5 feet MSL exceeds the CLB/IPEEE flood elevation of 22.5 feet MSL on the north side of the plant at the following three locations: the Emergency Diesel Generator (EDG) Building, the Reactor Building Truck Lock Door and the Water Treatment Area Ground Level Door. Referring to Table 4-3, the maximum flood depths at these doors varies between 0.6 and 0.8 feet. All three doors were included in the PNPS 2012 Fukushima walkdowns. In addition, per a PNPS 1993 Internal Memo, these doors are currently credited not to fail for a flood height of up to 1.5 feet. Therefore, potential flooding due to the LIP at these three doors is not of concern. (PNPS 2015b)

Referring to Table 4-1 and Table 4-3, the re-evaluated LIP maximum flood elevation of 25.2 feet MSL exceeds the CLB/IPEEE flood elevation of 24.5 feet MSL in one area on the south side of the plant at five important locations as follows: the Turbine Building Truck Rollup Door (Door 102), the O&M Building Ground Level Door, Turbine Building Hatch A, Turbine Building Hatch B, and the air vent(s) associated with the Redline Building. Turbine Building Hatches A and B lead into the Retube Building. The Retube building does not contain any operating, energized equipment. Based on a walkdown of the Retube Building, all piping, wiring and conduits were observed to be sealed at penetrations in the building's north wall adjacent to the Condenser Bay. If the Retube Building were to flood from water leakage through Hatches A and B, the Retube Building would hold more than 100,000 gallons of water prior to leakage, and subsequent leakage would be removed by the Turbine Building's sumps. As such, SSCs important to safety would not be adversely impacted by LIP flood levels at Hatches A and B. (PNPS 2015a and PNPS 2015b)

Referring to Figure 5-1, potential water ingress at the Turbine Building Truck Rollup Door (Door 102), the O&M Building Ground Level Door and the air vent(s) associated with the Redline Building is discussed in the following subsections.

##### 5.1.1.1 Turbine Building Truck Rollup Door –South Side of Plant

The Turbine Building Truck Rollup Door (Door 102) is not sealed from external flooding. Although water could potentially penetrate into the Turbine Building at this location, there are no SSCs important to safety within the vicinity of this door. However, the flow of water over the removable oil/water barriers near interior Doors 103, 105 and 311 could result in a flow path to the Lower Switch Gear Room which houses SSCs important to safety. Additionally, referring to Figure 5-2, it is not credible to assume that the leakage of water into the building areas near Door 102, due to the maximum flood depth of 2.5 feet at Door 102, would exceed the oil/water barriers and fail Doors 103, 105 and 311. Considering that the three interior doors will not fail, the building areas adjacent to Door 102 are judged to have adequate volume for protecting elevated switch gear within the Lower Switch Gear Room. (PNPS 2015a)

Per PNPS Procedure 8.C.42, Subcompartment Barrier Matrix, Door 102 has been analyzed for tornado and high energy line break (HELB) loads for its open and closed positions. Door 102 is also normally in its closed position with procedural steps in place to ensure that the door is in its closed position during storm preparations (PNPS 2015a).

While Door 102 is currently credited not to fail for a flood height of up to 1.5 feet, a PNPS 1993 Internal Memo notes that rolling steel doors for large openings, i.e., Door 102, could possibly experience noticeable deflections and approach the upper limit of their structural capacity due to 1.5 feet of water pressure. However, Door 102, which is approximately 20 feet wide by 21 feet high, is designed for a wind load of 25 pounds per square foot (psf) (0.17 pounds per square inch (psi)) over its entire surface. The average load on Door 102 from the LIP water depth of 2.5 feet is 0.325 psi. Considering that the entire door can withstand the stated wind load, it is judged that the door's bottom 2.5 feet can also withstand the higher water load of 0.325 psi since the wind load over the entire door would produce a stress much greater than the stress produced by the water load on a smaller section of the door. Additionally, there is a steel angle along the bottom of Door 102 which provides additional strength to withstand deflection. (PNPS 2015a)

A visual inspection of the interior side of Door 102 was performed. During the walkdown, it was observed that the door panels are bowed toward the inside of the Turbine Building up to a height of approximately five feet from the floor. All door panels are intact and no fractures were observed. The inside track on both sides of the door is intact and bolted to an adjacent angle that spans the height of the door. No damage to the track was observed. Therefore, the slightly bowed door will not fail under the LIP event and it will meet its design requirement. (PNPS 2015a)

If water leaks into the Turbine Building through Door 102, the water would be restrained to the truck lock area and adjacent area as shown on Figure 5-2. As previously noted, there are no SSCs important to safety in these areas. Interior Doors 103 and 311 lead to the Lower Switch Gear Room and interior Door 105 leads to stairs accessing the radwaste corridor. Door 103 is similar to other doors that have previously been tested and evaluated for a loading capacity of 1.48 psi with the swing of the door uniformly applied, or for a head of 6.82 feet without failure. Door 311 is a set of double doors and would have less head capacity than Door 103. However, based on previous testing and evaluations performed for PNPS doors, Door 311 is judged to be able to withstand up to 2.5 feet of head. For Door 105, applying a water load of 2.5 feet in depth against the swing of the door results in 1.92 psi or a uniform head of 8.86 feet prior to failure. Therefore, it is unlikely that interior Doors 103, 105 and 311 would fail; however, if water were to seep through these interior doors, since the switch gear is elevated, equipment important to safety would not be adversely impacted. (PNPS 2015a)

#### 5.1.1.2 O&M Building Ground Level Door – South Side of Plant

Potential water leakage through the O&M Building Ground Level Door would be via a torturous path into the Reactor Building and into the Lower Switch Gear Room. Flood water would need to enter the corridor for the O&M Building, seep through two double doors that lead into the Redline Building and then flow into the Radwaste Building via the failure of one of four other doors. The structural failure of personnel doors constructed of steel due to water acting on door exteriors is not addressed in the PNPS 1993 Internal Memo. However, doors of the same type and of similar configuration to the O&M Building Ground Level Door and those leading into the Radwaste Building from the Redline Building have previously been evaluated for internal flooding. Based on the prior internal flooding evaluation, the O&M Building Ground Level Door and doors between the Radwaste and Redline Buildings will not fail due to the LIP maximum flood depth of 2.6 feet at the O&M Building Ground Level Door, which would result in a uniformly distributed water load on building doors of less than 1.48 psi. (PNPS 2015a)

#### 5.1.1.3 Air Vent(s) – Redline Building – South Side of Plant

Potential water leakage at the air vent(s) – Redline Building location would be via a torturous path into the Reactor Building and into the Lower Switch Gear Room. Flood water would need to enter the Redline Building and flow into the Radwaste Building through the failure of one of four doors and then through two double doors that open between the Redline Building and into an adjacent hallway, which would allow the height of water to decrease. The structural failure of personnel doors constructed of steel due to water acting on door exteriors is not addressed in the PNPS 1993 Internal Memo. However, doors of the same type and of similar configuration to the doors leading into the Radwaste Building from the Redline Building have previously been evaluated for internal flooding. Based on the prior internal flooding evaluation, the Redline and Radwaste Building doors will not fail due to the LIP maximum flood depth of 2.2 feet at the air vent(s) – Redline Building, which would result in a uniformly distributed water load on building doors of less than 1.48 psi. (PNPS 2015a)

### 5.1.2 Combined Effect Flood Affected Locations

Referring to Table 4-1 and Table 4-4, the re-evaluated combined effect flood elevation of 22.1 feet MSL, associated with a maximum water surface elevation (i.e., still water plus wave setup) of 15.8 feet MSL, exceeds the CLB extreme storm tide level elevation of 13.5 feet MSL. As noted in Section 2.0, the wave action model studies performed to assist in the design of PNPS waterfront structures did not subject the Reactor Building to flooding. Therefore, SSCs important to safety that may be adversely impacted by the combined effect flood are situated within the EDG Building and the Intake Structure. However, penetrations, including doors into the EDG Building, are at a minimum elevation of 23 feet MSL; therefore, SSCs important to safety within the EDG Building would not be adversely impacted.

The Intake Structure contains a Class I structure inside a Class II structure. The entrance into the Intake Structure is at an elevation of 21.5 feet MSL. The entrance into the safety related (Class I) portion of the Intake Structure from the non-safety related (Class II) portion is at an elevation of 25.5 feet MSL. The salt service water pumps and their motors comprise the SSCs important to safety within the Intake Structure and are at an elevation of 25.6 feet MSL, situated above grating at an elevation of 25.5 feet MSL. Although water from the combined effect flood at an elevation of 22.1 feet MSL may enter into the Class II portion of the Intake Structure, it would not enter into the Class I portion. Furthermore, although the area below the concrete floor at elevation 21.5 feet MSL is open to the salt service water bay, the concrete floor would protect the salt service water pumps from wave effects. (PNPS 2015a)

As noted in Section 4.0, the maximum combined effect flood elevation at the upstream face of the Intake Structure is 19.8 feet MSL and the Intake Structure will be subject to hydrostatic, hydrodynamic and wave loads due to impact from wind-wave activity during the PMSS. Considering that station grade is at 20 feet MSL, no adverse impact to the Intake Structure is anticipated.

## 5.2 Conclusions

Plant walkdowns have confirmed that inundation associated with the LIP or the combined effect flood will not impact SSCs important to safety. As a result, no interim flood mitigating measures are planned. (PNPS 2015a)

## 5.3 References

**PNPS 2015a**. PNPS Response to AREVA Request for Information RFI #2015-003, Dated February, 2015. (AREVA Document No. 38-9236113-000).

**PNPS 2015b**. PNPS Response to AREVA Request for Information RFI #2015-004, Dated February, 2015. (AREVA Document No. 38-9236474-000)

**Figure 5-1**: LIP Select Locations on South Side of Plant

[Source: PNPS 2015a]

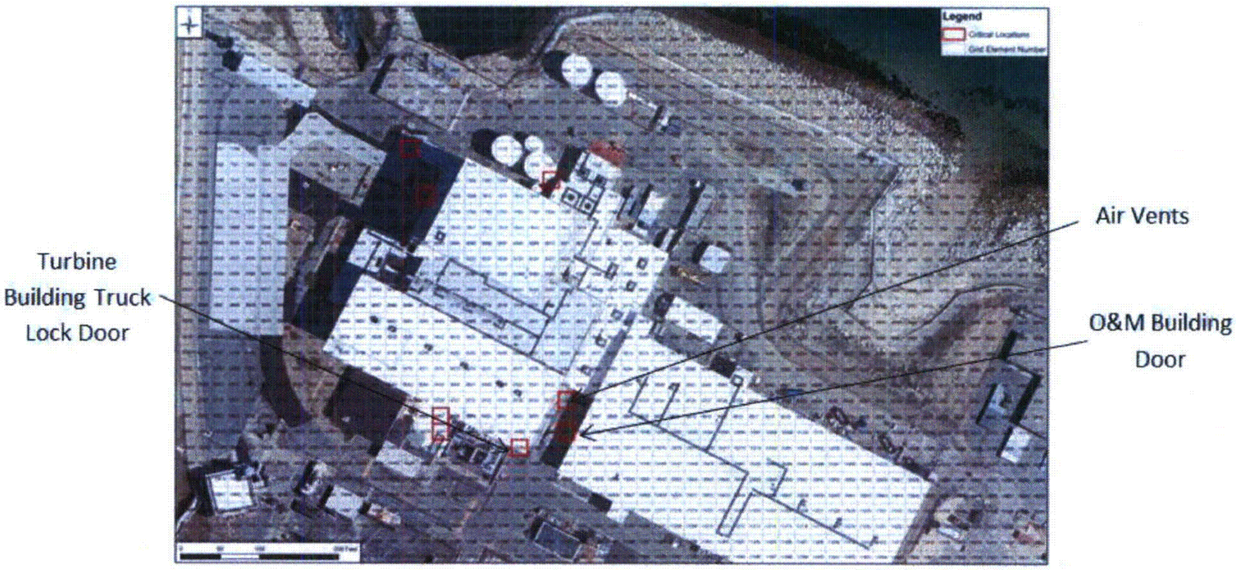
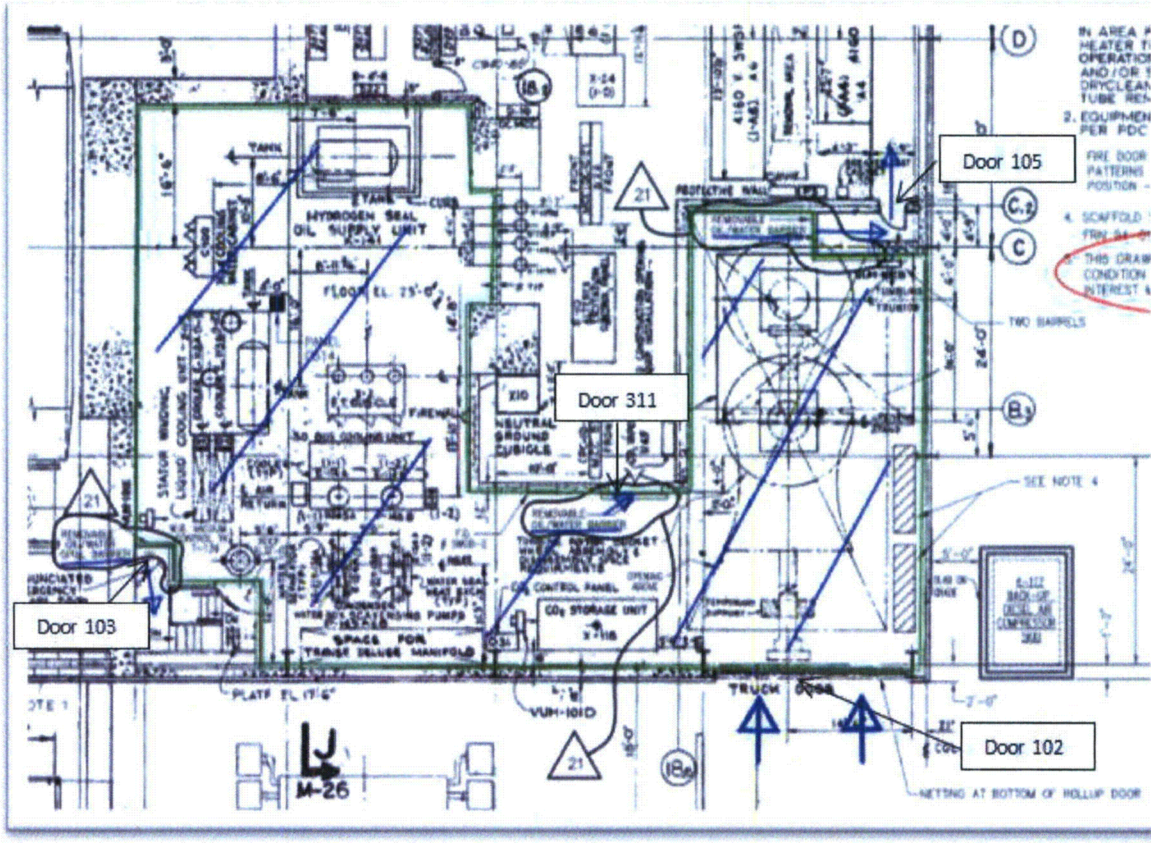


**Figure 5-2: Turbine Building Flow Path**

[Source: PNPS 2015a]




Notes:

1. Should water from the LIP seep through Door 102, potential flooding within Turbine Building areas is shown by the blue highlighting. It is anticipated that flood water would be contained within the walls and doors outlined in green highlighting.
2. Illegible text or features in this figure are not pertinent to the technical purposes of this document.

## 6.0 ADDITIONAL ACTIONS

As noted in Section 5.0, plant walkdowns have confirmed that inundation associated with the LIP or the combined effect flood will not impact SSCs important to safety. Therefore, no additional flood mitigating actions are planned.

# 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\51-9226940-000\official.

This information is also provided electronically with this report as 51-9226940-000_Appendix_A2.zip.

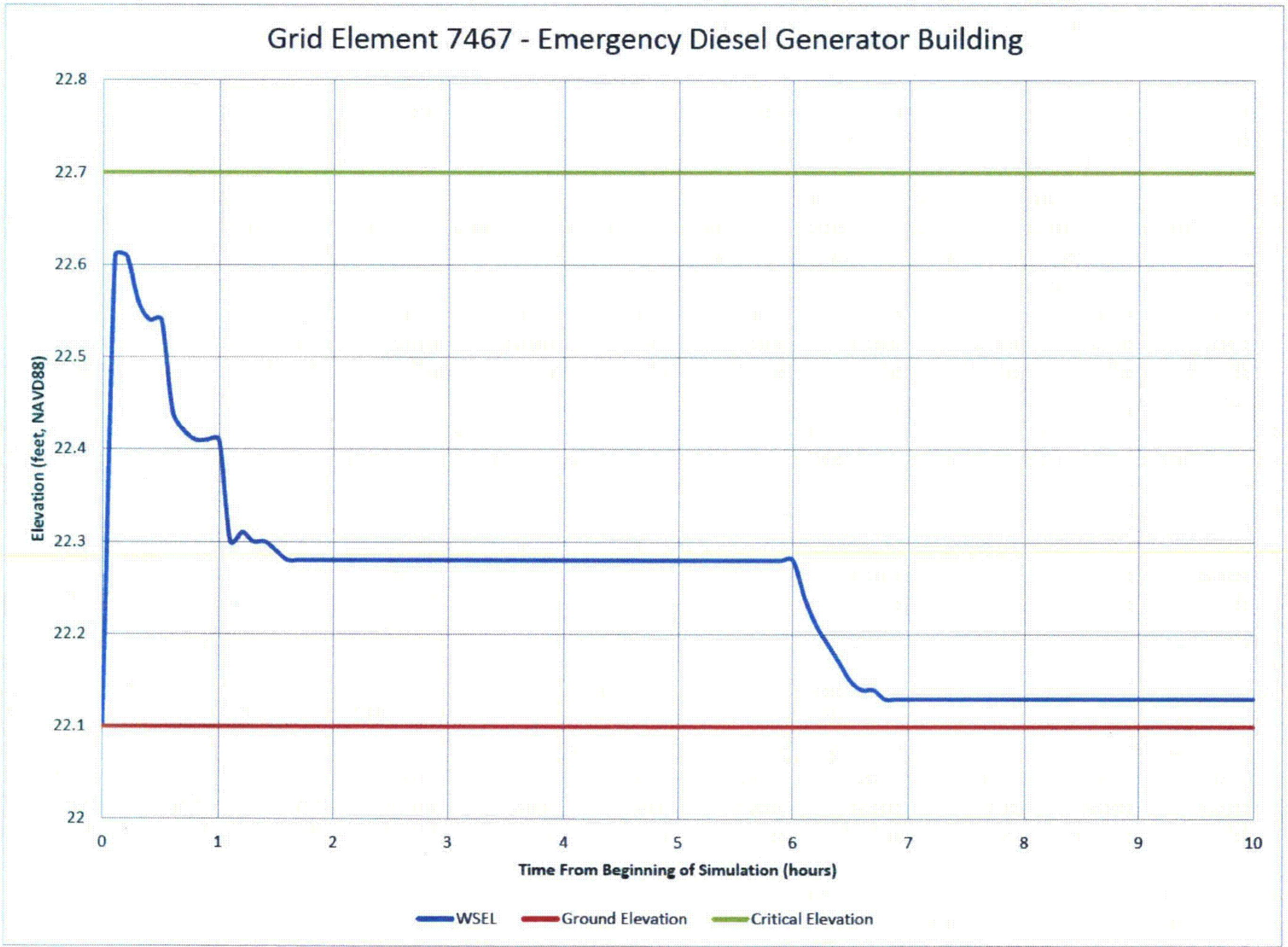
## A.3 LIP Results – Large Format Figures

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\51-9226940-000\official.

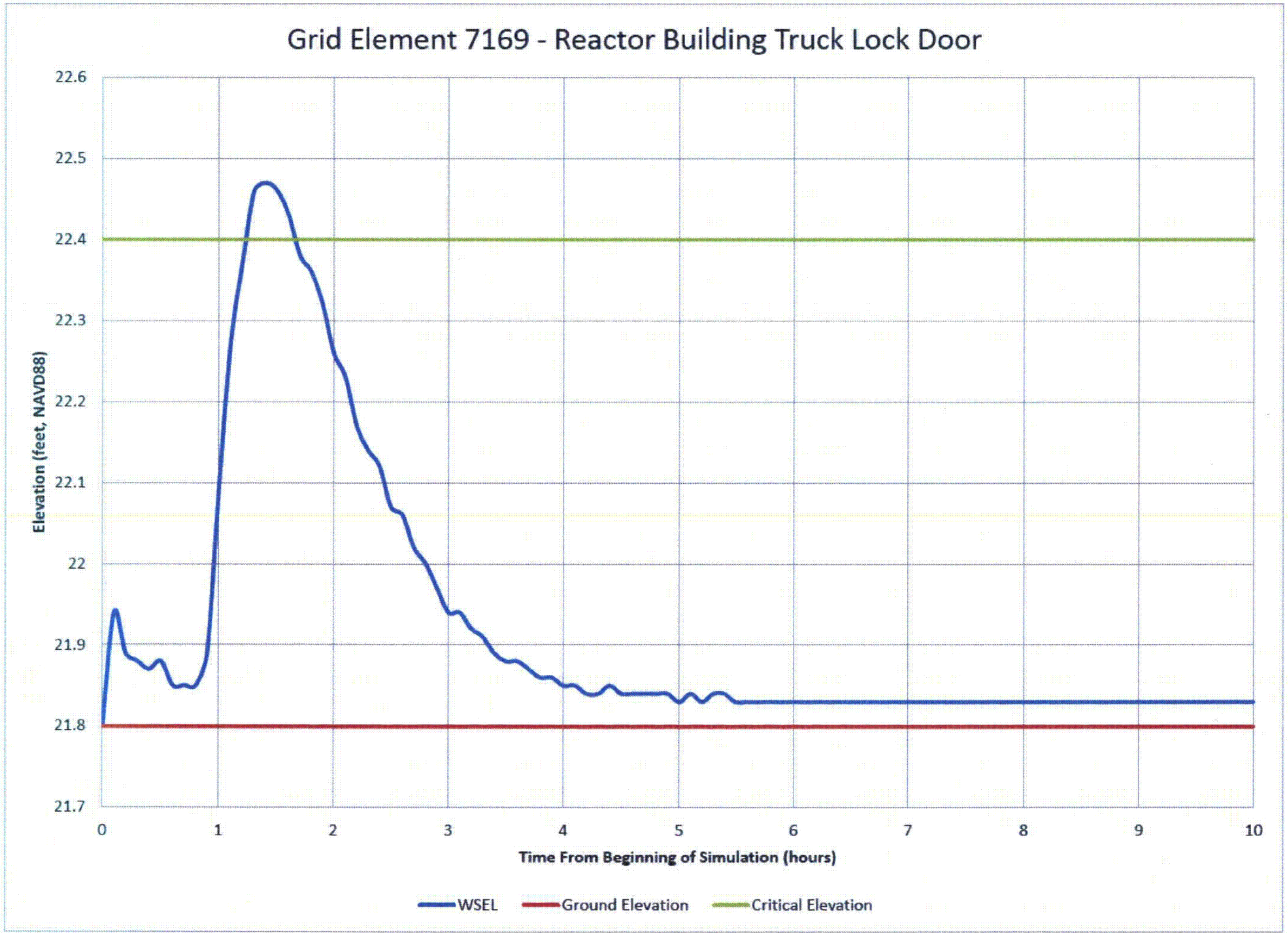
This information is also provided electronically with this report as 51-9226940-000_Appendix_A3.zip.

# LIP FLO-2 CRITICAL GRID ELEMENT TIME SERIES HYDROGRAPHS

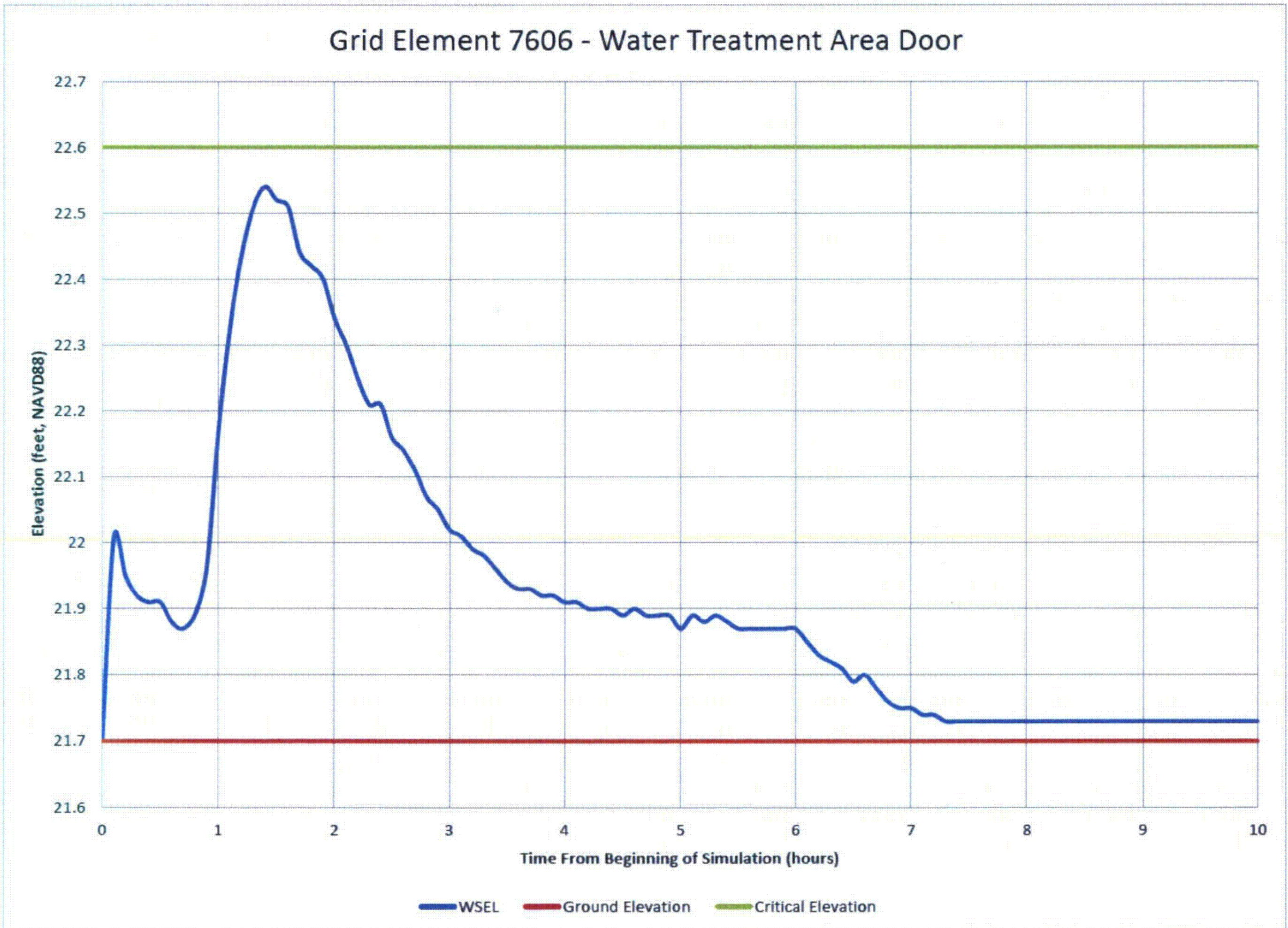
Emergency Diesel Generator Building Door – North Side



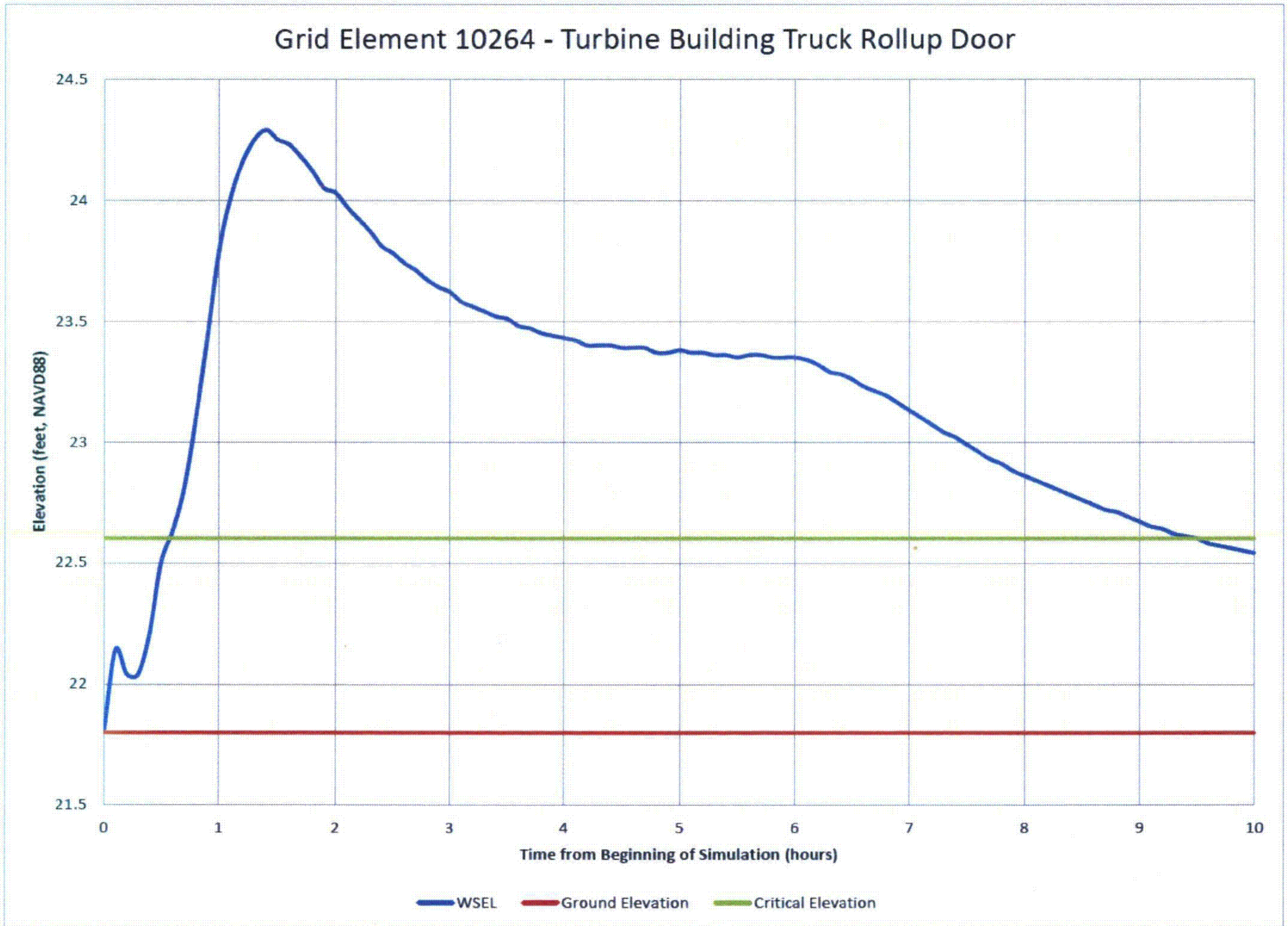
Reactor Building Truck Lock Door – West Side



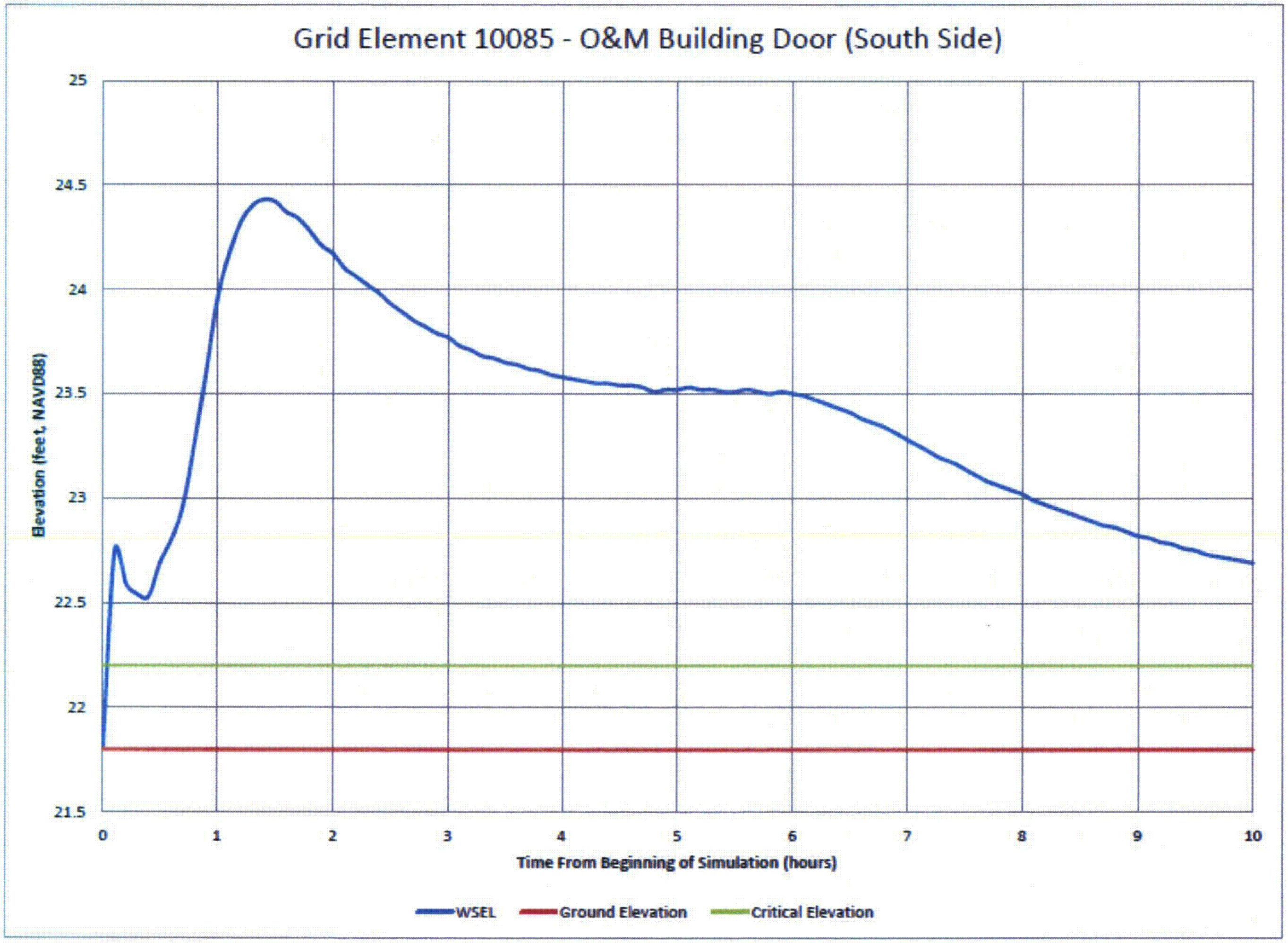
Water Treatment Area Ground Level Door – West Side



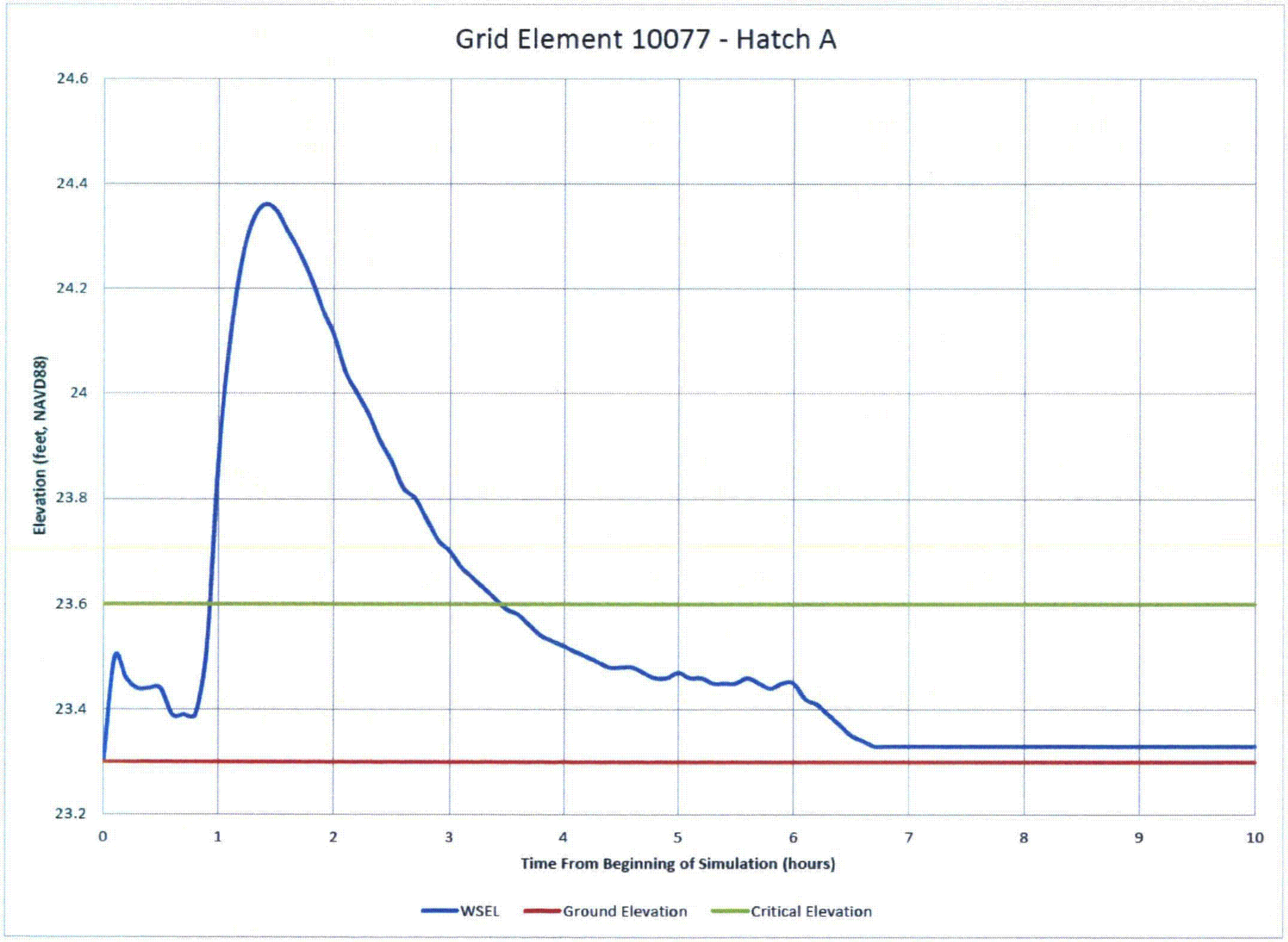
Turbine Building Truck Rollup Door – South Side



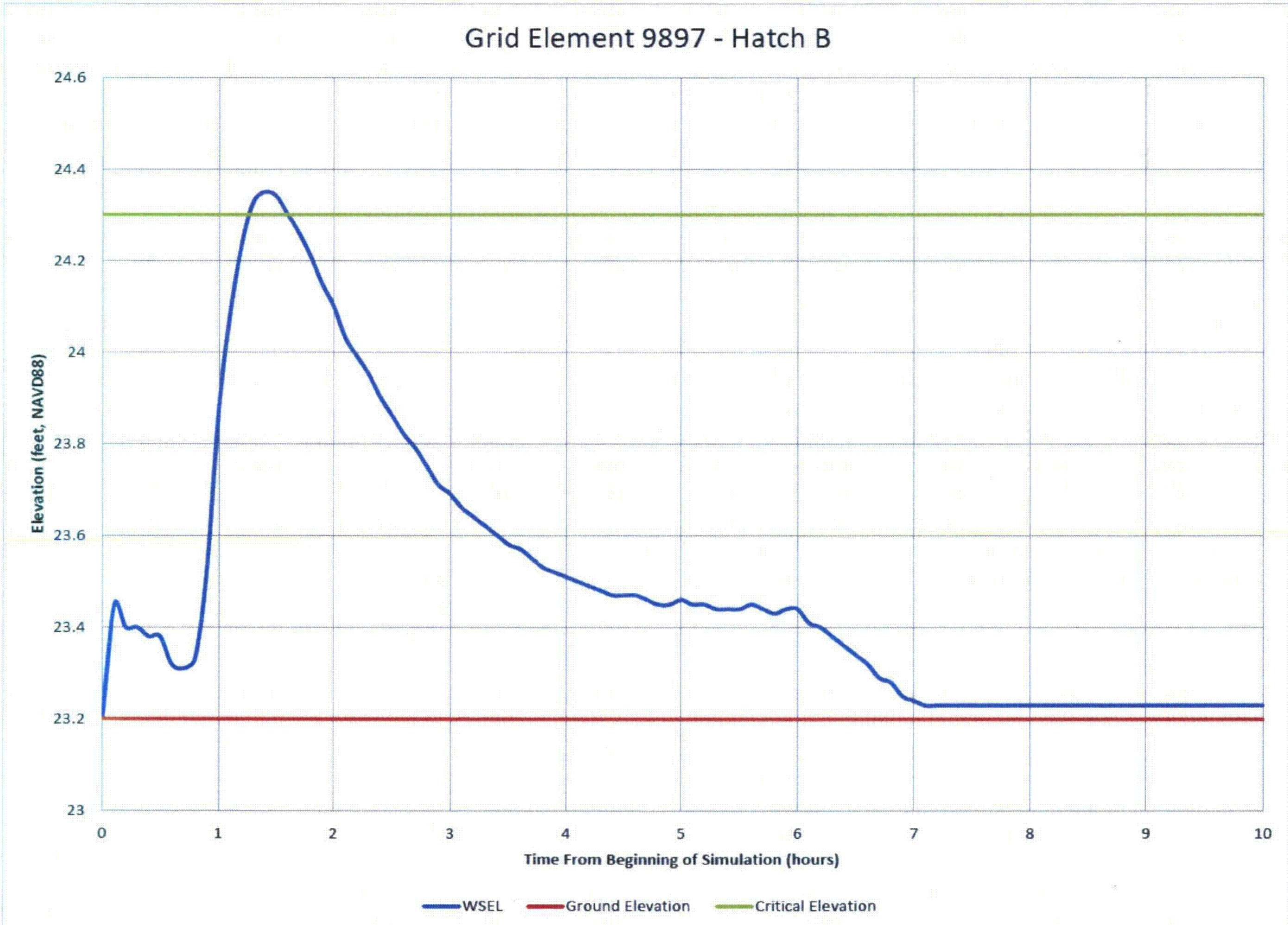
O&M Building Ground Level Door – South Side



Hatch A – Turbine Building – South Side



Hatch B – Turbine Building – South Side



Air Vent – Redline Building – South Side