FLOODING HAZARDS REEVALUATION REPORT
FPL-072-PR-002, Rev. 0

IN RESPONSE TO THE 10 CFR 50.54(f) INFORMATION REQUEST REGARDING NEAR-TERM TASK FORCE RECOMMENDATION 2.1: FLOODING
for the

St. Lucie Nuclear Power Plant
Units 1 & 2 (PSL)

6501 South Ocean Drive
Jensen Beach, FL 34957
Facility Operating License Nos. DPR-67 & NPF-16

NEXTera ENERGY RESOURCES

Presented to:
NextEra Energy Resources
700 Universe Boulevard
Juno Beach, FL 33408

Prepared by:
Enercon Services, Inc.
1501 Ardmore Blvd., Suite 200
Pittsburgh, PA 15221
February 6, 2015

	Printed Name/Title	Affiliation	Signature	Date
Preparer:	Paul J. Martinchich, P.E. / MGR	ENERCON	[signature]	2/6/15
Reviewer:	Kyle M. Kaminski, P.E. / Project Engineer	ENERCON	[signature]	2/6/15
Approver:	Gerald E. Williams, P.E. / MGR	ENERCON	[signature]	2/6/2015
Site Sponsor:	DAN H. WEST / ENG	FPL	[signature]	2/12/15

Table of Contents

List of Tables

List of Figures

1.0 PURPOSE

This report provides the NextEra Energy (NEE) St. Lucie Nuclear Power Plant ("Plant St. Lucie" or PSL) response to the U.S. Nuclear Regulatory Commission's (NRC) March 12, 2012 Request for Information (RFI) pursuant to the post-Fukushima Near-Term Task Force (NTTF) Recommendation 2.1 flooding hazards reevaluation of PSL.

1.1 Background

In response to the Fukushima Dai-ichi nuclear facility accident resulting from the March 11, 2011 Great Tōhoku Earthquake and subsequent tsunami, the NRC established the NTTF to conduct a systematic and methodical review of NRC processes and regulations, and to make recommendations to the NRC for its policy direction. The NTTF reported a set of recommendations that were intended to clarify and strengthen the regulatory framework for protection against natural phenomena.

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

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

In accordance with Enclosure 2 of the NRC 10 CFR 50.54(f) letter request, licensees are required to reevaluate the flooding hazards at their sites against present-day regulatory guidance and methodologies being used for early site permits (ESP) and combined license applications (COLA).

1.2 Requested Actions

Per Enclosure 2 of the NRC 10 CFR 50.54(f) letter request,

> *Addressees are requested to perform a reevaluation of all appropriate external flooding sources, including the effects from local intense precipitation on the site, probable maximum flood (PMF) on stream and rivers, storm surges, seiches, tsunami, and dam failures. It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for ESP and COL reviews including current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The requested information will be gathered in Phase 1 of the NRC staff's two phase process to implement Recommendation 2.1, and will be used to identify potential vulnerabilities.*
>
> *For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard with the hazard evaluation.*
>
> *Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them. The scope of the integrated assessment report will include full power operations and other plant configurations that could be susceptible due to the status of the flood protection features. The scope also includes those features of the ultimate heat*

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

NEE PSL submitted a 90-day response letter (Letter L-2012-241) to the U.S. NRC titled, "FPL/St. Lucie Plant's 90 Day Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding Flooding Aspects of Recommendations 2.1 and 2.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident," dated June 11, 2012 (NEE, 2012). In the letter, NEE PSL stated intentions regarding the RFI.

1.3 Requested Information

This report provides the following requested information for PSL, in accordance with Enclosure 2 of the NRC 10 CFR 50.54(f) letter request:

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

   i. Detailed site information (both designed and as-built), including present-day site layout, elevation of pertinent SSCs important to safety, site topography, as well as pertinent spatial and temporal datasets (Section 2.0).

   ii. Current design basis flood elevations for all flood causing mechanisms (Section 3.0).

   iii. Flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance (Section 2.3).

   iv. Changes to the watershed and local area since license issuance (Section 2.4.6).

   v. Current license basis (CLB) flood protection and pertinent flood mitigation features at the site (Section 3.0).

   vi. Additional site details, as necessary, to assess the flood hazards (i.e., bathymetry, walkdown results, and other pertinent data).

b. Evaluations of the flood hazards for each flood causing mechanism, based on present-day methodologies and regulatory guidance. Analyses are provided for each flood causing mechanism that may impact the site, including local intense precipitation (LIP) and site drainage, flooding in streams and rivers, dam breaches and failures, storm surge and seiche, tsunami, channel migration or diversion, and combined effects. Mechanisms that are not applicable at the site are screened-out, however, and justification is provided. Bases are provided for inputs and assumptions, methodologies and models used including input and output files, and other pertinent data (Section 4.0).

c. Comparison of current and reevaluated flood causing mechanisms at the site. An assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood causing mechanism is provided. This includes how the findings from Enclosure 4 of the 10 CFR 50.54(f) letter (i.e., Recommendation 2.3 Flooding Walkdowns) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, justifications are included (Section 5.0).

d. Interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment, if necessary (Section 6.0).

e. Additional actions beyond Requested Information Item d taken or planned to address flooding hazards, if any (Section 7.0).

1.4 Applicable Guidance Documents

The following documents were used as guidance in performing the flooding hazards reevaluation analyses:

ANS, 1992, American National Standards Institute/American Nuclear Society (ANSI/ANS), "Determining Design Basis Flooding at Power Reactor Sites," ANSI/ANS-2.8-1992, La Grange Park, Illinois, July 28, 1992.

NRC, 2007, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," NUREG-0800 (Formally issued as NUREG-75/087), Washington, D.C., Revision 3, March 2007.

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

NRC, 2011, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," NUREG/CR-7046, Washington, D.C., November 2011.

NRC, 2013a, "Guidance for Performing a Tsunami, Surge and Seiche Flooding Safety Analysis Revision 0," Japan Lessons-Learned Project Directorate Interim Staff Guidance, JLD-ISG-2012-06, January 4, 2013.

NRC, 2013b, "Guidance for Assessment of Flooding Hazards Due to Dam Failure," Japan Lessons-Learned Project Directorate Interim Staff Guidance, JLD-ISG-2013-01, Revision 0, July 29, 2013.

2.0 SITE INFORMATION

The PSL site is located on Hutchinson Island in St. Lucie County, Florida. Hutchinson Island is a 23-mile-long barrier island that lies offshore of the Floridian peninsula between the Ft. Pierce and St. Lucie Inlets. The coordinates for PSL Unit 1 are latitude 27° 20' 58" North, longitude 80° 14' 48" West. PSL Unit 2 is located approximately 300 feet (ft) south of PSL Unit 1 at latitude 27° 20' 55" North, longitude 80° 14' 47" West. The State Plane Florida East coordinates for the midpoint are 1,096,626.3 ft North and 900,780.8 ft East (NEE, 2014c; NEE, 2014e).

The eastern boundary of the site is the Atlantic Ocean and the western boundary is the Indian River, a tidal lagoon (Figure 2-1). Other prominent natural features within 50 miles of the site include Lake Okeechobee (which is located 30 miles to the west-southwest of the site) and a portion of the Everglades (which is located approximately 24 miles to the south of the site [NEE, 2014c; NEE, 2014e]). Pertinent site features described in this report are shown on a site plan as Figure 2-2.

The Unit 1 site information contained herein is from the PSL Unit 1 Updated Final Safety Analysis Report (UFSAR), Amendment 26 (NEE, 2014c; NEE, 2014e), PSL Units 1 and 2 Summary of Individual Plant Examination of External Events (IPEEE) Report (FPL, 1994), the PSL Flooding Walkdown Report FPL060-PR-001, Rev 0 (FPL, 2012), and the PSL Flooding Walkdown Report FPL060-PR-001, Rev 1 (FPL, 2014b).

The Unit 2 site information contained herein is from the PSL Unit 2 UFSAR, Amendment 21 (NEE, 2013b; NEE, 2014c), PSL Units 1 and 2 Summary of IPEEE Report (FPL, 1994), the PSL Flooding Walkdown Report FPL060-PR-001, Rev 0 (FPL, 2012), and the PSL Flooding Walkdown Report FPL060-PR-001, Rev 1 (FPL, 2014b).

2.1 Datums and Projections

Various horizontal and vertical datums and mapping projections are referenced throughout this report. This section describes the horizontal and vertical datums and mapping projections used, their definitions and relationships, and the methods used to convert from one datum or projection to another.

2.1.1 Horizontal Datums and Projections

A horizontal datum is a system which defines an idealized surface of the earth for positional referencing. The North American Datum of 1983 (NAD83) is the official horizontal datum for U.S. surveying and mapping activities. Latitude and longitude are typically used to identify location in spherical units.

A map projection is a mathematical transformation that converts a three-dimensional (spherical or ellipsoid) surface onto a planar surface. Different projections cause different types of distortions and, depending on their intended use, projections are chosen to preserve different relationships of characteristics between features. Projections in the United States are typically defined as State Plane coordinate systems with units of Northing and Easting. The United States is divided into many State Plane maps; large states can be defined by several maps. A site survey was performed in 2013 (Southern Resource Mapping, Inc., 2013; NEE, 2014b). The PSL site survey uses the NAD83 horizontal datum and is projected onto the State Plane Florida East coordinate system.

2.1.2 Vertical Datums

There are two types of vertical datums: tidal and fixed. Fixed datums are reference level surfaces that have a constant elevation over a large geographical area. Tidal datums are standard elevations that are used as references to measure local water levels. The following is a list of tidal and fixed datums, as defined by the National Oceanic and Atmospheric Administration (NOAA) (NOAA, 2013e):

- Mean Sea Level (MSL) – The arithmetic mean of hourly heights observed over the National Tidal Datum Epoch, where the National Tidal Datum Epoch is the specific 19-year period adopted by the National Ocean Service (NOS) as the official time segment over which tide observations are taken and reduced to obtain mean values for tidal datums.

- Mean Low Water (MLW[1]) – The average of all low water heights observed over the National Tidal Datum Epoch.

- North American Vertical Datum of 1988 (NAVD88) – Fixed vertical control datum determined by geodetic leveling, referenced to the tide station and benchmark at Pointe-au-Pere (Father Point), Rimouski, Quebec, Canada.

- National Geodetic Vertical Datum of 1929 (NGVD29) – Fixed vertical control datum, affixed to 21 tide stations in the United States and 5 in Canada.

The CLB and historical PSL survey drawings are typically referenced to "MLW" vertical datum, to which site benchmarks are referred. However, this datum does not coincide with current local tide MLW at nearby tide stations; thus, the vertical datum is referred to as MLW PSL Datum (or PSL Datum as referred herein).

The NRC has expressed a preference for flood level reporting in NAVD88. The updated PSL site survey (Southern Resource Mapping, Inc., 2013) and reevaluation are in NAVD88 datum. Other datums are referenced or used where appropriate.

2.1.3 Vertical Datum Relationships and Conversions

Where required, vertical transformations were performed using the site conversions as shown in Table 2-1.

For reference in this report, there is an offset of -3.35 ft from PSL Datum to NAVD88 (i.e., Elevation (EL) ft-PSL Datum – 3.35 = EL ft-NAVD88) (NEE, 2014b).

Note that these conversions only apply in the vicinity of PSL, and conversions would vary at other locations.

2.2 PSL Plant Description

PSL contains two pressurized water reactor steam generating stations, Unit 1 and Unit 2. The site is comprised of approximately 1,132 acres. The unimproved area of the site is generally flat, covered with water, and has a dense vegetation characteristic of Florida coastal mangrove swamps. At the ocean shore,

[1] PSL site data are referenced to an MLW that is unique to the plant site. The NAVD88 datum is 3.35 ft less than PSL Datum.

the land rises slightly to a dune or ridge. The eastern boundary of the site is the Atlantic Ocean and the western boundary is the Indian River, a tidal lagoon. The plant site is bounded by Herman Bay to the south and Big Mud Creek to the north. Big Mud Creek is not a flowing stream but rather an inlet off of the Indian River. Surface drainage from the site is either to the Atlantic Ocean, Indian River, or to Big Mud Creek and hence to the Indian River.

The principal buildings and structures that comprise PSL are the reactor building, turbine building, control building, radwaste building, administration building, machine shop, offgas retention building, intake structure, pump house, cooling towers, training center, low level radwaste processing and storage facility, offgas stack, plant support center, and independent spent fuel storage installation (ISFSI).

The plant island is defined as the area where all safety-related structures are located. The grade elevation for the plant island is EL +18.5 ft-PSL Datum (EL +15.15 ft-NAVD88). The minimum entrance elevation to all safety-related buildings is EL +19.5 ft-PSL Datum (EL +16.15 ft-NAVD88). The crown elevation of roadways on the plant site is EL +19.0 ft-PSL Datum (EL +15.65 ft-NAVD88).

2.3 Flood-Related and Flood Protection Changes to the Licensing Basis since License Issuance

Since the issuance of the license, no revisions to the flood hazards analysis have occurred and no significant changes to the flood protection strategies described in the current UFSAR have occurred: minor changes to waterproofing materials and stoplog installation details have been incorporated.

2.3.1 Hurricane Readiness Procedure

The PSL Hurricane Season Preparation and Severe Weather Preparations are outlined in PSL Administrative Procedure ADM-04.01, Revision 27 (NEE, 2013a), and Administrative Procedure 0005753, Revision 74 (NEE, 2014a), respectively. These procedures provide definitive direction and measures for the protection of the PSL site during the hurricane season (June through November) and for the preparation for severe weather, tornado, or response to a hurricane watch/warning.

2.3.2 Flood Protection Features and Protected Equipment

Seismic Category I SSCs are protected from the effects of high water levels and wave runup through:

- Design to withstand such effects
- Positioning to preclude inoperability
- Housing within waterproof structures

The plant grade around the structures is approximate EL +18.5 ft-PSL Datum. Structures are flood protected up to EL +19.5 ft-PSL Datum. Any exterior doors and penetrations below EL +19.5 ft-PSL Datum which lead to areas containing safety-related equipment are watertight. Flood protection features include:

- Reinforced concrete flood walls have been provided around Unit 1 SSCs (Diesel Oil Tank and CCW Structures) to EL +22 ft-PSL Datum.

- The site drainage system is designed to preclude flooding of safety-related structures and components under probable maximum hurricane (PMH) conditions; however, total flooding of the drain lines will not cause water to back up into areas which would jeopardize the required function of a safety-related system.

- Since there are no CLB-credited structures on the south side of the Unit 2 reactor auxiliary building (RAB), wave runup protection is provided by installing stoplogs in the entrance on the south wall and the southernmost entrance on the east wall prior to a hurricane event.
- Electrical conduits penetrating safety-related structures are constructed with seals to prevent flood water from entering connecting structures. Electrical conduits are sealed at the first opening at the exterior of the structure (i.e., manhole or pipe opening).
- The elevations of essential equipment in structures other than the reactor building and RAB are above the flood protection level of EL +19.5 ft-PSL Datum.
- All piping penetrations or electrical ducts are either encased in concrete where they penetrate the exterior wall or, when sleeves are used, enclosed in a pipe boot designed to prevent seepage. Each seal type has a specific design pressure rating.
- All permanent door openings in the exterior walls of the RAB, fuel handling building, and diesel generator building are provided with either roll-up or swing-type doors for protection from rain, wind, and other atmospheric effects. Doors are furnished with continuous, adjustable rubber stripping at jambs, head, and floor to provide a positive weather-tight closure.

2.3.3 Common Site Protection and Mitigation Features

All essential equipment on the intake structures is placed at EL +22 ft-PSL Datum or higher. The design basis flood level of EL +17.2 ft-PSL Datum is well below the minimum of EL +19.5 ft-PSL Datum for Category I building openings. Additional flood protection is afforded by virtue of the layout of the roads, buildings, and tornado missile protective structures permanently incorporated into the plant design. The elevation of the crown of the perimeter plant road along the east face of the plant island is EL +19.0 ft-PSL Datum. At EL +19.0 ft-PSL Datum, roads have the highest contours of plant island grading features. In the CLB, the structures along the immediate east face of the plant island are credited with forming an effective concrete barrier with respect to inhibiting wave runup. All of the barrier structures are Seismic Category I and have been designed to withstand hurricane and tornado wind loadings.

In areas where drain lines carry storm water from both units, the lines are sized to accommodate the additional flow. The site drainage system and building drainage systems are designed to preclude flooding of safety-related structures under PMH conditions except in the component cooling water (CCW) structures where components are located above the wave runup elevation.

The discharge canal nose area is protected by a steel sheet-piling barrier with its top at EL +22 ft-PSL Datum. Overtopping of the barrier is expected and the resultant water behind the barrier will be drained off into the discharge canals. The temporary flooding around the nose is of no concern since there is no Category I structure located in that part of the plant site. Additionally, intake and discharge canal side slopes are hardened with articulated fabric-formed concrete matting, although these erosion protection linings are not directly credited in the CLB.

The UHS barrier wall is a reinforced concrete buttressed retaining wall which extends across the UHS canal connecting Big Mud Creek (a finger of Indian River lagoon) to the intake canal. Its function is to separate the waters of Big Mud Creek from the intake canal during normal operation and, through valved openings, provide an alternative source of cooling water in the unlikely event that the ocean intake becomes unavailable. In the unlikely event of blockage of the intake canal or pipes, emergency cooling water is taken from Big Mud Creek through the emergency cooling water canal. This emergency source of water is designed to withstand the design basis earthquake, tornado, and PMH conditions.

Flooding of electrical manholes through backup within the site drainage system is prevented by the installation of check valves on the ends of the electrical manhole drainage system headers within the site drainage system catch basins. When the bottom elevation of an electrical manhole sump is below the catch basin elevation, a manhole sump pump is provided. If flooding of an electrical manhole were to occur through inoperability of a check valve, in-leakage through seal openings along the circumference of the manhole cover, or through the manhole cover ventilation holes, the flood water is prevented from entering connecting structures because the construction openings within those structures are filled with concrete and the conduits that route into safety-related structures are constructed with waterstops.

Although not directly credited as a safety-related flood mitigation feature, the vehicle barrier system (VBS) around the entire circumference of the site may help mitigate the effects of wave action, help break waves, and prevent wave-related erosion.

2.3.4 Unit 1 Protection and Mitigation Features

The reactor building and RAB are the only Seismic Category I structures with basements. These structures are completely waterproofed to EL +17 ft-PSL Datum with Nob-Lok waterproofing. All construction joints are waterstopped with 6-inch polyvinyl chloride. The exterior concrete walls of the reactor building below EL +22 ft-PSL Datum and RAB below EL +19.5 ft-PSL Datum are waterproofed. All external building penetrations are waterproofed and/or flood protected to preclude the failure of a safety-related system or component due to external flooding. All penetrations for pipes or electrical ducts are either encased in concrete where they penetrate the wall or, when sleeves are used, enclosed in a pipe boot designed to prevent seepage. The end result is a completely waterproofed structure below grade. Boots are not used below the normal groundwater table.

All permanent door openings in the exterior walls of the RAB, fuel handling building, and diesel generator building are provided with either roll-up or swing-type doors for protection from rain, wind, and other atmospheric effects. Doors are furnished with continuous, adjustable rubber stripping at jambs, head, and floor to provide a positive weather-tight closure.

All interconnections between safety-related structures that could be subjected to flooding are waterproofed. Additional flood protection beyond what is provided by the elevations of the openings of the safety-related structures is not required to protect any of the safety-related structures from wave runup or wind-driven rain, even during a PMH. Therefore, the use of gasketed aluminum stoplogs and/or sandbags and plastic sheeting is not required. All buildings, with the exception of the turbine building, are of the enclosed building type. The turbine building will be subjected to wind-driven water spray; consequently, all equipment inside this building is designed for outdoor service.

The two diesel oil storage tank foundations have a top elevation of EL +22.2 ft-PSL Datum and are surrounded by a 1-ft thick reinforced concrete retaining wall extending to EL +24.5 ft-PSL Datum. The operating floor of the Unit 1 diesel generator building is at EL +22.67 ft-PSL Datum. The Unit 1 diesel tank essentially has a concrete "half-wall" surrounding it which provides protection up to EL +24.5 ft-PSL Datum.

The CCW heat exchanger and pump area is surrounded by a 2-ft thick reinforced concrete wall extending to EL +23.5 ft-PSL Datum. The base elevation of principal equipment is EL +24 ft-PSL Datum.

The Unit 1 RAB north side openings are protected by the fuel handling building, reactor building, and steam trestle. The west side is protected by the turbine building. The east side is protected by the effective barrier formed by the structures immediately east of the RAB. The south side of the Unit 1 RAB has two openings near the west end of the structure. The additional margin of safety is provided by the length of the high fill area to the east and south of these openings. Approximately 1,200 ft of fill at EL +15 ft-PSL Datum extends to the east; about 1,700 ft of fill varying from EL +15 ft-PSL Datum to EL +10 ft-PSL Datum extends to the south.

2.3.5 Unit 2 Protection and Mitigation Features

The list below designates each Seismic Category I structure with identification of exterior or access openings:

Structure	Exterior or Access Openings
Shield Building	No openings below EL +22 ft-PSL Datum
RAB	Minimum entrance at EL +19.5 ft-PSL Datum
Fuel Handling Building	Minimum entrance at EL +19.5 ft-PSL Datum
Diesel Generator Building	Floor and equipment above EL +22 ft-PSL Datum and no openings below EL +22.67 ft-PSL Datum
Diesel Oil Storage Tank Building	No openings below EL +29.5 ft-PSL Datum
Condensate Storage Tank	No openings below EL +22.0 ft-PSL Datum
CCW Building[2]	Equipment is located above EL +23.66 ft-PSL Datum
Intake Structure	Motors located above EL +22 ft-PSL Datum

The reactor building and RAB are the only Seismic Category I structures with basements that contain safety-related SSCs. The reactor building and RAB are constructed with waterproofing to EL +17.0 ft-PSL Datum and therefore protected from in-leakage from phenomena such as cracks in exterior walls. The remaining Seismic Category I structures are founded above the groundwater table and therefore waterproofing is not required. Potential in-leakage within these structures through concrete during PMH flooding is collected by floor drainage systems and routed to sumps for removal. Construction joints within Seismic Category I structures, except the CCW building, contain polyvinyl chloride waterstops up to EL +17.0 ft-PSL Datum.

The CCW structure is not designed as a waterproof structure since all equipment is located above EL +23.66 ft-PSL Datum on pedestals. The CCW structure has a basement, but all SSCs are above grade and elevated as described. Penetrations for pipes or electrical ducts are either encased in concrete where they penetrate the wall or, where sleeves are used, enclosed in a pipe boot designed to prevent seepage. Boots are not used below the normal groundwater table.

[2] Intake cooling water (ICW) debris discharge Valves HCV-21-7A and HCV-21-7B are located below 23.66 ft. However, the actuator and electrical components for these valves are located above 19 ft, which is above the maximum flood level of 17.2 ft and the maximum CCW building backflood level of 18.5 ft.

The Unit 2 diesel oil tank is enclosed in a concrete structure that has been designed for seismic loading and missile impacts.

Wave runup protection is provided to the entrances of the fuel handling building and RAB by the presence of adjacent buildings and structures. Since no permanent structures were located on the south side of the RAB, additional wave runup protection has been provided by installing stoplogs in the entrance on the south wall and the southernmost entrance on the east wall. Rectangular aluminum stoplogs would be stacked to EL +22.0 ft-PSL Datum and secured with bolts. Gaskets provide a seal at both the bottom and sides of the protected openings. The stoplogs are stored onsite in a manner that reserves their readiness for use. When a hurricane watch is posted for the plant, the stoplogs are removed from storage and prepared for installation, with actual installation occurring when the "hurricane warning" is posted for the plant.

2.3.6 Intake Channel Erosion Protection Upgrade

Hurricanes Frances and Jeanne caused significant damage on Hutchinson Island, including effects at PSL. PSL experienced significant debris or sediment accumulation from Hurricanes Frances and Jeanne, which occurred three weeks apart. Because of the surplus storage capacity at the bottom of the intake channel, the cooling water system was not affected. Also, much of the accumulation was attributed to erosion and sloughing of the intake channel's embankments. Subsequently, the intake channel was armored to prevent the recurrence of embankment erosion. The Intake and Discharge Canal Restoration Project (PCM 08047) was issued as Revision 0 on April 23, 2008; construction started shortly after and appears to have completed in 2010.

2.3.7 Flooding Walkdowns

NEE submitted a Flooding Walkdown Report (FPL060-PR-001, Rev. 0), dated November 26, 2012, in response to the 50.54(f) information request regarding NTTF Recommendation 2.3: Flooding for PSL (FPL, 2012). The walkdowns were performed in accordance with Nuclear Energy Institute (NEI) 12-07 (Rev. 0), "Guidelines for Performing Verification Walkdowns of Plant Flood Protection Features," dated May 2012 (NEI, 2012), and endorsed by NRC on May 31, 2012 (NRC, 2012b).

An additional walkdown was performed in 2014, and NEE submitted a revised Flooding Walkdown Report (FPL060-PR-001, Rev. 1) dated September 26, 2014 (FPL, 2014b).

2.3.8 Flooding Walkdown Summary

Configuration and procedures were compared to the flood protection features credited in the CLB documents for external flooding events. Site-specific features credited for protection and mitigation against external flooding events were identified and evaluated. One operability issue was identified and reported to the NRC. This issue has been brought into full compliance in accordance with the guidance provided in Regulatory Issues Summary (RIS) 2005-20 (NRC, 2005). Minor deficiencies were identified and entered in the PSL Corrective Action Program (CAP).

2.4 Hydrosphere

2.4.1 General Climate

The prevailing climatology of PSL is dominated by the presence of the Azores-Bermuda high pressure system resulting in a subtropical marine-type climate for the eastern Florida coast. This climate is featured by a long, warm summer with abundant rainfall followed by a mild, relatively dry winter. The high frequency of onshore winds and the proximity of the warm waters of the Gulf Stream result in warm, humid conditions during most of the year. Temperatures in excess of 90°F typically occur on about 45 days each year, but summer heat is tempered by sea breezes along the coast and by frequent afternoon or early evening thundershowers in all areas. During the winter months, the area is occasionally subjected to an outbreak of cold continental air; however, the cold air mass usually moderates rapidly. Consequently, subfreezing temperatures rarely occur in the area.

2.4.2 Rainfall

Rainfall is unevenly distributed during the year. In general, the heaviest rainfall occurs during the period of June through October, coincident with the hurricane and thunderstorm season. A distinct dry period exists from November through March.

2.4.3 Severe Weather

Severe weather is characterized by thunderstorms, hurricanes, and tornadoes. Thunderstorms have been recorded during each month of the year. However, more than 80 percent occur during the period from May through September; July and August experience the maximum number of thunderstorm days, with 16 days during an average month. On an annual average basis, there are 79 days during which thunderstorms are observed.

During the period of 1899 to 1980, the Florida peninsula has been affected by 96 tropical cyclones. Of these, 39 were classified as hurricanes, 41 as tropical storms, and 16 as tropical depressions. Roughly half the storms in each category passed close enough to PSL to affect it with strong winds and/or heavy rainfall. Hurricanes have occurred most frequently in September and October in the site area.

Historically, tornadoes and waterspouts have been observed during all seasons in southeastern Florida; the greatest frequency of such events occurs during spring and summer. Two independent studies have been made to determine the severity of Florida tornadoes. Both conclude that the severe 360 mile per hour (mph) (Region I) tornadoes are not applicable to Florida, and that historical data do not substantiate speeds exceeding about 200 mph in Florida tornadoes.

2.4.4 Hutchinson Island

PSL is located on Hutchinson Island, a narrow barrier island along the east coast of south-central Florida. The plant is about 9 miles south of Ft. Pierce Inlet at the northern end of the island and about 13 miles north of St. Lucie Inlet at the southern end of the island. The width of Hutchinson Island varies irregularly from less than 1,000 ft to about 6,000 ft. In the immediate site vicinity, it is about 5,500 ft wide. The island is generally quite flat, with the natural high ground being about EL +15 ft-PSL Datum.

The eastern side of Hutchinson Island is bounded by the Atlantic Ocean. The Indian River, to the west of the island, separates the island from the mainland. The Indian River is not a flowing stream, but rather a long, fairly shallow tidal lagoon which parallels a portion of the eastern coast of Florida and separates the mainland from a series of barrier islands, one of which is Hutchinson Island. The Indian River is part of the intracoastal waterway. The east coast of Hutchinson Island is relatively smooth and regular. Fronting the Atlantic Ocean is a typical east coast grassed dune. This is generally continuous and ranges in elevation from about 8 to 20 ft above PSL Datum. State Route A1A has been constructed along most of the island to the west of the dune. West of this highway, Hutchinson Island is generally flat, swampy, and densely vegetated. The vegetation is typical of Florida coastal mangrove swamps. The western coast of

Hutchinson Island is very irregular and is typified by points, bays, and inlets. Large portions of the island have been diked to maintain minimal water levels (six to eight inches) as a mosquito control measure.

2.4.5 Drainage

The two primary drainage features in the area are the St. Lucie River (North Fork in St. Lucie County, South Fork in Marin County) and the Kissimmee River. Drainage from the mainland to the Indian River consists primarily of small streams and drainage canals, the largest of which are Sebastian Creek, the St. Lucie River, and a canal system in the Vero Beach vicinity. Because of its numerous connections with the Atlantic Ocean, the water levels in the Indian River is not affected by flow from the mainland. The Indian River nearshore drainage and circulation along Hutchinson Island are a combination of drift, wind-driven currents, and oscillatory flow due to astronomical tide interaction with the Atlantic Ocean. Water levels in the Indian River are essentially controlled by the levels in the Atlantic Ocean. The nearest tidal inflows and outflows occur at Ft. Pierce Inlet and St. Lucie Inlet, respectively, north and south of the site.

2.4.6 Changes to the Watershed and Local Area since License Issuance

Because the site is located on a barrier island, there are no areas beyond the site property that contribute rainwater run-on to the site. Thus, the contributing watershed comprises only site areas. As such, there are no changes to the watershed, besides the site itself, and there are no changes to adjacent local areas (e.g., local development) that influence site run-on/runoff conditions.

3.0 CURRENT LICENSE BASIS FOR FLOODING HAZARDS

The following describes the flood causing mechanisms and their associated water surface elevations (WSELs) and effects that were considered for the PSL CLB.

Unless otherwise referenced, any current PSL license basis information provided in this section was obtained from the UFSAR, Unit 1, Amendment 26 (NEE, 2014c; NEE, 2014e), and UFSAR, Unit 2, Amendment 21 (NEE, 2013b; NEE, 2014e).

3.1 CLB – Local Intense Precipitation

The probable maximum precipitation (PMP) is 32 inches and would occur over a 10-square mile area during a six-hour period. CLB local intense precipitation PMP for PSL is presented in Table 3-1.

The roof leaders of buildings housing safety-related equipment at PSL are designed for a rainfall intensity of six inches per hour. The UFSAR states that short periods of more intense rainfall would result in water running off the edges of roofs with no adverse effects to safety-related equipment and no water buildup on the roofs in excess of the 1'-6" high parapet.

The roofs of buildings housing safety-related equipment at PSL handle runoff in the following manner:

- Shield Building – Dome roof with parapet. Drainage is by three exterior leaders from parapet to storm water drainage system.
- RAB – Sloping roofs to area drains. Drainage is by various area leaders to storm water drainage system.
- Fuel Handling Building – Roof slopes from west to east to a gutter and exterior leaders to the storm water drainage system.
- Diesel Generator Building – Peaked roof slopes to north or south where water runs off the edges and eventually into catch basins at plant grade.
- Diesel Oil Storage Tank Building – Roof sloped to roof drain. Drainage is by leader to the storm water drainage system.
- CCW Building – Open to the atmosphere at the top of the structure.
- Condensate Storage Tank Building – Open to the atmosphere at the top of the structure.

3.2 CLB – Riverine (Rivers and Streams) Flooding

No significant streams flow by or near the site. The Indian River is a tidal lagoon and Big Mud Creek is an arm of the Indian River Lagoon. Water levels are affected by the Atlantic Ocean and by wind conditions, not by stream flow; therefore, a PMF runoff analysis was not performed.

3.3 CLB – Dam Breaches and Failure Flooding

A detailed dam breach flooding analysis was not performed because there are no upstream or downstream dams that would pose a flooding potential to PSL.

3.4 CLB – Storm Surge

For both units, the design basis flood level during a PMH event is EL +17.2 ft-PSL Datum. External flooding sources other than a PMH event (PMF, LIP, and tsunami) were considered in the UFSAR, but the PMH was determined to provide the greatest flooding threat.

3.4.1 CLB – Probable Maximum Hurricane

Selection of the parameters defining the size, intensity, and forward speed of the PMH was based on data from Memorandum HUR 7-97 (H.M.S. Weather Bureau, 1968) as follows:

- Central Pressure Index (P_O): The minimum barometric pressure in the eye of the hurricane is 26.28 inches of mercury (in. Hg).

- Peripheral Pressure (P_n): The surface pressure at the outer boundary of the hurricane, where the circulation ends, is 31.28 in. Hg.

- Radius of Maximum Winds (RMW):
  - RS (small radius) = 5 nautical miles (nmi)
  - RM (medium radius) = 11 nmi
  - RL (large radius) = 20 nmi.

- Translation Speed (T):
  - Stalled Hurricane = 2 knots
  - ST (slow translation speed) = 4 knots
  - MT (medium translation speed) = 11 knots
  - HT (fast translation speed) = 18 knots.

- Maximum Wind Speed (V_x):
  - V_x for RS is 157 mph
  - V_x for RM is 156 mph
  - V_x for RL is 155 mph.

To generate the highest storm surge at the site, the critical PMH path was selected so that the maximum isovel would approach the site oriented in a direction normal to the bathymetric contours. Using the above meteorological parameters, nine PMH wind fields were generated and analyzed. The method for generating the wind field is described in Memorandum HUR 7-97 (H.M.S. Weather Bureau, 1968).

3.4.2 CLB – 10 Percent Exceedance High Tide and Initial Rise

The peak of the PMH surge was assumed to coincide with the 10 percent exceedance high tide. This is defined by ANSI/ANS-2.8-1992 (ANS, 1992) as the high tide level equaled or exceeded by 10 percent of the maximum monthly tides over a continuous 21-year period. The records of the tide gage at Ft. Pierce were not considered suitable for open coast tide prediction because they are influenced by the inlet. The closest tide records on the open coast were taken at Vero Beach, 22 miles north of the site. The mean tide range observed at this location (3.4 ft) indicates that the Vero Beach data could be conservatively applied to the site, where the mean range for the one-year monitoring program was 3.28 ft. Information for the prediction of 21 years of tide heights was provided by the Marine Predictions Branch of the National Ocean Survey in Rockville, Maryland. This included the results of harmonic tide analysis for Vero Beach based on data from the 365-day period from August 1, 1972 to July 31, 1973. A tide prediction program, which uses the harmonic constants as input, was also provided. This program, called NTP4, was used without modification to predict tide heights for the 21-year period from 1972 to 1992. The resulting 10 percent exceedance high tide, as defined above, was EL +4.6 ft-PSL Datum. A typical sequence of high tides, including a 4.6 ft spring tide, was selected for use with stalled hurricane surge hydrographs with a sequence of highs 12.42 hours apart. Additionally, an initial rise of 1.5 ft was estimated for the Florida coast by the Coastal Engineering Research Center.

3.4.3 CLB – Storm Surge Model

Calculation of the surge height was accomplished using a computerized bathystrophic storm surge model that extended out to the edge of the continental shelf approximately 18 nmi offshore, with a bottom friction coefficient estimated to be 0.0025. The wind stress coefficient was estimated from the following equation:

$$1.1 \times 10^{-6} + 2.5 \times 10^{-6}\left[1 - \frac{W_c^2}{W}\right] \qquad \text{(Equation 3.1)}$$

where:
W_c = *a critical wind speed taken as 14 knots*
W = *the wind speed*

The calculated surge height, or still water level (SWL), included the wind setup, the water level rise due to barometric pressure drop, the astronomical tide, and the forerunner or initial rise. The surge was computed along the path of the maximum wind as the storm moved onshore. In determining the maximum surge at the open coast at the site, the path of maximum wind was brought onshore along a track normal to the general orientation of the bathymetric contours.

A total of 18 surge hydrographs were computed: thirteen were for the steady-state (constant wind field) PMHs, four for the stalled PMHs, and one for the historical Hurricane Flora (1963).

Of the 13 steady-state PMHs, three have T = 2 knots for each of RS, RM, and RL, and one has ST = 4 knots for RL at low astronomical tide. These four steady-state PMHs were used entirely for the erosion study because of their slow translation speed. In fact, the three PMHs with T = 2 knots are also listed as stalled PMH Cases 1, 2, and 3. The initial nine cases with the most critical case (RM, HT) for surge height are listed in Table 3-2. Another case involves a PMH with T = 4 knots for RL, which was investigated for both low and high astronomical tides. Lastly, three storms involve very slow-moving (T = 2 knots) PMHs for each of RS, RM, and RL. Because T = 2 knots is a very slow translation speed, these three cases are also included as stalled PMHs. The stalled PMH was for the purpose of the erosion study (ST = 4 knots for RL for both cases of high tide and low tide). As a case of practical importance, the historical looping Hurricane Flora (1963) has been simulated for the site region to determine the erosional impact. The PMH with RM (11 nmi) and HT (18 knots) yields the maximum surge level (still water) of EL +17.2 ft-PSL Datum.

The resulting surge levels of stalled PMHs were lower than those of the steady-state PMHs. However, the durations of surge level above a certain elevation are longer for the stalled PMHs.

Since the western boundary of the plant site is the Indian River, the surge analysis was extended to include surge inflow into the river. The open coast surge will move into the river initially through the St. Lucie and Ft. Pierce Inlets and eventually by eroding and overtopping Hutchinson Island. For the latter condition, the 23-mile stretch of Hutchinson Island is divided into five broad-crested weirs of various lengths and crest elevations. The effect of antecedent runoff into the river from LIP has been included in the estimate by allowing an increase up to a total of 1.5 ft in river stage before the hurricane landfall. The resulting maximum surge level in the Indian River was also 17.2 ft-PSL Datum.

3.5 CLB – Seiche

Seiche was not considered in the CLB because of the open, shallow characteristics of the ocean and the Indian River.

3.6 CLB – Tsunami Flooding

When the FSAR was prepared, no historically recorded or observed tsunamis had occurred along the Florida coastline and were not examined. Since no seismic or geological evidence was found to indicate the existence of potential tsunami generators offshore in the site area, any possible tsunami effect at the site location was postulated to generate from far-field sources such as those off the east coast of Canada and the Caribbean Sea. The magnitude of such tsunami effects at the site is believed to be negligible compared to the effect of surges caused by the PMH. Consequently, no evaluation of potential tsunami flooding was performed. Tsunami flooding was not considered in the CLB.

3.7 CLB – Ice-Induced Flooding

Prior to PSL construction, a comparison of the Hutchinson Island and West Palm Beach temperatures showed that the temperature data for the two locations are quite comparable. The long-term temperature statistics for West Palm Beach can be reasonably applied to Hutchinson Island. The long-term temperature statistics at West Palm Beach indicate that the minimum extreme of 29°F occurred in 1970. According to information supplied by the U.S. National Climatic Data Center (NCDC), subfreezing temperatures were recorded for a three-day period (January 18, 19, and 20, 1977) at Ft. Pierce, Florida, which is located approximately eight miles north of the PSL site. The lowest temperatures for January 18, 19, and 20, 1977 are 26°F, 28°F, and 24°F, respectively. The rare occurrence of subfreezing temperatures and the short duration of the freezing period in the general area of the site did not result in postulating any possible ice damage to any of the intake structures. Therefore, no preventive ice control measures are considered in the CLB.

3.8 CLB – Channel Migration or Diversion

The intake canal receives water directly from the Atlantic Ocean through subaqueous intake water pipes which run under the beach and end at the start of the canal east of State Route A1A. In the unlikely event of blockage of the intake canal or pipes, emergency cooling water is obtained from Big Mud Creek through the emergency cooling water canal. This emergency water source is designed to withstand the design basis earthquake, tornado, and PMH conditions. Big Mud Creek and the connecting Indian River are saltwater estuaries of the Atlantic Ocean; therefore, no channel migration or diversion evaluations are documented in the CLB.

3.9 CLB – Wind-Generated Waves

The characteristics of waves generated by the PMH in deep water were calculated by the method of Bretschneider. These characteristics are generally represented by the significant wave height and period and the maximum wave height as described in the U.S. Army Corps of Engineers (USACE) Shore Protection Manual of 1975. As the waves travel shoreward over the continental shelf, they are reduced in height by bottom friction and affected by continued action of the hurricane winds and shoaling. The straightness of nearshore contours indicates that the shoreline in the vicinity of the plant site is not an area of wave energy convergence; therefore, refraction was not considered.

The shallow water breaking waves in the vicinity of the plant island are of major concern because the deep water waves generated in the Atlantic Ocean will break before they reach the plant site. Consequently, the resulting wave runup and shore erosion were estimated based on the breaking waves in the shallow water region around the plant site. The significant waves break at depths from 24 to 31 ft at distances between 450 and 900 ft offshore during the times of peak surges. The breaking depths of the maximum waves range from 48 to 110 ft which occur over a wide range of offshore distances between 2,200 ft and 9.5 nmi. Waves smaller than the significant waves may continue propagating towards the plant island in shallower water. The maximum breaking wave height around the plant island was limited by the depth of water corresponding to the PMH surge.

Along the east face, the frontal sand dunes were conservatively assumed to be eroded to the stable base plain elevation of the mangrove area. With the peak surge levels and the base plain elevation, the breaking wave height is estimated by multiplying 0.78 by the depth of water. Since the steady-state PMH (R=11 nmi, T=18 knots) gives the highest surge water level, this PMH was used to estimate the wave runup. A base plain elevation of EL +4 ft-PSL Datum is generally used for estimating wave runups at various locations around the plant island except that EL +0.0 ft-PSL Datum was used for assessing wave condition in the discharge canal. The steady-state PMH (R=20 nmi, T=4 knots for both high and low tide) and the stalled PMH storms were investigated for erosion impact using a realistic base plain elevation of EL +4 ft-PSL Datum for Hutchinson Island.

Along the north face, the base plain elevation was assumed to be EL +5 ft-PSL Datum. The duration of wave action on this face was limited by the shifting of wind directions (from northerly to easterly) as the PMH passed over the site. The wind directions as a function of time were generated by the bathystrophic model used in the surge analysis. The wave action was effective only when the wind direction was between northwest and northeast.

To obtain the probable maximum water levels around the plant island, seven transects were taken to accommodate a time varying PMH wind field approaching from the east, north, and northwest. Runups along these transects are estimated based on a composite slope procedure described in the Shore Protection Manual. These slopes are assumed to be smooth and impermeable for added conservatism. For the steady-state PMH, the maximum surge level was estimated to be EL +17.2 ft-PSL Datum. Considering the effects of wave runup for the maximum postulated surge level of EL +17.2 ft-PSL Datum, the maximum water elevation is EL +18.8 ft-PSL Datum except for waves from the east over eroded areas (dunes and mangroves), which propagate up the discharge canal approaching the nose where Unit 1 and Unit 2 canals join, where a maximum water elevation (surge level and runup) of EL +28.0 ft-PSL Datum was postulated. The discharge canal nose area is protected by a barrier with its top at EL +22 ft-PSL Datum. During the peak surge water level of EL +17.2 ft-PSL Datum, the refracted wave will break on the slope in front of the sheet piling and result in a wave runup of about 11 ft on a hypothetical extension of the slope of the canal nose. Overtopping of the barrier is expected and the resultant water behind the barrier will be drained off into the discharge canals. The temporary flooding around the nose is of no concern because there are no Category I structures located in this part of the plant island. For this surge and wave runup analysis, it was assumed that the fore-dunes were completely washed away along the entire east coast of the site and that the incident wave propagates from the ocean without any attenuation prior to reaching State Route A1A. Therefore, the presence of mangroves or the elevation of the beach dunes did not affect the analysis and neither are required to mitigate the consequences of the design basis steady-state PMH.

It should be noted that many conservative assumptions were made in the analysis of runup. They are:

- The fore-dunes are completely washed away along the entire east coast of the site.
- The State Route A1A bridge spanning the discharge canal is assumed swept away in a manner that would not interfere with a wave moving up the discharge canal.
- The incident wave propagates from the ocean without any attenuation prior to reaching State Route A1A. The incident wave is the maximum breaker supported by a surge water level of EL +17.2 ft-PSL Datum with the base plain elevation in the mangrove and fore-dune areas at EL +0.0 ft-PSL Datum. The resulting incident wave height is 13.4 ft.

Since the PMH causing the maximum surge level approaches from the east, it was postulated that the west face of the plant island would not experience any significant wave runup. This is also true along the south face of the plant island. Hence, the maximum water level along both faces is EL +17.2 ft-PSL Datum.

It is noted that the flood protection level of EL +19.5 ft-PSL Datum is maintained for all Category I structures.

3.10 CLB – Flooding-Related Loading

The UFSAR considers some flooding-related loading conditions for certain cases and structures. Also, the UFSAR does not consider hydrodynamic loading and waterborne projectiles; however, tornado missiles and wind loading are considered.

3.10.1 CLB – Hydrostatic and Hydrodynamic Loads

The CLB does not consider hydrodynamic loading; however, the UFSAR considers subsurface hydrostatic loading. According to the UFSAR, all Seismic Category I structures were designed for hydrostatic loading.

Civil design criteria for Category I plant island structures specify the following groundwater levels:

- Normal groundwater table – EL +3.0 ft-PSL Datum.
- PMH groundwater table – EL +17.2 ft-PSL Datum for all structures (except EL +21.0 ft-PSL Datum at the reactor building).

The above design criteria were based upon the maximum ocean surge level at EL +17.2 ft-PSL Datum.

3.10.2 CLB – Waterborne Projectiles

Waterborne projectiles were not considered at other locations; however, windblown projectiles were analyzed. The plant was designed to withstand the effects of tornado-generated missiles.

3.10.2.1 CLB – Wind-Generated Missile Hazard

Tornado missiles provide a basis for impact loading and thus a comparison for loading by waterborne projectiles. The UFSAR considered the following tornado design criteria:

1. Plank, 4 inch by 12 inch, 12 ft long, with a density of 50 pounds per cubic ft (lb/ft^3), an impact area of 0.333 square ft (ft^2), an impact velocity of 322 ft per second (ft/s), a weight of 200 lbs, and an impact height from grade to top of structure

2. Steel rod, 1 inch diameter by 3 ft, with a density of 490 lb/ft^3, an impact area of 0.00545 ft^2, an impact velocity of 163 ft/s, a weight of 8 lbs, and an impact height from grade to top of structure

3. Pipe, 6 inch diameter Schedule 40 by 15 ft, with a density of 490 lb/ft^3, an impact area of 0.196 ft^2, an impact velocity of 116 ft/s, a weight of 284.5 lbs, and an impact height from grade to top of structure

4. Pipe, 12 inch diameter Schedule 40 by 15 ft, with a density of 490 lb/ft^3, an impact area of 0.785 ft^2, an impact velocity of 116 ft/s, a weight of 743.4 lbs, and an impact height from grade to top of structure

5. Wood utility pole, 13.5 inch diameter by 35 ft long, with a density of 43 lb/ft^3, an impact area of 0.995 ft^2, an impact velocity of 153 ft/s, a weight of 1,497 lbs, and an impact height from grade to 25 ft above grade

6. Automobile, with an impact area of 20 ft^2, an impact velocity of 84 ft/s, a weight of 4,000 lbs, and an impact height from grade to 25 ft above grade

7. Plank, 2 inch by 4 inch 10 ft long, with a density of 50 lb/ft^3, an impact area of 0.0556 ft^2, an impact velocity of 3,403 ft/s, a weight of 27.8 lbs, and an impact height from grade to top of structure

Safety-related SSCs (or the structures they are housed within) are protected to withstand a spectrum of wind and tornado loadings. Barriers are designed to withstand both local damage (e.g., penetration, perforation, scabbling, spalling, punching shear) in the impacted area and overall response of missile impact (NEE, 2013b; NEE, 2014c).

3.11 Debris and Sedimentation

Debris and sedimentation were not considered in the CLB; however, erosion was analyzed. Wave-induced erosion around the plant island was investigated for the design of PSL. This investigation included examination of historical beach erosion, the use of laboratory test results which simulate wave erosion during a storm, and the use of conservative methods for estimating the quantity of erosion.

The total quantities of erosion computed were the sum of erosion by frontal wave action, littoral drift losses from waves breaking at an angle to shore, and current-induced scouring resulting from hypothesized breaches of the barrier island at Big Mud Creek and the intake canal.

The PMH approach towards the site was postulated from due east; however, wind and wave action from the east, northeast, and the southeast is also expected. Wave attack to the western side, hence erosion, was expected to be minor because the time at which this wave direction applies coincides with low surge.

No significant erosion is expected on the south face of the plant island because of low wave heights and generally high topographic features of EL +10 to EL +15 ft-PSL Datum.

Consequently, emphasis was given to the erosion that could occur along the north and east faces of the plant island.

3.11.1 Historical Storm-Induced Erosion

To predict the magnitude of beach erosion that can occur at PSL (Hutchinson Island), a thorough list of severe storms which occurred along the Atlantic and Gulf Coasts of the United States, including one in England and one in the Baltic Sea, was studied. Among these storms, three U.S. storms were chosen for detailed investigation because they caused the highest storm-induced erosion on record (at the time of the analysis in 1975). These three storms are Hurricane Carla (1961), the November Storm of 1953, and the September Hurricane of 1938.

3.11.2 Laboratory Erosion Tests

Simulated beach erosion due to frontal wave attack at Hutchinson Island was also investigated with the use of laboratory test models. The large wave tank experiments used sand of either 0.22 mm or 0.4 mm in diameter to mold a sand beach with a 1:15 slope in the tank. A wave height of 5.5 ft and a period of 11.3 seconds were considered to be most representative of actual hurricane waves. Astronomical tidal changes with a range of 2.8 ft were also simulated in the wave tank experiments, and no significant impact on the erosion results was found.

3.11.3 Erosion Quantity Estimates

Erosion around the plant island was estimated based on the combined effects of frontal wave attack, littoral drift and current-induced scouring. To estimate the amount of erosion and its rate, representative soil samples were obtained from various locations about the plant site; a total of 17 samples were analyzed. The medium grain size (d_{50}) was about 0.3 mm. However, the erosion analysis was performed using a more conservative and smaller grain size of 0.22 mm as used for laboratory erosion tests. Both frontal wave erosion and littoral drift loss were estimated based on the breaking wave height, which is invariably associated with depth of water. For determining the breaking wave height, the base plain elevation along the east face of the plant island was selected to be EL +4 ft-PSL Datum, while along the north face, EL +5 ft-PSL Datum. Beach dunes and mangroves areas were not relied upon to provide protection from hurricanes. The dunes were conservatively assumed to be eroded to EL +4 ft-PSL Datum prior to computing the maximum possible quantities of erosion from State Route A1A and the plant island. No credit was taken for the energy dissipated or the time consumed in erosion of the dunes. Since the "stable base plain" elevations of EL +4 ft-PSL Datum to the east and south of the plant island and EL +5 ft-PSL Datum on the north are the natural ground elevations of extensive flat areas, their viability was not dependent on the existence of the mangroves. No reduction of wave height or energy by the mangroves was assumed.

3.11.3.1 Frontal Wave Erosion

Frontal wave erosion is associated with wave attack normal to the shoreline based on the laboratory erosion test results and the surge hydrographs of various stalled PMHs during the rising, quasi steady-state, and falling phases of the surge ranges. Different methods were used in determining the erosion quantities:

1. The first method was to utilize the test data by arbitrarily doubling the duration of each phase of the surge for conservatism and then rounding each duration upward to the next higher duration. To transform the laboratory test results into the field erosion prediction, the test results were multiplied by a scale factor, which indicated the relation between the PMH wave energy flux and that of the laboratory as the square of the wave height ratio. Since the rate of setdown for Cases 3 and 6 was slow enough to allow building up a bar, the scale factor for the falling phase was reduced to 1.2 to account for wave breaking on the bar. An alternative to the above estimate was also investigated by not differentiating the various phases of the surge above EL +8 ft-PSL Datum. The total erosion is obtained from the laboratory results by multiplying by a factor of two for conservatism. The scale factor for predicting field erosion was established by selecting an average breaking wave height of 8 ft during the entire duration of surge.

2. The second method applied the laboratory test results of erosion over the duration of the rising and falling tidal phases during a surge. To accomplish the prediction of field erosion, a break energy flux factor was multiplied by the laboratory test result. For an initial 6-hour interval that approximates a rising or falling tidal phase, an estimated erosion of 160 ft^3/ft occurs, which is equivalent to 320 ft^3/ft over a 12-hour full tidal cycle. However, if a full tidal cycle is considered for erosion, the laboratory test gives an erosion of 240 ft^3/ft during the 12-hour period. This demonstrates that a reduction in frontal wave erosion during an ebb tide phase can be expected. In the process of estimating, each 6-hour erosion time interval is treated as though it was the beginning of an erosion cycle; thus, this approach will result in a very conservative estimate.

3.11.3.2 Littoral Drift Loss

Littoral drift is the sediment moved along the shoreline by the action of waves and currents. Its alongshore rate can be estimated by an empirical method derived from field observations. The Shore Protection Manual discusses the available methods for estimating the littoral drift rate under various conditions. This method was used to derive a littoral drift rate versus breaker height curve for estimating the drift loss around the perimeter of the plant island. Two methods were adopted to estimate the littoral drift losses. The first method utilized a constant breaker wave height of 8 ft for rising, quasi steady-state, and falling phases of the surges to estimate the loss. The second method was more refined since it considered the average breaker heights at various time intervals during a PMH.

3.11.3.3 Current-Induced Scour

Current-induced scour takes place when particles composing an embankment are acted upon by forces sufficient to cause them to move. Conservatively assuming that sandy material composing an embankment is non-cohesive, it was possible to allow a quantitative assessment of the extent of current-induced scour resulting from storm breaching. This assessment is best made by applying tractive force theory as utilized in estimating the bed-load transports for alluvial rivers. The rate of bed-load transport depends upon the magnitude of the flow's prevailing tractive force in excess of the critical tractive force of an embankment particle.

For a study of current-induced scour, the worst case breach would be a single breach in Hutchinson Island occurring at the eastern extremity of Big Mud Creek, which is located north of the plant island. The openings of more than one breach along Hutchinson Island would reduce the amount of current-induced scour along Big Mud Creek because water backed up in the Indian River would return to the ocean via several breach channels. Breaching through the ocean intake canal is also possible but would not be as serious as a single breach at Big Mud Creek with respect to scour at the plant island. These analyses were presented separately as follows:

1. It was conservatively assumed that at the onset of storm surge, the bridge and the embankment at State Route A1A were cut down to a bottom elevation of EL -6 ft-PSL Datum with a breach width of 1,000 ft, which approximates the average width of Big Mud Creek at the MLW level. This assumption would result in the greatest flow rate via Big Mud Creek.

2. Apply a single breach at the ocean intake canal cut down to a bottom elevation of EL -6 ft-PSL Datum.

3.11.3.4 Total Quantity of Erosion

The amounts of sediment material eroded around the plant island due to frontal wave action and littoral drift loss were determined by the first and second methods described above. The estimate of erosion by the first method was not much different from the alternative estimate; therefore, the results from the first method were used in constructing the erosion profiles. From the total erosion quantities of various PMHs, Case 3 and the steady-state PMH (R = 20 nmi, T = 4 knots, high tide) were dropped from further investigation because the former is similar to Case 6 and the latter is less conservative than its low tide case. As a result, the stalled PMH Case 6 and the steady-state PMH (low tide) were selected for delineating the erosion profiles. In addition, Case 8 was also selected because of its practical importance.

The erosion estimates by the second method were used to construct the erosion profiles for both stalled PMH Cases 6 and 7. In general, the estimate of erosion for the stalled PMH Case 7 (second method) gives the most conservative value along the east face of the plant island, while the estimate by the first method for stalled PMH Case 6 gives the most conservative value of erosion along the north face of the plant island.

Since the amount of erosion due to current-induced scour was estimated in terms of recession of shore embankment, the total wave action impact on the PSL site was evaluated by adding this recession to the positions of the erosion contours resulting from the frontal wave action and littoral drift loss.

3.11.4 Erosion Profiles and Contours

Having determined the erosion quantities due to frontal wave attack and littoral drift loss for various PMH cases, erosion profiles and contours were constructed around the plant island along transects at selected site locations. These transects are shown on Figures 3-1 and 3-2. Along the north and south faces of the plant island, additional recession of embankments due to channel scour along the Big Mud Creek and intake canal was taken into consideration in constructing the erosion contours.

Of the 13 transects, six (A, B, C, D, M, and N) are located along the north face, four (E, F, G, and H) are located along the east face, one (I) is on the south face, and Transects J and K are located to the east of the plant island for studying the erosion of State Route A1A and the discharge canal dikes, respectively. A summary of reserve distances is provided in Table 3-3 for the various PMH cases.

3.11.5 Debris Control in Cooling Water Intake Canal

Debris control is provided throughout the PSL intake canal. Water entering the canal has to travel through submerged velocity caps located offshore of Hutchinson Island and through multiple submerged nets that span the width of the canal. Water then enters the intake structure through a coarse screen followed by a fine mesh travel screen that continuously removes debris that may have entered past the coarse screen.

3.12 CLB – Low Water Considerations

For relatively straight coastlines such as Hutchinson Island, the offshore winds will seldom provide sufficient offshore transport to depress the water level below the initial level because of the southward alongshore currents developed during the storm. However, to conservatively estimate the possible extreme low water elevation due to a PMH, it is assumed that the alongshore currents are negligible and do not offset the offshore transport. Since the hurricane is basically a low pressure system, the water level rise due to pressure drop is subtracted from the surge level computation. With the lowest astronomical tide of EL -1.25 ft-PSL Datum and neglecting initial surge, it is found that the extreme low tide on the open coast due to the PMH (140.6 mph) prevailing over the Indian Ocean would result in an extreme low tide of EL -3.0 ft-PSL Datum. Low water effects due to tsunamis and seiche were not considered.

At low tide and under 50 to 60 mph winds, water levels in the Indian River would be lowered to no more than EL -0.2 ft-PSL Datum at the plant site.

The intake structure bottom elevation is EL -31 ft-PSL Datum. The circulating water pump suction elevation is EL -16 ft-PSL Datum. The intake cooling water (ICW) pump suction elevation is EL -18.5 ft-PSL Datum. Minimum submergence is 6 ft for the circulating water pumps. For the ICW pumps, the minimum submergence is 4 ft for 14,500 gallon per minute (gpm) flow. Therefore, the minimum water level which will sustain the required cooling water flow (14,500 gpm) is EL -14.5 ft-PSL Datum. Since the lowest water level in the intake canal with all circulating water pumps operating would be EL -6 ft-PSL Datum, and EL -9 ft considering 3 ft additional drawdown from PMH maximum wind, the intake water pumps are assured sufficient submergence under all conditions. In the unlikely event that the ICW canal is unavailable, water can be provided from Big Mud Creek via valves in the UHS barrier wall.

3.13 CLB – Combined Events

Combined events were examined for the probable maximum storm surge (PMSS) and associated wind-wave runup as discussed in Sections 3.4 and 3.9.

4.0 FLOODING HAZARDS REEVALUATION

The following sections discuss the flood causing mechanisms and the associated WSELs and related effects that were considered in the PSL flooding hazards reevaluation.

4.1 Local Intense Precipitation

LIP is a measure of extreme precipitation (high intensity/short duration) at a given location. Generally, for smaller basin areas (up to ten square miles), shorter storm durations produce the most critical runoff scenario. High intensity rainfall in a small area has a short time of concentration and therefore a high intensity runoff. Therefore, the shorter storm over a small watershed will result in higher flow rates for the PSL LIP.

4.1.1 Local Intense Precipitation Intensity and Distribution

As prescribed in NUREG/CR-7046 (NRC, 2011), the LIP is the 1-hour, 1-square mile (2.56 square kilometer [km^2]) PMP at the PSL site location. Parameters to estimate the LIP are from USACE Hydrometeorological Report 51 (HMR-51) (NOAA, 1978) and Hydrometeorological Report 52 (HMR-52) (NOAA, 1982). Point rainfall (one square mile) LIP values for durations of one hour and less are determined using the charts provided in HMR-52. Using HMR-52 and the site location (Figure 4-1), the 1-hour, 1-square mile precipitation depth estimate is 19.4 inches per hour. As described in Section 2.4.3, severe precipitation can result from thunderstorms or tropical storms. Note that the HMR-51 and HMR-52 studies do not differentiate between thunderstorms and tropical storms for the causative mechanism for the LIP.

When applying LIP to determine the flood hydrograph, it is necessary to specify how the rain falls with time; that is, in what order various rain increments are arranged with time from the beginning of the storm. Such a rainfall sequence in an actual storm is given by a mass curve of rainfall, or the accumulated rainfall plotted against time from the storm beginning. Therefore, once the depth duration is determined, a critical temporal distribution is created for a synthetic storm hyetograph. HMR-52 uses 12 ordered segments for an event to define a synthetic storm hyetograph. HMR-52 ranks the 12 segments based on the total rainfall and defaults to a center loaded distribution (third quartile), which has the most intense rainfall in the middle of the storm duration. By comparison, a front loaded temporal distribution (first quartile) has the most intense rainfall at the beginning of the rainfall duration; likewise, an end loaded temporal distribution (fourth quartile) has the most intense rainfall at the end of the rainfall duration.

4.1.2 Local Intense Precipitation Considerations With and Without Hurricane Preparation Implementation

Because the HMR-51 and HMR-52 studies do not distinguish between thunderstorms and tropical storms, it is presumed that the LIP could occur during either event. Therefore, the plant may or may not be in hurricane preparation mode at the onset of an LIP event. During flood preparedness mode, the site would have implemented physical barriers (i.e., stoplogs at plant doors) and procedures to prevent flooding; these barriers and procedures can also be effective at preventing potential effects of LIP. The more conservative plant mode would be to not be in the hurricane preparedness mode. Under these conditions, excess or accumulated runoff could enter openings, penetrations, or pathways to SSCs.

4.1.3 Local Intense Precipitation Modeling

To estimate the WSELs resulting from the LIP, a two-dimensional runoff model of the PSL property was created. The model is capable of simulating complex precipitation run-on and runoff processes using full mass and energy conservation methods. The plant drainage system, including catch basins, floor drains, and associated piping, was conservatively assumed to not be functional for the analysis.

The sections below describe the LIP evaluation process for PSL:

- Runoff model development
- Selection of surface infiltration and roughness characteristics
- Impediments and obstructions to flow
- Runoff transformation, translation, and conveyance processes
- Precipitation input
- Model results: maximum water depths and flow velocities

4.1.3.1 Local Intense Precipitation Model Development

FLO-2D PRO (Build 13.11.06) software by FLO-2D Software, Inc. (FLO-2D, 2013) is used to create an elevation grid and render the results of the LIP.

The model has a number of components to simulate sheet flow, buildings and obstructions, sediment transport, spatially variable rainfall and infiltration, floodways, and many other flooding details. Predicted flow depth and velocity between the grid elements represent average hydraulic flow conditions computed for a small time step (on the order of seconds). Typical applications have grid elements that range from 5 to 500 ft on a side and the number of grid elements is theoretically unlimited (FLO-2D, 2012). The resultant output files yield surface water elevations, flow velocities, and other hydraulic parameters at individual grid elements.

Bathymetry and topography data points were imported into FLO-2D, and a 20-ft grid system was then derived from these points. Figure 4-2 shows the study area with the rendered elevation grid system overlaid. The boundary elements were prescribed as outflow points with no prescribed hydrograph; thus, the water can flow freely out of the domain.

Six different scenarios were considered (A to F), all of which included four different precipitation hyetograph distributions of the PMP. Four synthetic storm distributions were considered. Each of the hyetographs has a peak in the corresponding quartile. The synthetic storms were created by dividing the 60-minute duration into 12 incremental segments and arranged such that four storm distributions were created: first quartile, second quartile, third quartile, and fourth quartile (Figure 4-3). Figure 4-4 presents the cumulative precipitation distributions.

Scenarios A, B, and C consider the plant as is and Scenarios D, E, and F consider the plant with the inclusion of the proposed FLEX building located within the plant area. Each scenario also considers the number of reactor units in operation, which determines the water level in the intake canal of PSL. The water level in the intake canal is the highest when no units are in operation and the lowest when the two units are running. Table 4-1 presents the characteristics associated with the scenarios.

4.1.3.2 Surface Infiltration and Surface Roughness

Because of the large percentage of impervious area over the model domain, the relatively short duration of the storm, and the extreme value nature of the simulation (19.4 inches of rain in one hour), the antecedent conditions are of full ground saturation; thus, zero infiltration values (zero runoff losses) were used. The zero infiltration assumption is consistent with the prescribed methodology of NUREG/CR-7046.

Manning's n surface roughness values are selected for the various site cover conditions based on published values. The Manning's n values for surfaces within the model area were chosen using data presented in the FLO-2D Reference Manual (FLO-2D, 2012).

Figure 4-5 shows the study area overlaid with the Manning's n values. Manning's values for concrete/asphalt areas ranged from 0.02 to 0.03, while non-concrete/asphalt areas ranged from 0.04 to 0.05. Both the intake and discharge canals were assumed to have water at an elevation corresponding with that of the survey; thus, for the simulated event, an n value of 0.013 was assumed for those areas. Due to the nature of the revetment (articulated concrete matting) on the sides of those canals, those areas were ascribed a value of 0.03. The dense vegetation areas were ascribed a value of 0.5. Due to the nature of the building roofs and to avoid high velocities from the rooftops, which conduce to model instabilities, a roughness value of 0.05 was assigned.

If during the computation, sheet flow occurs within a grid cell (water depth is less than six inches), the Manning's roughness value of the cell is automatically changed to the recommended value of 0.2 (FLO-2D, 2012).

4.1.3.3 Obstructions and Impediments to Flow

Obstructions and impediments to surface water flow include buildings, barriers, topographic features, and the VBS. Buildings were entered explicitly into the model elevation grid. Barriers (such as the VBS) were entered as levees. Topographic features are captured in the digital elevation model (DEM) surface rendering.

Runoff from buildings is a hydrologic feature of the model. The roofs of buildings are entered explicitly into the model. The elevations were ascribed as those obtained from the survey, where available. When no survey information was available, a rough estimate of 10 ft per floor was assigned to the roof heights of the buildings and entered manually by uniformly increasing the grid cell elevation attribute. Care was taken to guarantee the existing flow paths on the ground since water will only flow around elevated grid cells that represent buildings. As a result, the building rooftops are not credited for water storage and water is not able to flow into the buildings in the model. Flows from rooftops are routed directly to the ground adjacent to the building.

The FLO-2D levee component confines flow on the surface by blocking one of the eight flow directions. Levees are designated at the grid element boundaries and often follow the boundaries along a series of consecutive elements. A levee crest elevation can be assigned for each of the eight flow directions in a given grid element. Inherent to the model, the levee feature does not allow for any way to pass through the barrier–it can only be overtopped. When the flow depth exceeds the levee height, the discharge over the levee is computed using the broad-crested weir flow equation. Levees are used to represent walls and berms within the project area (FLO-2D, 2012). Thus, the levee feature for this calculation was used to model the VBS that surrounds the building compound of the PSL (Figure 2-2). The barrier wall present at the western middle end of the discharge canal was also represented in this fashion. Figure 2-2 also identifies other features modeled.

4.1.3.4 Runoff Processes

FLO-2D is a two-dimensional, physical process model that routes rainfall-runoff and flood hydrographs over unconfined flow surfaces or in channels using the continuity equation and a dynamic wave approximation to the momentum equation. A set of partial differential equations (St. Venant equations) is solved using the second-order Newton-Raphson tangent finite difference method. The solution domain in the FLO-2D model is discretized into uniform, square grid elements, where the discharge is computed in eight different flow directions across the grid element (FLO-2D, 2012).

The full dynamic wave equation is a second-order, nonlinear, hyperbolic partial differential equation. To solve the equation for flow velocity at a grid element boundary, initially the flow velocity is calculated with the diffusive wave equation using the average water surface slope (bed slope plus pressure head gradient). This velocity is then used as a first estimate in the second-order Newton-Raphson tangent method to determine the roots of the full dynamic equation and Manning's equation is applied to compute the friction slope. If the solution fails to converge after three iterations, the algorithm defaults to the diffusive wave solution (FLO-2D, 2012).

The three keys to a successful application of the model are:

1. Checking volume conservation
2. Checking the area of inundation
3. Checking the maximum velocities and ensuring there is no numerical surging

The maximum mass balance error (volume conservation error) for the LIP estimates is 2×10^{-6} percent (1.6×10^{-5} acre-ft) with six significant figures reported. These values oscillate between 1.4 to 1.6×10^{-5} acre-ft for all the simulations (2×10^{-6} percent), which are within the acceptable continuity error range (FLO-2D, 2012).

The maximum area of inundation is examined as the maximum flow depth, temporally and spatially. Results show that the area of inundation is reasonable since the entire plant area lies within the 1-square mile LIP of 19.4 inches in one hour.

Similarly, the maximum velocity and numerical surging are related to the area of inundation. Numerical surging is the result of a mismatch between flow areas, slope, and roughness and can cause an over-steepening of the flood wave. To avoid numerical surging, the FLO-2D model is subject to the Courant-Friedrichs-Lewy (CFL) condition due to the explicit numerical scheme used for the solution. The CFL condition states that the numerical solution remains stable only for Courant numbers $C \leq 1$ defined as:

$$C = \frac{|V| \pm c}{\frac{\Delta x}{\Delta t}} \qquad \text{(Equation 4.1)}$$

where:

C = Courant number
V = velocity
c = wave celerity
Δx = grid size
Δt = time step

To preserve stability, a conservative maximum $C_{max} \leq 0.6$ is imposed on the floodplain cell solution. This setting forces a reduction in time step if the stability threshold is approached as the solution progresses. A 20-ft grid resolution (Δx) is used to describe the surface topography of the PSL because the flow pathways on the site are of that width or wider. The resulting solution of the LIP runoff produced wave celerities (c) and velocities (v) above 0.0167 ft/s. The default time step (Δt) of FLO-2D model is 30 seconds (FLO-2D, 2012); violation of the CFL condition was a consideration for the PSL FLO-2D model. FLO-2D internally modified the time step until the specified CFL condition was met. For example, for $C = 0.6$; $c = 9.8$ ft/s; $V = 13.4$ ft/s; and $\Delta x = 20$ ft, from Equation 4.1, Δt is calculated as:

$$\Delta t = \frac{C \cdot \Delta x}{v + c} = \frac{0.6 \cdot 20}{13.4 + 9.8} = 0.52s \qquad \text{(Equation 4.2)}$$

A minimum depth of flow of 0.01 ft was imposed across solution cells to prevent small flow oscillations from unnecessarily increasing the computational time. As the PSL LIP runoff model is primarily concerned with flooding, and therefore water depths greater than 0.01 ft, this modification was assumed to be valid.

A non-zero storage volume must be applied to each grid cell for FLO-2D to compute a solution. A storage depth of 0.01 ft was applied to all grid cells since 0.01 ft is small in comparison to the LIP rainfall of 19.4 inches in one hour. This artificial abstraction is assumed to have a negligible effect on the final result.

Artificial viscosity (dynamic wave stability coefficient) and change of flow depth tolerance (DEPTOL) were not required to preserve numerical stability of the solution. Coefficients for these terms were set to zero.

4.1.3.5 Precipitation Input

The 1-hour, 1-square mile LIP of 19.4 inches was used as input to the model. Four LIP hyetographs were evaluated for all the scenarios, to determine the bounding LIP effects, with the following rainfall temporal distributions: first quartile, second quartile, third quartile, and fourth quartile. The maximum rainfall intensity occurs at the beginning of the first, second, third, and fourth temporal quartile, respectively. The hyetographs are presented on Figure 4-3. All hyetographs have a total rainfall of 19.4 inches for a one-hour duration.

4.1.3.6 Model Results

The FLO-2D program displays the results by storing attributes within each grid element. Attributes such as flow depth, flow velocity, and flow direction can then be rendered and displayed as a map to give an overview of the results, or an individual element can be selected to reveal the results at a particular grid cell.

Six different scenarios were considered (A to F), all of which included four different precipitation hyetograph distributions of the PMP. Each of the hyetographs has a peak in the corresponding quartile. The number of reactor units in operation determines the water level in the intake canal of PSL. The water level in the intake canal is the highest when no units are in operation and the lowest when the two units are running. The effect of the water level conditions appears to be significant, but not in the vicinity of the reactor buildings. The hyetograph distribution has an obvious impact in the time distribution of the maximum water depths and velocities. However, the corresponding maximum values, upon which the final evaluations are carried out, are very similar.

Maximum water depths and flow velocities were determined over the entire domain for all the scenarios and precipitation hyetographs considered (Table 4-2). In addition, 36 points of interest (POIs), shown on Figure 4-6, were considered. The time series of both depth and velocity were obtained at these POIs. All the scenarios were simulated for a time period of two hours to cover the recession and establish the duration of flooding. The duration of the rainfall event was always set equal to one hour. A summary of the results of flow depth for all the simulations at the POIs is shown on Figure 4-7. The maximum water depth values for each of the scenarios at the POIs are presented in Table 4-2.

4.2 Flooding in Streams and Rivers

PSL is located on Hutchinson Island, and there are no streams or rivers that contribute to flooding conditions at the site. The site area itself (less than one square mile) is the only contributing surface water runoff area; there are no adjacent surface water run-ons to the site. The Indian River and Big Mud Creek are inappropriately named. The Indian River is a tidal lagoon, not a stream. Big Mud Creek is an arm of the Indian River Lagoon. Water levels are affected by the Atlantic Ocean and by wind conditions, not by any stream flow; thus, PMF on streams is not applicable.

4.3 Dam Breaches and Failures

There are no dams located upstream or downstream of the site; therefore, there is no potential for dam breach-related flooding.

4.4 Storm Surge

For the storm surge at PSL, a computer-based numerical model is used to estimate the surge and wave effects from a suite of sufficiently large design storms to determine the PMSS. The numerical model is developed using the Delft3D Version 4.00.01 software package (Deltares, 2011d). The design hurricanes are developed from the National Weather Service (NWS) Technical Report 23 (NWS23) PMH methodology (NWS, 1979).

Numerical modeling and PMH development are described in Sections 4.4.1 and 4.4.2, respectively.

Subsequent sections provide:

- Description of the Delft3D modeling system (Section 4.4.3)
- Development of the numerical surge model (Section 4.4.4)
- Coupled FLOW and WAVE model (Section 4.4.5)
- Model processes (Section 4.4.6)
- Physical and numerical parameters (Sections 4.4.7 and 4.4.8)
- Boundary conditions (Section 4.4.9)

- Selection and treatment of antecedent water levels (AWL), tides, and sea level rise (Section 4.4.10)
- Calibration and validation of the numerical model (Section 4.4.11)
- Methodologies and development of design hurricane parameters (Section 4.4.12)
- Suite of storm scenarios analyzed (Section 4.4.10)
- Final results of storm surge analyses (Section 4.4.11)
- Coincident wind-wave runup (Section 4.4.12)
- PMSS maximum water level (Section 4.4.13)

4.4.1 Overview – Numerical Surge Model

Delft3D is an advanced numerical modeling program that is capable of simulating flows, sediment transports, waves, water quality, morphological developments, and ecology. For these analyses, the Delft3D-FLOW and Delft3D-WAVE modules are used to simulate the coupled effects of flow movement (surge) and wave propagation (wave spectra, height, period, and setup) through a water body (Atlantic Ocean) when acted upon by external forcing functions (wind fields, atmospheric pressure fields, and tides) at the planetary boundary. The physical features of the numerical model are created from regional and local bathymetry and topography. The model is calibrated and validated to observed tides and historical storms (Hurricanes Irene, Floyd, Frances, and Jeanne). The AWL conditions, including 10 percent exceedance high and low tides and potential sea level rise, are included in the numerical model. Sea level rise is estimated for the remaining 30-year licensed life of PSL.

4.4.2 Overview – Design Hurricane

For these analyses, the design hurricane is selected in accordance with applicable guidance documents (NUREG/CR-7046, NUREG 0800, JLD-ISG-2012-06), which refer to the PMH methodology of NWS23. JLD-ISG-2012-06 states that the PMH methodology is acceptable for licensing decisions. Using the NWS23 methodology, the critical PMH parameters of storm size, pressure, and wind fields are determined for a storm making landfall near PSL. A sufficient number of storm radii, headings, and forward speeds are analyzed to determine the critical storm (i.e., the PMH).

4.4.3 Delft3D Modeling System

Delft3D uses a gridded domain to solve two-dimensional and three-dimensional flow problems with the capability of coupling a model with wave simulation algorithms. The gridded domain is created with the Delft3D-RGFGRID module, flow processes are simulated with the Delft3D-FLOW module, and wave simulations are computed with the Delft3D-WAVE module.

The Delft3D-RGFGRID module uses an open form approach where curvilinear or orthogonal grids can be used. Delft3D can employ a nested grid and/or domain decomposition approach when models extend over large domains. Large-spaced grids are used in the overall domain, and successively finer detailed grids are used nearer the area of interest. In this model, four domain decomposition FLOW grids and five nested WAVE grids are used.

In Delft3D-FLOW, the hydrodynamics in storm surge conditions are simulated by solving the system of two-dimensional shallow water equations that consists of two horizontal momentum equations and one continuity equation (IHE, 2003). For each control volume in the computational grid, the depth-averaged shallow water equations are solved. These are derived from Navier-Stokes equations for incompressible free surface flow under shallow water and the Boussinesq assumptions. In the vertical momentum equation, the vertical accelerations are neglected, which leads to the hydrostatic pressure equation. Delft3D solves these equations to compute the storm surge water level.

The conservation of momentum in the x-direction (depth and density averaged):

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+g\frac{\partial \eta}{\partial x}-fv+\frac{1}{\rho}\frac{\partial p_a}{\partial x}+\frac{g|U|u}{C^2(d+\eta)}-\frac{\tau_{wx}}{\rho_w(d+\eta)}-\varepsilon\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)=0 \quad \text{(Equation 4.3)}$$

The conservation of momentum in the y-direction (depth and density averaged):

$$\underbrace{\frac{\partial v}{\partial t}}_{[1]}+\underbrace{u\frac{\partial v}{\partial x}}_{[2]}+\underbrace{v\frac{\partial v}{\partial y}}_{[3]}+\underbrace{g\frac{\partial \eta}{\partial y}}_{[4]}\underbrace{\mp fu}_{[5]}+\underbrace{\frac{1}{\rho}\frac{\partial p_a}{\partial y}}_{[6]}+\underbrace{\frac{g|U|v}{C^2(d+\eta)}}_{[7]}-\underbrace{\frac{\tau_{wy}}{\rho_w(d+\eta)}}_{[8]}-\underbrace{\varepsilon\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)}_{[9]}=0 \quad \text{(Equation 4.4)}$$

The depth- and density-averaged continuity equation is given by:

$$\frac{\partial \eta}{\partial t}+\frac{\partial (d+\eta)u}{\partial x}+\frac{\partial (d+\eta)v}{\partial y}=0 \quad \text{(Equation 4.5)}$$

where:

[1]	= local accelerations
[2], [3]	= convective accelerations
[4]	= surface slope
[5]	= Coriolis force
[6]	= atmospheric pressure gradient
[7]	= bottom friction
[8]	= external force by wind
[9]	= depth averaged turbulent viscosity
C	= Chézy coefficient
d	= bottom depth
f	= Coriolis parameter
ε	= diffusion coefficient (eddy viscosity)
U	= absolute magnitude of total velocity, $U = (u^2 + v^2)^{1/2}$
η	= water level above reference level
U, v	= depth-averaged velocity
ρ_w	= mass density of water
τ_{wx}, τ_{wy}	= x and y components of wind shear stress, $\tau_w = \rho_a C_d W^2$
ρ_a	= density of water
C_d	= wind drag coefficient
W	= wind speed at 10m above the free surface

Wave transformation is performed using Simulating WAves Nearshore (SWAN), which is part of the Delft3D-WAVE module. SWAN is a spectral wave model that computes random, short-crested wind-generated waves in coastal regions and inland waters. SWAN models the action density spectrum contained in waves as they travel over the ocean surface towards the shore.

All information about the sea surface is contained in the wave variance spectrum or energy density E(σ, θ), distributing wave energy over (radian) frequencies σ (as observed in a frame of reference moving with current velocity) and propagation directions θ (the direction normal to the wave crest of each spectral component). Usually, wave models determine the evolution of the action density N (~x, t, σ, θ) in space ~x and time t. The action density is defined as N = E/σ and is conserved during propagation in the presence of ambient current, whereas energy density E is not. In SWAN, it is assumed that the ambient current is uniform with respect to the vertical coordinate and is denoted as ~U (Deltares, 2011e).

$$\underset{[1]}{\frac{\partial}{\partial t}N} + \underset{[2]}{\frac{\partial}{\partial x}c_x N} + \underset{[3]}{\frac{\partial}{\partial y}c_y N} + \underset{[4]}{\frac{\partial}{\partial \sigma}c_\sigma N} + \underset{[5]}{\frac{\partial}{\partial \theta}c_\theta N} = \underset{[6]}{\frac{S}{\sigma}} \qquad \text{(Equation 4.6)}$$

where:

[1]	=	local rate of change of acceleration density in time
[2], [3]	=	propagation of action in geographical space with velocities $c_x, and\ c_y$ in x and y space, respectively
[4]	=	shifting of the relative frequency due to variations in depths and currents (with propagation velocity c_σ in σ space)
[5]	=	depth-induced and current-induced refraction (with propagation velocity c_θ in θ space
[6]	=	source term in terms of energy density representing the effects of generation, dissipation, and nonlinear wave-wave interactions

The SWAN model accounts for (refractive) propagation due to current and depth and represents the processes of wave generation by wind, dissipation due to whitecapping, bottom friction and depth-induced wave breaking, and nonlinear wave-wave interactions (both quadruplets and triads) explicitly. Wave blocking by currents is also explicitly represented in the model. Output from the model includes significant wave height, wave period, wave dissipation, and wave direction at each point within the computational grid (Deltares, 2011e).

The SWAN model is based on the discrete spectral action balance equation and is fully spectral (in all directions and frequencies). The latter implies that short-crested random wave fields propagating simultaneously from widely different directions can be accommodated (e.g., a wind sea with superimposed swell). SWAN computes the evolution of random, short-crested waves in coastal regions with deep, intermediate, and shallow water and ambient currents (Deltares, 2011e).

Coastal surges are driven primarily by momentum transmitted to the water column towards the coast by winds. Waves propagate energy and momentum towards the coast due to the processes of refraction, diffraction, and dissipation. This momentum transfer causes a horizontal variation of the water column, commonly called wave setup. This is obvious in the variation of a significant wave height. The corresponding variation in momentum transport is less obvious. This notion of spatially varying momentum transport in a wave field is called radiation stress (WMO, 1998).

The momentum transfer and loss rate from wave breaking is dependent on the slope and depth of the sea bottom and varies considerably throughout a region of interest and from site to site (Resio and Westerink, The momentum transfer and loss rate from wave breaking is dependent on the slope and depth of the sea bottom and varies considerably throughout a region of interest and from site to site (Resio and Westerink, 2008). The wave forces will enhance the energy dissipation near the bottom in the storm surge model and generate a net mass flux affecting the current, especially in the cross-shore direction. These effects are accounted for by passing on radiation stress gradient determined from the computed wave parameters to the storm surge flow model. The water levels and currents computed by the storm surge model are then passed on to the wave model after this interval, which is then used by the wave model to compute the wave parameters (Vatvani et al., 2012).

Delft3D-WAVE allows for accurate representation of the coastline near and surrounding the PSL plant and includes the ability to model refraction, generation, and dissipation. The maximum wave setup calculated with a detailed Delft3D-FLOW and Delft3D-WAVE coupled model system was determined by comparing coupled model storm surge results with standalone Delft3D-FLOW storm surge (i.e., without wave effect).

4.4.4 Numerical Surge Model Development

The following subsections describe the model geometry, physical and numerical parameters, and boundary conditions.

4.4.4.1 Model Geometry

The numerical model uses a spherical coordinate horizontal grid (World Geodetic System 1984 [WGS84] latitude and longitude) and, because the primary domain of the model is the Atlantic Ocean, the vertical datum is referenced to 0 m-MSL.

A sufficiently detailed numerical model is created from local and regional bathymetric and topographic data sources:

- Deep ocean bathymetry acquired from General Bathymetric Chart of the Oceans (GEBCO) at a horizontal resolution of 30 arc seconds (1 km) (GEBCO, 2008). The grid data are in WGS84 Geographic Coordinate System (GCS), referring to m-MSL (GEBCO, 2014).

- County-wide LiDAR-derived 10-ft DEMs for St. Lucie County. The 10-ft DEM is in the State Plane Florida East coordinate system referenced to the NAD83 and NAVD88 vertical datum–all units are in ft (SFWMD, 2007).

- 1 Arc Second (30 meter) and 1/3 Arc Second (10 meter) National Elevation Dataset (NED). The NED is in the State Plane Florida East coordinate system referenced to the NAD83 horizontal datum and the NAVD88 vertical datum–all units are in meters (USGS, 2009).

- National Ocean Survey (NOS) hydrographic surveys: H08713 (NOAA, 2005); H01513A, H01513B, H01570, H01571, H04914, H01419A, H01419B, H08956, H08954, H08955, and H08959 (NOAA, 2004); H08958 (NOAA, 2002); H05027, H05028, H05031, H05032, H05047, H08783, H08839, and H08957 (NOAA, 1979); and H08955 (NOAA, 1967). NOAA's NOS surveys are in the NAD83 GCS, referring to ft-MLW (NOAA, 2005), with resolution of approximately 1/100 of a second.

- Topographic survey of PSL by Southern Resource Mapping, Inc., with 1-ft contours, in State Plane Florida East coordinate system referenced to the NAD83 horizontal datum (Southern Resources Mapping, Inc., 2013).

These data required conversion to consistent horizontal (WGS84) and vertical (MSL) datums and units for use as the base geometry for the numerical model.

According to NRC (2011 and 2013a), a two-dimensional model capable of modeling hurricane wind and pressure fields, such as Delft3D, is necessary to model a hurricane storm surge over complex topography. Four FLOW numerical domains (Figure 4-8) and five WAVE numerical domains were developed in Delft3D for storm surge modeling. Due to the size of the area under study, four FLOW domains are necessary: one overall grid with rough resolution and three local grids of increasingly finer grid resolution. The intent is to include three local fine grids within the overall domain rough grid using domain decomposition within Delft3D-FLOW; the model conveys the information from the coarse grid to provide boundary conditions for the fine grid, which vary with time during the evolution of the storm.

Only the fine grids were created close to the PSL site. The effort required to run a model on a domain decomposition grid is only justified when the fine grid provides information that is not already obtainable with the coarse grid. Three fine grids were created: a 525-meter resolution grid for representation of the continental shelf, a 75-meter resolution grid for representation of the coastline geometry, and an 18.75-meter resolution grid for representation of the PSL site.

The parameters to create the model domain, which include grid structure and resolution, bathymetry, and open boundary forcing conditions, must conform to the guidelines of the Delft3D modeling software to accurately resolve the finite differential approximations within the model.

Delft3D-RGFGRID is designed to create grids for Delft3D with minimum effort while fulfilling the requirements of smoothness and orthogonality. Smoothness is the ratio between adjacent grid cell lengths, and orthogonality is the measure of the cosine of the angle between intersecting grids. Numerical computations in Delft3D require a smooth transition between grid cells, and the error in the direction of the pressure gradient in Delft3D-FLOW computations is proportional to the deviation of the cosine value from zero. Guidelines in the Delft3D-RGFGRID User Manual (Deltares, 2011a) state the following for an acceptable computational grid:

- Size between adjacent cells should vary less than 20 percent.
- Orthogonality should be less than 0.2 for offshore areas, but as close to zero as possible.
- Smoothness between adjacent grid cell lengths is generally preferred to be less than 1.2 in the area of interest.
- Aspect ratio should be less than 2.

Two modes of grid generation are acceptable for Delft3D: curvilinear grids and rectangular grids. Curvilinear grids are applied in finite difference models to provide a high grid resolution in the area of interest and a low resolution elsewhere to decrease computational time by the computer. Curvilinear grids can be generated to take the shape of a coastline, river, structure, or other physical feature. Rectangular grids provide equal resolution over the computational area in the form of equally sized grid cells. For this calculation, the rectangular grid is used. The rectangular grid tends to have better model stability because the orthogonality, smoothness, and aspect ratio are automatically within the recommended limits when a rectangular grid is created in Delft3D-RGFGRID.

The grid size was selected carefully to minimize the inaccuracy associated with the finite differential approximations but large enough to not negatively affect the computational speed run time of the program for simulations. The overall domain covers the entire Caribbean region and the Gulf of Mexico and extends approximately 2,000 miles east of PSL towards the middle of the Atlantic Ocean (Figure 4-8). The intent is to pass boundary conditions to subsequently finer grids within the overall domain rough grid using domain decomposition.

Due to the large size of the study area, and to properly account for the Coriolis effects of the earth, the spherical grid system was used in projected coordinates of the WGS84 GCS.

To ensure the stability of the model, the Courant number (parameter that relates resolution of the grid to time step for model stability purposes) for the coarse domain was less than 10 (a threshold defined in Deltares, 2011b) near the area of interest. The Courant number for the regional (coarse) Grid Domain 1 was less than 10 as long as the time step was less than 60 seconds during the computation run for the storm surge. The Courant number for the local (coarse) Grid Domain 2 was less than 10 as long as the time step was less than 10 seconds during the computation run for the storm surge. The Courant number for the local (fine) Grid Domain 3 was less than 10 as long as the time step was less than 10 seconds during the computation run for the storm surge. The Courant number for the site (fine) Grid Domain 4 was less than 10 as long as the time step was less than 5 seconds during the computation run for the storm surge.

4.4.5 Coupled FLOW and WAVE Model

To account for the effect of flow on the waves (via setup, current refraction, and enhanced bottom friction) and the effect of waves on current (via forcing, enhanced turbulence, and enhanced bed shear stress), there are three different types of wave computations within the Delft3D module (Deltares, 2011e):

1. A WAVE computation that uses user-defined flow properties – For each wave condition, one specifies a spatially uniform water level and a spatially uniform current velocity so that the effect of flow on waves is accounted for.

2. An offline coupling of WAVE with Delft3D-FLOW – The wave computation uses flow characteristics from a completed Delft3D-FLOW computation so that the effect of flow on waves is accounted for.

3. An online coupling of WAVE with Delft3D-FLOW – The WAVE model has a dynamic interaction with the FLOW module of Delft3D (i.e., two-way wave-current interaction). Through this coupling, both the effect of waves on current and the effect of flow on waves are accounted for.

In the model, the third option (i.e., online coupling of WAVE with Delft3D-FLOW) is used so that the effect of waves on current and the effect of flow on waves are accounted for and the most accurate results are provided by each Delft3D module. Data are exchanged between the FLOW and WAVE module communication file (com-file), which contains the most recent data of the flow and wave computations. The FLOW and WAVE models are coupled every 30 minutes. This coupling interval provides the benefit of the third option listed above while maintaining computational efficiency.

In the case of coupling with Delft3D-FLOW, it is useful to extend FLOW data on the wave grid(s). In this way, a (more) uniform wave field is computed at the boundaries of the FLOW grid (Deltares, 2011e). The option "Use and Extend" in the WAVE module is activated in the model. This process allows online transfer of water level, current, bathymetry, and wind from Delft3D-FLOW to Delft3D-WAVE during the online coupling. Therefore, wind, current, and water level are directly read from the Delft3D-FLOW master definition file.

4.4.6 Model Processes

In the model, the data group specifies which processes or quantities that might influence the hydrodynamic simulation will be taken into account in the model.

4.4.6.1 Delft3D-FLOW Processes

Tidal Forcing – In the numerical models of sections of the deep ocean or large closed basins, the contribution of the gravitational forces on the water motion increases considerably and should not be neglected (Deltares, 2011c). The tide-generating forces originate from the Newtonian gravitational forces of the terrestrial system (sun, moon and earth) on the water mass. Changes in water level caused by the gravitational forces of the sun and moon occur in semidiurnal (twice daily), diurnal, and long period patterns. For the model, semidiurnal tidal forcing modes (M2, S2, N2, and K2) are included in the model as this family of tide force components dominated the tidal signal along the Florida eastern coast.

Wind and Pressure Fields – The space and time varying wind and pressure fields for each simulation were created using the space and time varying wind and pressure fields. Using the NWS23 methodology, the historical storms were recreated using parameters of storm radii, headings, and forward speeds. Hurricane track, central pressure, and the RMW at landfall were derived from the following references: Hurricane Floyd (NOAA, 1999), Hurricane Frances (NOAA, 2004a), Hurricane Jeanne (NOAA, 2004b), and Hurricane Irene (NOAA, 2011a). The Delft3D-FLOW "Additional Parameters" function was used to model the space and time varying wind and pressure fields. Wind and pressure field inputs are used in the calibration of storm surge parameters resulting from the hurricane simulations.

4.4.6.2 Delft3D-WAVE Processes

There are three generations of wave models available to compute the sea surface state in Delft3D-WAVE (SWAN). First generation wave models do not consider nonlinear wave interactions. Second generation models parameterized these interactions and include the coupled hybrid and coupled discrete formulations. Third generation models explicitly represent all the physics relevant for the development of the sea state in two dimensions, without assumptions regarding the spectral space, and energy terms are described explicitly with the addition of bottom dissipation and reflection, diffraction, and refraction terms. For PSL, the model computes the sea state from the hurricane using the third generation mode of physics.

In shallow water, six processes contribute to total wave energy density (Deltares, 2011e):

$$S_{tot} = S_{in} + S_{nl3} + S_{nl4} + S_{ds,w} + S_{ds,b} + S_{ds,br} \qquad \text{(Equation 4.7)}$$

where:

S_{tot} = total wave energy density or total wave variance spectrum
S_{in} = wave growth by wind
$S_{nl3} + S_{nl4}$ = nonlinear transfer of wave energy through three-wave and four-wave interactions

$S_{ds,w}$ = wave decay due to whitecapping
$S_{ds,b}$ = wave decay due to bottom friction
$S_{ds,br}$ = wave decay due to depth-induced wave breaking

Wave growth in the model is induced by the wind field from the FLOW module. This feature is activated in the model. Details on the specific physics of energy wave growth in the model are described in Cavaleri and Malanotte-Rizzoli (1981), Komen et al. (1984 and 1994), and Booij et al. (1999).

Whitecapping is a process of energy dissipation and is primarily controlled by the steepness of the waves. This feature is activated in the model. In the model, the whitecapping formulations are based on a pulse-based model of Komen et al. (1984). Details on the specific physics of whitecapping in the model can be found in Komen et al. (1984) and Booij et al. (1999).

Bottom friction is a process of energy dissipation. The bottom friction model that is selected for Delft3D-WAVE (SWAN) is the empirical model of the JOint North Sea WAve Project (JONSWAP). Details on the specific physics of bottom friction energy dissipation can be found in Hasselmann et al. (1973) and Booij et al. (1999). The Delft3D-WAVE manual suggests $C_{bottom} = C_{JON} = 0.038$ m^2s^3 for swell conditions and $C_{JON} = 0.067$ m^2s^3 for fully developed wave conditions in shallow water. For the computational wave grid, the wave bottom friction parameter (C_{JON}) was calibrated to be C_{JON} = 0.067 m^2s^{-3}.

When waves propagate towards shore, shoaling leads to an increase in wave height. When waves become too steep (measured by the ratio of the wave height to the wavelength), waves become unstable and break, thereby dissipating energy rapidly. This process is depth-limited in that waves of particular heights break in the same water depth. To model the energy dissipation of random waves due to depth-induced breaking, the bore-based model of Battjes and Janssen is used in Delft3D-WAVE (SWAN) (Battjes and Janssen, 1978; Booij et al., 1999). In the model, a constant breaker parameter of 0.73 (ratio of breaking wave height to breaking wave depth) was used.

When waves propagate from deep to shallow water, they slow down, grow taller, and change shape. This transformation is described as wave shoaling in coastal zone processes. The wave shoals when the wave is approaching a shoreline perpendicular with the wavelength shortened. If waves come closer to the shore at an angle, wave refraction takes place because of varying water depth. Wave refraction on a planar beach can be described by Snell's law, which relates the wave angle variation with the wave celerity change. Throughout the wave shoaling and refraction processes, the wave period remains the same and the wave energy is conserved. In the model, wave refraction was not activated in the computations.

Wind-generated waves are impacted by variability in ambient currents and depths. The statistical properties of the waves, such as their significant heights and peak periods, are modulated by directional turning (refraction) and (Doppler-like) frequency shifting. To model refraction and the frequency shift of a wave generated during propagation in spectral space, the "Refraction" and "Frequency shift" functions are activated in the Delft3D-WAVE model. Details on the specific physics of depth-induced shoaling and refraction in the model, as well as current-induced shoaling and refraction, are presented in the Delft3D-WAVE User Manual (Deltares, 2011c; Booij et al., 1999; and Dietrich et al., 2013).

The nonlinear effect is responsible for the changes in wave shape, which produce a number of harmonic components of the frequency spectrum from the original wave. In deep and intermediate water, four-wave interactions (so-called quadruplets) are important. In deep water, quadruplet wave-wave interactions dominate the evolution of the spectrum. They transfer wave energy from the spectral peak to lower frequencies (thus moving the peak frequency to lower values) and to higher frequencies (where the energy is dissipated by whitecapping) (Booij et al., 1999). In the model, quadruplets are activated. Details on the specific physics of quadruplets in the model are presented in Hasselmann et al. (1985) and Booij et al. (1999). For the nonlinear triad interactions, the default proportionality coefficient (alpha) equal to 0.1 and the default ratio of the maximum frequency over the mean frequency (beta) equal to 2.2 are used. Worldwide, the default parameters for computing waves are generally used and accepted in the industry. Changes to the default parameters are generally not made unless warranted and supported by field data. Case in point, refer to U.S. Geological Survey (USGS, 2013), Strauss and Tomlinson (2009), and Giardino et al. (2010).

4.4.7 Physical Parameters and Model Constants

The physical parameters are values associated with the conditions and properties of the physical world.

4.4.7.1 Delft3D-FLOW Physical Parameters and Model Constants

The physical parameters and constants of the model are selected as follows:

Gravitational Acceleration – A constant gravitational acceleration of 9.81 m/s^2 is used. The National Geodetic Survey (NGS), Office of Charting and Geodetic Services, establishes and maintains the basic national horizontal, vertical, and gravity networks of geodetic control (NOAA, 1986). The gravitational constant varies slightly across the study area; however, a constant value of 9.81 m/s^2 is selected for use in the model.

Water Density – Water density of 1,025 kg/m^3 is used. In a study of tide stations along the Atlantic coast of North America and South America, the seawater density is approximately 1,025 kg/m^3 (USDC, 1953). The density of surface seawater varies from 1,020 to 1,030 kg/m^3 depending on the water depth, water temperature, and influence of freshwater sources; however, an average value of 1,025 kg/m^3 is appropriate for this study given the large size of the study area.

Air Density – Air density of 1.229 kg/m^3 is used. The density of air depends on the location on the earth, altitude, and the temperature. The typical value of the density of air at sea level static conditions for a standard day is 1.229 kg/m^3 (NASA, 2010).

Wind Drag Coefficient – The wind drag coefficient is dependent on the wind speed, reflecting increasing roughness of the water surface with increasing wind speed.

Bottom Roughness – Uniform Manning's roughness values were supplied to the Delft3D-FLOW model.

Wall Roughness – Due to the large size of the four FLOW domains, the free slip condition is used; in other words, zero tangential shear stress is applied in the model at walls. In very large-scale hydrodynamic simulations, the tangential shear stress for all lateral boundaries or vertical walls can be safely neglected (Deltares, 2011c).

Horizontal Eddy Viscosity and Diffusivity – In Delft3D-FLOW, for the Reynolds-averaged Navier-Stokes equations, the Reynolds stresses are modeled using the eddy viscosity concept. The horizontal eddy viscosity is mostly associated with the contribution of horizontal turbulent motions and forcing that are not resolved (sub-grid scale turbulence) either by the horizontal grid or a priori removed by solving the Reynolds-averaged shallow water equations (Deltares, 2011c). The value for both horizontal eddy viscosity and horizontal eddy diffusivity depends on the flow and the grid size of the simulation. For large tidal areas with a grid that is hundreds of meters or more, the values for eddy viscosity and eddy diffusivity typically range from 0 m^2/s to 100 m^2/s. Herbert (Herbert, 1987) found that horizontal eddy viscosity is approximately 50 m^2/s for the Gulf Stream due to internal waves. Therefore, 50 m^2/s was used in the overall model domain for horizontal eddy viscosity and 50 m^2/s was used for horizontal eddy diffusivity. For the fine grid model domains, a horizontal eddy viscosity of 5 m^2/s was used. Deltares (Deltares, 2011c) recommends a typical value of 1 m^2/s to 10 m^2/s for grid sizes on the order of tens of meters. Secondary flow, which adds the influence of helical flow to the momentum transport, was ignored due to the large size of the domain area as these flows are insignificant.

4.4.7.2 Delft3D-WAVE Physical Parameters and Model Constants

The physical parameters and constants of the model are selected as follows:

North Convention – The direction of north with respect to the x-axis (Cartesian convention). The default value of 90 degrees (i.e., x-axis pointing east) is selected for the model (Deltares, 2011e).

Wind and Wave Convention – The nautical convention for wind and wave direction is used, the direction of the vector from the geographic north measured clockwise plus 180 degrees. This is the direction from which the waves are coming or from where the wind is blowing (Deltares, 2011e).

Gravitational Acceleration – A constant gravitational acceleration of 9.81 m/s^2 is used, consistent with the value selected for the FLOW model.

Water Density – Water density of 1,025 kg/m^3 is used. This value is consistent with the value selected for the FLOW model.

Forces – With the integration of the fully spectral SWAN model within the Delft3D model, it is possible to compute the wave forces on the basis of the energy wave dissipation rate or on the gradient of the radiation stress tensor (Deltares, 2011c). The radiation stress tensor describes the additional forcing due to the presence of the waves, which changes the mean depth-integrated horizontal momentum in the fluid layer. As a result, varying radiation stresses induce changes in the mean surface elevation (wave setup) and the mean flow (wave-induced currents) (Deltares, 2011c). The wave forces were computed by the radiation stress tensor to account for wave setup in the model.

4.4.8 Numerical Parameters

4.4.8.1 Delft3D-FLOW Numerical Parameters

Numerical parameters are specified based on the physics of flow. In Delft3D-FLOW, three primary algorithms are available: Cyclic, Waqua, and Flooding schemes. Both the Cyclic and the Flooding schemes were used.

The Cyclic scheme (also known as the Alternating Direction Implicit) can also be used to solve the continuity and horizontal momentum equations. This is a method which is computationally efficient, at least second-order accurate, and stable at Courant numbers of up to approximately 10. The applied scheme has been tested and applied in a wide range of conditions, varying from wave-dominated to tide-dominated, in two-dimensional and three-dimensional mode, and is proven to be very stable (Deltares, 2011c).

The Flooding scheme can be applied for problems that include rapidly varying flows such as hydraulic jumps and bores (Deltares, 2011c). For this scheme, the accuracy in the numerical approximation of the critical discharge rate for flow with steep bed slopes can be increased by the use of a special approximation (slope limiter) of the total water depth at a velocity point downstream. The limiter function is controlled by the threshold depth for the critical flow limiter (Deltares, 2011c).

The threshold depth is the depth above a grid cell which is considered to be wet. The threshold depth must be defined in relation to the change of the water depth per time step to prevent the water depth from becoming negative in just one simulation time step (Deltares, 2011c). To prevent this, the threshold depth is calculated in such a way that it is larger than the maximum distance the water level can fall over half time step (the time which the flooding and drying algorithm uses). The Delft3D User Manual (Deltares, 2011c) suggests the following for estimating the threshold depth:

$$\delta \geq \left|\frac{\partial \zeta}{\partial t}\right| \Delta t \approx \frac{2\pi|a|}{T} \Delta t = \frac{2\pi|a|}{N} \qquad \text{(Equation 4.8)}$$

where:

δ = threshold depth

$\left|\frac{\partial \zeta}{\partial t}\right|$ = change in the water level as a function of time

$|a|$ = amplitude

T = tidal period

Δt = time step of the computation

N = number of time steps per tidal period

For tide stations near PSL, with tidal amplitudes less than 1 m, tidal periods of approximately 12 hours, and a computational time step of 1 minute, the threshold depth should be approximately 0.01 m. Therefore, a threshold depth value of 0.01 m was used in the model.

A smoothing time of 60 minutes was selected in the numerical modeling. The smoothing time determines the time interval in which the open boundary conditions are gradually applied, starting at the specified initial condition to the specified open boundary conditions. This smoothing of the boundary conditions prevents the introduction of short wave disturbances into the model.

4.4.8.2 Delft3D-WAVE Numerical Parameters

The data group's numerical parameters that affect the stability and accuracy of the numerical computation can be modified. To obtain robust results with acceptable accuracy, the default diffusion parameters are applied (Deltares, 2011e).

Spectral Space – The amount of diffusion of the implicit scheme in the directional space through the Directional space (CDD) parameter and frequency space through the Frequency space (CSS) (Deltares, 2011e).

Directional Space – A value of CDD = 0 corresponds to a central scheme and has the largest accuracy (diffusion ≈ 0), but the computation may more easily generate spurious fluctuations. A value of CDD = 1 corresponds to an upwind scheme and it is more diffusive and therefore preferable if (strong) gradients in depth or current are present (Deltares, 2011e). The default value of CDD = 0.5 is used in the model.

Frequency Space – A value of CSS = 0 corresponds to a central scheme and has the largest accuracy (diffusion ≈ 0), but the computation may more easily generate spurious fluctuations. A value of CSS = 1 corresponds to an upwind scheme and it is more diffusive and therefore preferable if (strong) gradients in current are present (Deltares, 2011e). The default value of CSS = 0.5 is used in the model.

Accuracy Criteria – These options influence the criteria for terminating the iterative procedure in the SWAN computation (for convergence criteria of SWAN). In the model, SWAN stops the iteration if:

a) The change in the local significant wave height (H_s) from one iteration to the next is less than:

   - the fraction relative change of wave height or
   - the fraction relative change with respect to the mean value of the average significant wave height (averaged over all wet grid points)

   In the model, the relative change and the relative change with respect to the mean value of 0.01 were used.

b) And if the change in the local mean wave period from one iteration to the next is less than:

   - the fraction relative change of period or
   - the fraction relative change with respect to mean value of the average mean wave period (averaged over all wet grid points)

   In the model, the relative change with respect to mean value of 0.01 was used.

c) And stops if Conditions a) and b) are fulfilled in more than some fraction percentage of wet grid points. In the model, the percentage of wet grid points was set to 98 percent.

The integration of the action balance equation was implemented in SWAN with finite difference schemes in all dimensions (time, geographic space, and spectral space). In Delft3D-WAVE, SWAN is applied in a stationary mode so that time is omitted from the equations. The stationary mode should be used in case of waves with a relatively short residence time in the computational area under consideration, i.e., the travel time of the waves through the region should be small compared to the time scale of the geophysical conditions (wave boundary conditions, wind, tides, and storm surge) (Deltares, 2011c). The stationary computational method is applied to coupling of the Delft3D-FLOW and Delft3D-WAVE modules. This setting allows flow computations to be performed with respect to time, while WAVE computations are stationary, or time invariant. Wave parameters are calculated at the moment of coupling. The stationary analysis of the wave spectrum is repeated at the specified coupling interval. Within the model, the FLOW and WAVE modules are coupled every 30 minutes.

4.4.9 Boundary Conditions

Boundary conditions represent the influence of the outer world beyond the model area which is not modeled. This model uses both external and internal boundary definitions.

4.4.9.1 Delft3D-FLOW Boundary Conditions

The computational grid domains cover only a portion of the Atlantic Ocean; therefore, it is necessary to assign boundary conditions to the model. Boundary conditions represent the influence of the outer world beyond the model area which is not modeled. The flow may be forced using water levels, currents, water level gradients, discharges, and a combination of water levels and currents. The hydrodynamic forcing can be prescribed using harmonic, astronomical components, time series, and discharge head (QH) relationships. The choice of boundary condition used depends on the phenomena to be studied; however, for a large tidal domain such as in this study, forcing by prescribing water levels only is generally a sound procedure (Deltares, 2011c). Subsequently, the domain decomposition boundaries pass the time series of data from the coarser resolution grid to the finer grids.

Flow-forcing boundaries are located far away from the area of interest. Small errors in the boundary conditions could significantly influence the model results if they are located too close to the area of interest (Deltares, 2011c). Water levels in tidal basins are a globally varying quantity, and there could be a substantial difference in water level in areas not too far apart. Therefore, boundary conditions in the domain are assigned every 10 grid cells for the overall domain.

Tidal movement results from the gravitational attraction of the moon and sun acting upon the rotating earth. The movement includes both the vertical rise and fall of the tide and the horizontal flow of the tidal currents. Although the acting forces are well understood, the resultant tidal movement is exceedingly complicated because of the irregular distribution of land and water on the earth and the retarding effects of friction and inertia (USDC, 1971).

The boundary conditions for this model are the tidal water level movement of the Atlantic Ocean. The harmonic analysis of tides is based upon an assumption that the rise and fall of the tide in any locality can be expressed mathematically by the sum of a series of harmonic terms having certain relations to astronomical conditions. A simple harmonic function is a quantity that varies as the cosine of an angle that increases uniformly with time. The general equation for the height (h) of the tide at any time (t) is written as (USDC, 1971):

$$h = H_o + A\cos(at + \alpha) + B\cos(bt + \beta) + C\cos(ct + \gamma) + \cdots \quad \text{(Equation 4.9)}$$

where:

h	=	height of the tide
H_o	=	the height of the mean water level above the datum used
$A, B, C\ldots$	=	the amplitudes of the constituents derived from observed tidal data in each locality
$at+\alpha$, $bt+\beta$, $ct+\gamma$ ...	=	the angle (phase) in which a is a constant and t represents time as measured from initial epoch

Each cosine term is known as a constituent or component tide. The expression in parentheses is a uniformly varying angle and its value at any time is called its phase. Any constituent term has its maximum positive value when the phase of the angle is zero and a maximum negative value when the phase equals 180 degrees, and the term becomes zero when the phase equals 90 degrees or 270 degrees. The coefficient of t represents the rate of change in the phase and is called the speed of the constituent and is usually expressed in degrees per hour (USDC, 1971).

At each location, water level was prescribed by specifying the tidal constituents in terms of amplitude and phase. These constituents are used at the open boundaries of the numerical model. The ten tidal harmonic constituents are as follows:

- Principal lunar semidiurnal constituent (M2)
- Principal solar semidiurnal constituent (S2)

- Larger lunar elliptic semidiurnal constituent (N2)
- Lunisolar semidiurnal constituent (K2)
- Lunar diurnal constituent (Kl)
- Lunar diurnal constituent (O1)
- Solar diurnal constituent (Pl)
- Larger lunar elliptic diurnal constituent (Ql)
- Lunisolar fortnightly constituent (MF)
- Lunar monthly constituent (MM)

Tidal harmonic constituents extracted at locations at the open boundaries of the coarse domain are from the TPXO7.2 tidal data solution (OSU, 2012). TPXO7.2 is the current version of a global model of ocean tides, which best fits, in a least-squares sense, the Laplace tidal equations and along track-averaged data from TOPEX/POSEIDON and Jason altimeters on TOPEX/POSEIDON tracks since 2002. (TOPEX is the Topography Experiment for Ocean Circulation and the TOPEX/POSEIDON mission is the Joint U.S. National Aeronautics and Space Administration (NASA)–French orbital mission to track sea level height with radar altimeters.)

The methods used to compute the model are described in detail by Egbert et al. (1994) and further by Egbert and Erofeeva (2002). The tides are provided as complex amplitudes of earth-relative sea surface elevation for eight primary (M2, S2, N2, K2, K1, O1, P1, Q1), two long period (MF, MM) and three nonlinear (M4, MS4, MN4) harmonic constituents, on a 1,440 by 721, ¼-degree resolution, full global grid. Each latest version (7.2) of the TPXO model is of better quality compared to the earlier versions since it assimilates longer satellite time series, more data sites are included into assimilation, bathymetry is improving from version to version, and resolution of global and local grids is improving from version to version.

4.4.9.2 Delft3D-WAVE Boundary Conditions

The coarsest WAVE grid in the model has open boundaries along the north and east boundaries. The west and south boundaries are land boundaries. The initial conditions of the Delft3D-WAVE runs were set such that the significant wave height, peak wave period, and wave direction are prescribed along the east model boundary only. Wave boundary conditions are considered calibration parameters and are determined through comparison to ambient observed wave parameters at monitoring stations.

Wave Direction – A uniform wave direction is imposed on the boundary. Wave buoy data for the historical storms in the model calibration suggest the primary direction of waves is 90 degrees (NOAA, 2013b). A uniform wave boundary simplifies the conditions supplied to the model allowing for recreation of the observed 90-degree wave direction.

Significant Wave Height – A uniform significant wave height is imposed on the east boundary. The significant wave height of one meter is prescribed, and the calibration of the significant wave height from historical storms best responds to this initial boundary condition.

Wave Period – A uniform significant wave period is imposed on the east boundary. The wave period of 10 seconds is prescribed, and the calibration of the wave period from historical storms best responds to this initial boundary condition.

Directional Spreading – The default cosine power (*s*) of the Delft3D-WAVE module is 4. Larger values of *s* correspond to very narrow directional spectrums (Deltares, 2011e). A uniform directional spreading is imposed on the east boundary. The directional spreading of 4 is prescribed, and the calibration of the wave parameters from historical storms best responds to this initial boundary condition.

4.4.10 Antecedent Water Level

The AWL includes the 10 percent exceedance high (or low) tide and the sea level rise due to climate change. In summary, EL +5.09 ft-PSL Datum (EL +1.74 ft-NAVD88) is adopted as the 10 percent exceedance high tide, and EL -0.32 ft-PSL Datum (EL -3.67 ft-NAV88) is conservatively adopted for use as the 10 percent exceedance low tide (see section below for derivation). The 30-year, second-order, nonlinear-trend sea level rise of 0.2 ft at the Fernandina Beach tide station is adopted for use in the storm surge analyses tide (see section below for derivation). These values will represent the AWL condition in the numerical model simulations:

- AWL_{high} (10 percent exceedance high tide + sea level rise) = EL +5.29 ft-PSL Datum = EL +1.94 ft-NAVD88
- AWL_{low} (10 percent exceedance low tide, neglect sea level rise) = EL -0.32 ft-PSL Datum = EL -3.67 ft-NAVD88

4.4.10.1 10 Percent Exceedance High and Low Tides

The AWL for storm surge estimations should include the 10 percent exceedance high spring tide, including initial rise. The 10 percent exceedance high spring 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. For locations where the 10 percent exceedance high spring tide is estimated from observed tide data, a separate estimate of initial rise (or sea level anomaly) is not necessary (NRC, 2013a).

Drawdown may be an issue when safety-related structures and equipment (e.g., UHS intakes) depend on water sources where storm surge or seiche may affect the availability of water (NRC, 2013a). Therefore, the 10 percent exceedance low tide is estimated for the evaluation of low water effects using the same methodology described to calculate the 10 percent exceedance high tide.

Long-term records of measured tidal levels are available at Virginia Key, Florida; Lake Worth Pier, Florida; and Trident Pier, Florida. The period of record reported from each station was considered and evaluated to determine conservative and appropriate exceedance levels. The best estimate of the 10 percent exceedance high tide and 10 percent exceedance low tide was from the station at Lake Worth Pier, which is approximately 52.2 miles south of PSL. A tidal epoch is 21 years. Although this station contains only 16 years of observed tidal data, it is considered reasonable to use 21 years of resynthesized tidal data to estimate the 10 percent exceedance high and low tides (ANS, 1992; NRC, 2013a). Locations of the tide stations used in these analyses are presented on Figure 4-9.

The monthly maximum values from the astronomic tidal signal reconstructed from the harmonic constituents were compared to the maximum monthly values from 16 years of observed tidal elevations to quantify the effects of sea level anomalies. Sea level anomaly is the departure of the observed tide from the astronomic tide. The cumulative density function presented on Figure 4-10 demonstrates that the sea level anomaly at Lake Worth Pier at the 10 percent exceedance level (indicated by the red dashed line) is computed to be 0 m. The calculated results for the 10 percent exceedance high and low tides are presented in Table 4-3.

The 10 percent exceedance high tide equals EL +5.09 ft-PSL Datum (EL +1.74 ft-NAVD88), and the 10 percent exceedance low tide equals EL -0.32 ft-PSL Datum (EL -3.67 ft-NAVD88). These exceedance thresholds represent part of the initial water level condition in the numerical model simulations.

4.4.10.2 Sea Level Rise

Measured tidal levels indicate that global sea level rise is occurring; however, there is no scientific consensus on the causal mechanisms and the long-term projections of sea level rise. Most of the debate related to long-term climate change is based on the argument that the global surface temperature has been increasing at an accelerated rate over the last few decades. Sea level rise is monitored and reported by the NOAA NOS, the U.S. Global Change Research Program, and the Intergovernmental Panel on Climate Change (IPCC) and should be included in PMF analysis for coastal sites (IPCC, 2007).

The IPCC defines climate change as a change in the state of the climate that can be identified by detecting changes in the mean and/or the variability of its properties that persists for an extended period, typically decades or longer. The IPCC's definition of climate change includes changes because of both natural variability and human activity (NRC, 2011), where the natural variability is the combined effect of water level change and land subsidence.

Sea level rise due to climate change is considered in determination of AWL conditions. Observed sea level rise data at local tide stations are extrapolated to estimate future sea level rise. Linear and second-order statistical trends are estimated out to 100 years in the future. At PSL, the sea level rise estimate for the remaining license life (30 years, out to 2043) is adopted as a contributing factor to the AWL.

For PSL, parameters to estimate sea level rise are determined from the NOAA tidal gage stations. Measurements at any given tide station include both global sea level rise and vertical land motion, such as subsidence, glacial rebound, or large-scale tectonic motion (NOAA, 2013f). NOAA maintains several tidal gage stations along the Atlantic Ocean shoreline near PSL. However, for the NOAA tidal data to be usable in predicting sea level rise, a long record of data must be available. The long-term sea level rise was derived for the expected life of the nuclear power plant (in the case of PSL, 30 years [NRC, 2013c]). The NOAA stations used to develop water level trends are shown in Table 4-4.

Two approaches were used for estimating sea level rise at PSL. The first approach fitted a linear trendline to monthly MSL tidal gage data using linear regression. The second approach fitted a nonlinear second-order trendline to monthly MSL tidal gage data using nonlinear regression. The linear trend model is based on NOAA's approach (NOAA, 2013g) to estimating sea level rise using the mean monthly sea level record and fitting a linear trend model to the data. The nonlinear second-order trend model is based on Walton (2007) who used a second-order polynomial for projecting sea level rise in Florida.

The stations used to estimate sea level rise are (Figure 4-9):

- Virginia Key (Station ID 8723214) (NOAA, 2013h)
- Lake Worth Pier (Station ID 8722670) (NOAA, 2013i)
- Trident Pier (Station ID 8721604) (NOAA, 2013j)
- Fernandina Beach (Station ID 8720030) (NOAA, 2013k)

The projected 30-year and 100-year sea level rises for the linear and nonlinear models are presented in Table 4-4 for all the stations analyzed. There is a limited amount of data (16 years) gathered over a nonconsecutive period for Lake Worth Pier. This is likely to yield inaccurate results. Based on findings from studies involving projected sea level rise in Florida, Virginia Key and Trident Pier stations have an insufficient amount of data to project 30-year and 100-year trends (Walton, 2007). Therefore, the 30-year and 100-year linear trend models were based on 43 years of data from the Fernandina Beach tide station. The sea level rise is estimated to be:

- 0.20 ft for the 30-year linear trend
- 0.25 ft for the 30-year nonlinear second-order trend
- 0.66 ft for the 100-year linear trend
- 0.91 ft for the 100-year nonlinear second-order trend

For the surge mechanisms evaluated herein, a sea level rise of 0.20 ft is used (30-year linear trend). The scatter plot monthly tide data, linear trendline, and resultant estimated sea level rise are provided on a plot as Figure 4-11.

4.4.11 Parameter Calibration

The numerical model is calibrated to near- and far-field tidal records as well as historical hurricane surge observations. The model parameters are adjusted until the simulated water levels are close to observed tidal elevations, surge elevation levels, and wave characteristics. The calibration is performed in three steps:

1. Calibrate the model to resynthesized tidal signals at representative stations near and far from PSL.

2. Calibrate the model to a historical hurricane (Hurricane Irene) using wind and pressure forcing (hurricane) as well as astronomic forcing (tides).

3. Validate the model by simulating additional historical hurricanes (Hurricanes Floyd, Frances, and Jeanne) using wind and pressure forcing (hurricane) as well as astronomic forcing (tides).

The tidal calibration demonstrates a strong correlation between simulated and observed tidal signals at representative stations. Particular attention is given to tide stations along the Florida eastern coast. The tidal calibration demonstrates that the model bathymetry (grid structure, resolution, and depth) accurately represents the Gulf of Mexico and Atlantic Ocean.

The hurricane calibration similarly shows a strong correlation between simulated and observed peak surge elevations. Storm surge records along the Florida eastern coast are reproduced by generating hurricane wind fields for historical hurricane events based on NOAA best track data (NOAA, 2013l).

A review of historical hurricanes is conducted to determine which storms would yield the calibration that is most representative of model performance. Hurricanes Floyd, Frances, Jeanne, and Irene are selected based on the magnitude of the storm energy (central pressure) and therefore the magnitude of observed surge elevations, the close proximity to PSL with which these storms passed, and the availability of surge and wave observations.

Discrete observations of maximum surge during Hurricane Floyd are given by NOAA (2000). Surge observations for Hurricane Frances are given by NOAA (2004a), Wang and Manausa (2005a), and NOAA (2004c). Surge observations for Hurricane Jeanne are given by NOAA (2004b), NOAA (2004d), and Wang and Manausa (2005b). Surge observations for Hurricane Irene are given by NOAA (2011b). The model parameters of wind drag coefficient, Manning's roughness, JONSWAP bottom friction coefficient, and the depth-induced breaking gamma parameter were identified as calibration parameters for storm surge. Parameter values are selected based on calibration to Hurricane Irene. The selected parameter values are validated through comparison to observations of maximum surge for Hurricanes Floyd, Jeanne, and Frances.

4.4.11.1 Tidal Calibration

Matching observed to simulated tidal water levels validates the deep ocean bathymetry (Provost and Lyard, 2003). In deep water, the speed and direction of propagation of the tides are primarily controlled by the ocean bathymetry.

Water level results at select tide stations are calibrated to resynthesized tidal water elevations to ensure that the model is correctly modeling the tidal forcing and the propagation of long period waves throughout the simulation domain. A two-month simulation is run using only tidal forcing along the model domain open boundaries and astronomic forcing of the water column to simulate tides throughout the overall domain. The calibration was carried out by manually adjusting the bottom roughness Manning's n coefficient. Predicted time series tidal WSELs are compared with resynthesized tidal elevations. The objective functions root mean square error (RMSE) and Nash and Sutcliffe Model Efficiency (NSE) (Nash and Sutcliffe, 1970) are calculated to objectively demonstrate good model performance. The NSE provides a quantitative measure of model performance. Values closer to 1 suggest better model performance. An NSE value of 0 suggests the predictive power of the model is equal to a model that simply reproduces the average of the observed time series. An NSE value of 1 suggests an ideal model that has no error in reproduction of observed data. Values between 0 and 1 suggest a model that has some predictive power but is not ideal. Negative values mean the model has no predictive power.

Eleven tide stations (Figure 4-12) are used as observation points for the model calibration. The tidal signals used in the model are represented by resynthesized tidal constituents, as obtained by the International Hydrographic Office (IHO) through Deltares (Deltares, 2012), which maintained harmonic constituent data for tide stations around the world until the year 2000. Tidal water level is computed through a resynthesis of the historical tidal constituents at each station used in the model domain. Resynthesis of a tidal signal involves reconstruction of the time series from the individual tidal constituents. Tide stations contain a total of 37 historical tidal constituents from the IHO. Although some IHO tide stations have less historical tidal constituents, they may still be compared with simulation results as these tidal constituents are the dominant tidal constituents in the astronomical tide; therefore, significant resynthesized plots can be generated. These tidal constituents are composed of phase (in degrees) and amplitude (in meters) information. The tidal constituents are presented in Table 4-5.

Resynthesis of tidal data is an acceptable method for tidal calibration and has been/is currently used in the USACE and Office of Naval Research in their state-of-the-art Western North Atlantic Tidal (WNAT) model for storm surge and tide calculations.

The tidal model was run to simulate a two-month tidal cycle. The results, presented in Table 4-6, show that the tide model reproduces the resynthesized tidal water levels well. The calibrations study demonstrates that physical ocean parameters that control the propagation of long period waves (e.g., open boundary forcing and Manning's roughness coefficient) are applied correctly. The model open boundary conditions and Manning's roughness values were validated through comparison of the resynthesized historical time series tidal water levels with Delft3D simulated tidal water levels.

From the comparison of the Delft3D model results and resynthesized historical time series tidal water levels (Figure 4-13), the Delft3D model accurately reproduced the tidal water levels at the representative tidal monitoring locations. These results were yielded using a bottom roughness Manning's n coefficient of 0.02. These tidal calibrations demonstrate the ability of Delft3D-FLOW to model astronomical tides.

4.4.11.2 Calibration and Validation Hurricanes

Calibration and validation of a storm surge model are critical to the success of storm surge modeling, the defensibility of the technical approaches that are taken, and ultimately to acceptance of storm surge results. As required by NUREG/CR-7046, the parameters of a given model may need to be calibrated using data from large historical storm events and then validated on comparatively large storm events not used in the calibration. To verify the prediction capability of the coupled Delft3D-FLOW and Delft3D-WAVE model, calibration and validation are performed by comparing the Delft3D output with measured historical storm surges, significant wave heights, and wave periods for four separate hurricane storm events. As previously mentioned, Hurricanes Frances, Irene, Jeanne, and Floyd are used for the calibration and validation of the model. The paths of these hurricanes relative to PSL and the model domain are shown on Figures 4-14 (Hurricanes Jeanne and Floyd) and 4-15 (Hurricanes Frances and Irene).

It is important to note that all hurricane wind and pressure fields used to force the hydraulic model are synthetic storms intended to recreate the actual parameter conditions of historical hurricanes. Although measured observations of the storm conditions (i.e., RMW, central pressure, and maximum wind speeds) are incorporated into the meteorological inputs, the estimated geometry of the wind and pressure fields does not exactly reproduce actual conditions. As each hurricane is an approximation of the wind and pressure fields, some uncertainty in model calibration results is attributable to simplification of the hurricane forcing data.

Hurricane surge calibration was performed by reviewing a linear regression of observed peak WSELs recorded along the Florida eastern coast and simulated peak WSELs. The wind drag and bottom roughness coefficients were modified until the slope of the best-fit line through the peak regression was close to 1.

Delft3D-WAVE parameters were calibrated by comparing the time series of simulated to observed wave characteristics. A visual inspection of the model calibration was performed as opposed to calculation of an objective function.

4.4.11.2.1 Calibration Hurricane

The Delft3D-FLOW and Delft3D-WAVE model parameters were adjusted based on a calibration to Hurricane Irene (2011). Hurricane Irene was selected for calibration due to the availability and magnitude of observed surge elevations. Storm surge at a given location is affected by the track direction, the RMW, the pressure differential, the forward speed, and the wind field distribution. Although Hurricane Irene did not make landfall exactly at PSL, it was a large enough storm to cause significant surge along the coast.

Hurricane surge and wave observation locations used in this analysis represent all published data in references from NOAA for locations along the Florida eastern coast. The observation locations along the Florida eastern coast were selected for their close proximity to PSL. Several far-field surge observations are considered to verify the physical processes of the model.

Hurricane Irene is defined at its most intense point as having a minimum central pressure of 946 mbar, maximum wind speed of 90 knots (46.3 m/s), RMW of 22 nmi (AOML, 2013), and a forward speed of 21.33 knots (10.97 m/s). The paths of Hurricanes Frances and Irene in relation to the overall model domain are shown as Figure 4-15. The paths of Hurricanes Jeanne and Floyd in relation to the overall model domain are shown as Figure 4-14.

4.4.11.2.2 Calibration Results

A piece-wise linear function of wind drag and wind speed is used that is defined by discrete values for three wind velocity ranges, as described by "breakpoints." As a first estimate, the three wind drag coefficients are based on the piece-wise function, from empirical data by Vickery et al. (2009). Varying the wind drag coefficients through calibration, the values for the wind drag coefficient breakpoints are selected as follows:

- Breakpoint A = 0.00063 at 0 m/s
- Breakpoint B = 0.0025 at 25 m/s
- Breakpoint C = 0.0025 at 100 m/s (Vickery et al., 2009 relationship only shows to 60 m/s)

The calibrated values are in close agreement with the relationship from Vickery et al. (2009).

In the simulation, parameters are calibrated by varying the parameters until WSELs computed by the model closely match the results of the observed values of the historic storm. The Delft3D model parameters were adjusted based on a calibration to Hurricane Irene (2011). The simulated versus observed peak tidal surge levels for Hurricanes Frances, Floyd, Jeanne and Irene, are shown on Figure 4-16.

4.4.11.2.3 Model Validation to Historical Hurricane

Validation simulations are performed to ensure that the model performs accurately for storms other than the calibration storm. A validation of the selected model parameters was performed for Hurricanes Floyd (1999), Frances (2004), and Jeanne (2004). Hurricanes Floyd, Frances, and Jeanne were selected based on the magnitude of the storm, the close proximity to PSL with which these storms passed, and the availability of calibration data. The paths of Hurricanes Floyd, Frances, and Jeanne in relation to the model domain are presented on Figures 4-14 and 15.

The results from the Delft3D-FLOW model validation runs are presented on Figure 4-16; and the representative results from the Delft3D-WAVE model validation runs are presented on Figure 4-17.

The simulated versus observed maximum WSELs for Hurricanes Floyd, Frances, and Jeanne are shown in Tables 4-7 to 4-10. Peak surge for all values referenced to NOAA (2013m, 2013n, 2013o, 2013p, 2013q, 2013r, 2013s, 2013t) was determined by taking the maximum WSEL of the tidal time series for the period that the hurricane was in close proximity to the PSL site.

4.4.11.2.4 Summary of Model Parameters

A summary of all the final PMSS model parameters used in the Delft3D-FLOW and Delft3D-WAVE models is presented in Tables 4-11 and 4-12.

4.4.12 Probable Maximum Hurricane Model

The input hurricane to the model is developed based on the criteria presented by NWS23 (NWS, 1979). NWS23 provides a methodology for developing an idealized PMH for locations along the U.S. Atlantic seaboard and the Gulf of Mexico.

For these analyses, the NWS criteria are applied to develop the following parameters:

- Overall storm size (diameter)
- Idealized spiral-shaped hurricane wind field
  - Maximum wind velocity
  - RMW velocity
  - Radially distributed wind velocity profile
  - Inflow angles of velocity vectors
- Idealized pressure field
  - Peripheral atmospheric pressure
  - Maximum pressure drop
  - Radially distributed pressure profile
- Range of storm forward speeds
- Range of storm track directions

A series of storm tracks are selected and a suite of PMH candidate storms is analyzed in the model to determine the candidate storm that creates the highest water level surge at PSL. This storm is then designated as the PMH, and the maximum surge is referred to as the PMSS. A storm track, starting offshore from approximately 2,000 miles, will be derived from these parameters. An offshore boundary in the deep Atlantic Ocean will allow for the model to accurately capture basin-to-basin and shelf-to-basin physics, which is important in estimating high water levels that often occur well in advance of a hurricane's landfall. The time sequence of the movement of a hurricane or the hurricane track is a required input to the model. The storm track is represented in the model by a series of successive locations of the center of hurricane derived as a function of the hurricane direction (angle), forward speed, and landfall location (defined as the location where the hurricane crosses the shoreline).

4.4.12.1 Applicability of NWS23 to Present-Day Climatology

Since 1977, several intense hurricanes have made landfall on the Gulf of Mexico and Atlantic coasts. Research on the effects of El Niño/southern oscillation indicates that while El Niño conditions tend to suppress hurricane formation in the Atlantic basin, La Niña conditions tend to favor hurricane development (NOAA, 2006). Additionally, research has been performed into the relationship between the Atlantic Multi-decadal Oscillation (AMO), which is the variation of long-duration sea surface temperature in the northern Atlantic Ocean with cool and warm phases that may last for 20 to 40 years, and hurricane intensity (NOAA, 2006). It shows that hurricane activities increase during the warm phases of the AMO compared to hurricane activities during the AMO cool phases. Recent hurricane data indicate that Atlantic hurricane seasons have been significantly more active since 1995. However, hurricane activities during the earlier years, such as from 1945 to 1970, were apparently as active as in the recent decade (NOAA, 2006; Blake et al., 2007).

Blake et al. indicated that during the 35 years from 1970 through 2004, the conterminous United States was affected by the landfall of three Category 4 or stronger hurricanes: Hurricane Charley of 2004, Hurricane Andrew of 1992, and Hurricane Hugo of 1989 (Blake et al., 2007). Based on the analysis of hurricane data from 1851 to 2006, they summarized that, on average, the United States is affected by a Category 4 or stronger hurricane approximately once every seven years so, in an average 35-year period, five hurricanes make landfall at Category 4 or stronger.

The newest report by Blake et al. (Blake et al., 2011) shows a similar trend from 2004 to 2010; therefore, it is reasonable to assume that the PMH parameters derived in NWS23 are still applicable even in consideration of future climate variability. Characteristics of historical hurricanes passing in the vicinity of PSL are presented in Tables 4-13 and 4-14.

4.4.12.2 Steady-State Probable Maximum Hurricane Parameters

A summary of PMH parameters, as derived from NWS23 methodology, for each potential PMH scenario evaluated is shown in Table 4-15. These parameters are derived as described below:

1. The approximate location of PSL is shown on the map on Figure 4-18. The distance of the site (in nmi) from the U.S.-Mexico border was determined to be approximately 1,550 miles. The latitude at PSL is 27.349 degrees North.

2. The peripheral pressure (P_w) is the sea level pressure at the outer limits of the hurricane circulation and represents the average pressure around the hurricane where the isobars change from cyclonic to anticyclonic curvature. Per NWS23, the peripheral pressure (P_w) for the PMH (at the site location) is kept constant at a value of 30.12 in. Hg (1,020 mbar).

3. The central pressure is the lowest sea level pressure at the hurricane center. In general, the central pressure (P_o) increases with latitude. The central pressure of a PMH at PSL is shown on Figure 4-19. The central pressure for the PMH is given as 26.15 in. Hg (885.5 mbar).

4. The RMW is the radial distance from the hurricane center to the band of strongest winds within the hurricane wall cloud, just outside the hurricane eye. In general, RMW increases with latitude (NWS23). The RMW range for a PMH at PSL is shown on Figure 4-20. The RMW has a range of 4 nmi to 20 nmi.

5. The forward speed (T) refers to the rate of translation of the hurricane center from one geographical point to another. It is one component of the wind field of a moving storm and results in higher wind speeds on the right side of the storm and lower wind speeds on the left. The approximate range for forward speed for a PMH at PSL is shown on Figure 4-21. The NWS23 forward speed has a range of 6 knots to 20 knots.

6. The track direction is the path of forward movement along which the hurricane is coming in nautical convention (i.e., measured clockwise from north, 0 degrees). The permissible track directions are limited based on possible, naturally occurring directions over the open ocean, sea surface temperatures, forward speed, and other meteorological features (NWS23). As the angle between the coastal orientation and track direction decreases, slower hurricanes weaken more than faster-moving hurricanes. The approximate range for track direction for a PMH at PSL is shown on Figure 4-22. The track direction has a range of 70 degrees to 160 degrees. Note that the track direction defined in NWS23 is the same as the direction in Delft3D, which is defined according to nautical convention.

4.4.12.3 Historical Storm Tracks

This section describes the number of major and non-major hurricanes to strike in the region of PSL since hurricanes were first reliably recorded in 1842. A major hurricane is defined as a Category 3 or higher on the Saffir-Simpson Scale (NOAA, 2013u).

Since 1842, 27 major hurricanes (Category 3 or above) have passed within 120 nmi of PSL (NOAA, 2013v). Of these storms, 11 have made landfall along the 240 nmi of coastline centered at PSL (NOAA, 2013v). Thus, a major hurricane impacts this stretch of coast, on average, approximately every other decade. All hurricanes making landfall had either an east-to-west or southeast-to-northwest trajectory. No record was found to support a hurricane path from the north or northeast.

Large historical hurricanes (Category 3 or above) whose storm center came within 120 miles of PSL are shown on Figure 4-23. The hurricane names, dates, and categories are presented in Tables 4-14 and 4-15.

4.4.12.4 Radius of Maximum Winds – Parameter Comparison to Region-Specific Data

JLD-ISG-2012-06 (NRC, 2013a), with reference to NUREG-0800 Section 2.3, states that the hurricane climatology during the period evaluated in NWS23 with hurricanes making landfall after 1975 indicates that the NWS23 parameters for the PMH are still applicable for licensing decisions. However, a detailed site- or region-specific hurricane climatology study should be provided to show that the PMH parameters are consistent with the current state of knowledge. A region-specific hurricane climatology study is provided to support the selection of the RMW parameter.

NWS Technical Report 38 (NWS38) (NWS, 1987) provides an in-depth study of the relationship between central pressures and RMW. NWS38 analyzed the joint probability of whether hurricane size (RMW, R) and intensity (central pressure, P_o) are dependent or independent parameters. NWS38 found that hurricanes with very large radii of maximum winds (R, in excess of 45 nmi) are generally found to be of moderate or weak intensity. Also, NWS38 observed that extremely intense storms (low P_o) have low radii of maximum winds (R) because, if angular momentum is conserved, a vortex contracts in size as it increases in rotational speed. Furthermore, NWS38 concluded that more intense storms (P_o less than 920 millibars) exhibited a closer correlation between R and P_o than that exhibited by less intense storms (P_o greater than 920 millibars). A plot of P_o versus maximum R for a set of historical hurricanes demonstrates a correlated relationship (Figure 4-24, adapted from NWS38 Figure 14). The trendline plotted shows that R is generally bounded by a maximum of 15 nmi when P_o is less than 920 millibars and greater than 900 millibars. When P_o is less than 900 millibars, the trendline shows that the RMW tightens as P_o decreases.

To update the information used in NWS38, data for additional hurricanes since 1985 are analyzed. These hurricanes include:

- Gilbert (1988) from Willoughby et al., 1989
- Andrew (1992) from NOAA, 2012 and Rappaport, 2005
- Opal (1995) from Mayfield, 1995
- Mitch (1998) from Guiney and Lawrence, 2000 and NOAA, 2012
- Floyd (1999) from Pasch, 2006b and NOAA, 2012
- Isabel (2003) from Beven and Cobb, 2004 and NOAA, 2012
- Ivan (2004) from Stewart, 2005 and NOAA, 2004
- Katrina (2005) from Knabb et al., 2006a and NOAA, 2012
- Rita (2005) from Knabb et al., 2006b and NOAA, 2012
- Wilma (2005) from Pasch, 2006a and NOAA, 2012
- Dean (2007) from Franklin, 2008 and NOAA, 2012

Series of data relating P_o and R for the most intense periods of these hurricanes were collected from various sources.

The P_o versus R data for the additional hurricanes are plotted in relationship to the NWS38 plot. The envelope of P_o versus maximum R results in a linear trendline as drawn on Figure 4-24. The updated trendline shows a larger envelope for P_o versus maximum R than NWS38 but a more strongly defined P_o versus maximum R based on the linear trend exhibited. The trend shows distinctly that maximum R decreases with lower central pressure.

The range of R versus P_o, as developed from NWS23 PMH methodologies, is also plotted on Figure 4-24. Based on NWS23, the PMH candidate storms consider P_o to be 884 millibars, with a corresponding range of R from 4 to 20 nmi. This range of storms is entirely outside of the historical hurricane R-P_o envelope. Based on observed historical storms of low P_o, it is seen that R is in the lower range of the NWS23 values for R (i.e., 4 nmi); therefore, extending the RMW (R) to 20 nmi is very conservative for a storm of extremely low central pressure (884 millibars), as used in the models. Thus, it is concluded that the selection of a storm with P_o = 884 millibars and R = 20 nmi bounds observed historical parameter combinations with substantial margin.

4.4.12.5 Pressure Field Computation

The pressure profile field for each radial distance out from the center of the storm is determined using Equation 4.10 as given by NWS23. The pressure field is computed in 1 nmi increments (r) from the storm center out to 300 nmi. The pressure field is used as input to compute the storm surge associated with the lowering of the air pressure within the hurricane (i.e., the pressure setup). Linear interpolation is performed at computational nodes in between the calculated radii. Inherent to the NWS23 pressure field model, the equation assumes that the pressure field is a constant value at each radial r around the hurricane center:

$$p = p_c + \Delta p \cdot e^{\left(\frac{-RMW}{r}\right)} \qquad \text{(Equation 4.10)}$$

where:

p = sea level pressure at distance r from the hurricane center (in. Hg)
p_c = central pressure (in. Hg)
Δp = pressure differential (in. Hg) = $p_o - p_c$
p_o = peripheral pressure (in. Hg)
r = radius (nmi)
RMW = radius of maximum winds (nmi)

4.4.12.6 Overwater Wind Field Computation

The gradient winds in a hurricane blow with circular motion, parallel to the pressure isobars, in which the centripetal and Coriolis accelerations together balance the horizontal pressure gradient force per unit mass (NOAA, 1979). The NWS maximum gradient wind is defined based on Equations 4.11 through 4.13.

$$V_{gx} = K\sqrt{\Delta p} - \frac{RMWf}{2} \qquad \text{(Equation 4.11)}$$

where:

V_{gx} = maximum gradient wind speed (knots)
f = Coriolis parameter, dependent on latitude (hr^{-1}) = $14.5444 \times 10^{-4} \cdot \sin(\psi)$

K = density coefficient
Δp = pressure differential (in. Hg) = $p_0 - p_c$
RMW = radius of maximum winds (nmi)
Ψ = latitude (degrees)

The K constant was determined using:

$$K = \sqrt{\frac{1}{\rho e}} \qquad \text{(Equation 4.12)}$$

where:

$\frac{1}{\rho}$ = conversion factor for "density term" to metric units

e = Euler's constant (~2.71828)

The adjusted 10-meter, 10-minute overwater winds over open water have been found to vary from about 75 percent to over 100 percent of V_{gx} (NWS23). To estimate V_x in a stationary hurricane, NWS23 gives Equation 4.13 (NWS23 Equation 2.4).

$$V_x = V_{xs} = 0.95V_{gx} \qquad \text{(Equation 4.13)}$$

where:
V_{gx} = maximum gradient wind speed (knots)
V_x = maximum 10-meter, 10-minute wind speed (knots)

The profile of wind speed (V_s) for a stationary hurricane at a distance r (nmi) out from the center of a hurricane outside of the RMW to 300 nmi is determined using a series of ratios from NWS23. The ratios are dependent of the RMW distance from the hurricane center. Note that in the NWS23 methodology, the relative wind speed (V_s/V_{xs}) for the RMW of 4 nmi and of 6 nmi is not given for a radius extending out to 300 nmi. Therefore, it is assumed that the relative wind speed (V_s/V_{xs}) can be estimated for values out to 300 nmi by extrapolating the curves for the RMW of 4 nmi and of 6 nmi from the last known relative wind speed point out to 300 miles. The wind speeds (V_s) are tabulated against the distance r from the center of the hurricane.

The profile of wind speed (V_s) for a stationary hurricane at a distance r (nmi) out from the center of a hurricane inside of the RMW is determined using a list of ratios as given by NWS23. The wind speeds (V_s) are tabulated against the distance r from the center of the hurricane out to radial distance of the RMW at 1 nmi increments.

An asymmetry factor is added to the wind speed (V_s) for a stationary hurricane to account for the forward speed (T) of the hurricane. Equations 4.14 through 4.17 are used to account for the forward speed of the hurricane. The wind speeds (V) are tabulated against the distance r from the center of the moving hurricane. For the hurricane wind field model, wind speeds (V) are tabulated in 1 nmi (r) increments from the storm center out to 300 nmi.

$$V = V_s + A_T \qquad \text{(Equation 4.14)}$$

where:

V_s = maximum 10-meter, 10-minute wind speed for a stationary hurricane (knots)
V = maximum 10-meter, 10-minute wind speed for a moving hurricane (knots)
A_T = asymmetry factor to account for forward speed of the hurricane

$$A_T = 1.5(T^{0.63})(T_o{}^{0.37})\cos\beta \qquad \text{(Equation 4.15)}$$

where:

T = forward speed (knots)
T_o = 1 when T, V, and V_s are in knots; 0.514791 when T, V, and V_s are in m/s; 1.151556 when T, V, and V_s are in mph; 1.853248 when T, V, and V_s are in km/hr
β = difference in inflow angle between radius r and the radius of maximum winds (R) (radians) [Note: 1 degree = 0.01745329 radians, i.e., 33.5 degrees = 0.584685 rad]

$$for\ r \neq RMW,\ \ \beta = \varphi_r - \varphi_{RMW} \qquad \text{(Equation 4.16)}$$

$$for\ r = RMW,\ \ \beta = \varphi_{RMW} - \varphi_R = 0 \qquad \text{(Equation 4.17)}$$

where:

β = difference in inflow angle between radius r and the radius of maximum winds (R) (radians) [Note: 1 degree = 0.01745329 radians, i.e., 33.5 degrees = 0.584685 rad]
φ_r = inflow angle at radius r [Note that the inflow angles for values past 130 nmi on the x-axis (distance from hurricane center) are assumed to be constant using the last known values at 130 nmi]
φ_{RMW} = inflow angle at radius RMW [Note that the inflow angles for values past 130 nmi on the x-axis (distance from hurricane center) are assumed to be constant using the last known values at 130 nmi]

Wind speeds at other various degree angle positions around the hurricane center were computed using the NWS23 methodology. Wind speeds at other various angles are calculated the same; however, an extra set number of (M or θ) degrees is added to β to obtain the wind speed at β+M from north of the hurricane center. For this analysis (M = 8 degrees), wind speeds are calculated at 45 different angles (360 degrees / 8-degree segments = 45 segments) around the hurricane center. The end result is a wind field in the form of a spider web grid. A wind speed was computed for 1 nmi radial increments out from the center of the storm at 8-degree segments around the storm.

The analysis requires that the wind speed at each location on the spider web grid has an associated wind direction (ξ) clockwise from north (0 degrees) to be input into the Delft3D program. The wind direction (ξ) at any location on the spider web grid is computed by trigonometry using Equation 4.18. For example, at radius (r) 50 nmi, degree segment (M = θ) 140 degrees, the inflow angle (ϕ) is 20 degrees. The wind direction at this location of the spider web grid is computed to be 140 degrees + 90 degrees – 20 degrees = 210 degrees.

$$\xi = \theta + 90 - \varphi \qquad \text{(Equation 4.18)}$$

where:

ξ = wind direction from north (degrees)
θ = M = degree segment around the hurricane center (with respect to north, 0 degrees)
φ = inflow angle (degrees)

Examples of the pressure and wind fields are shown on Figures 4-25 and 4-26, respectively.

4.4.13 Storm Surge Computations

The hurricane wind and pressure fields are created using the NWS23 (NWS, 1979) wind and pressure field model for describing the hurricane. The NWS23 model was used to develop a suite of synthetic storms. NOAA NWS23 defines the PMH as a hypothetical steady-state hurricane with a combination of meteorological parameters that will give the highest sustained wind speed that can occur at a specified coastal location. Actual hurricane wind fields are complex but can be well represented by a small range of parameters, which are storm intensity (central pressure), forward speed, storm size, RMW, and track direction. These parameters were used to conduct a screening approach at PSL to determine the most critical hurricane that will produce the worst case storm surge.

The screening for the critical hurricane was conducted as follows:

1. Hold central pressure and peripheral pressure to the PMH criteria of 26.15 in. Hg and 30.12 in. Hg, respectively.

2. Synchronize the timing of the maximum storm surge with the incoming high tide for the coarse FLOW grid model.

3. Compute the difference between the ambient WSEL at landfall and the 10 percent exceedance high tide plus sea level rise. This difference will be added to the final WSEL.

4. Test different track directions within the range of 70 to 160 degrees with the coarse FLOW grid model.

5. Test different RMW within a range of 4 to 21 nmi with the coarse FLOW grid model. NWS23 prescribes an RMW range of 4 to 20 nmi; however, 21 nmi was examined for parameter sensitivity.

6. Test different forward speeds within the range of 6 knots to 20 knots with the coarse FLOW grid model.

7. Test different track striking positions relative to the site location with the coarse FLOW grid model. Striking position notation is given such that 0 is landfall at 1 RMW south of PSL. A value of -1 is landfall 2 RMW south of PSL. A value of +1 is landfall at PSL.

8. Determine the critical combination of hurricane parameters from above coarse FLOW grid screening.

9. Run the critical combination of hurricane parameters (PMH) to determine the PMSS with coupled Delft3D-WAVE and Delft3D-FLOW utilizing four FLOW grids and five WAVE grids.

10. Run the PMH with Delft3D-FLOW only to determine the influence of waves in conjunction with the water flow on the peak storm surge. The influence of waves adds wave setup, thereby increasing the total storm surge height.

11. Run two scenarios, utilizing both Delft3D-FLOW and Delft3D-WAVE with and without a breach of the dune separating PSL from the Atlantic Ocean, to determine the influence of the beach dune on the PMSS water level at PSL.

12. Run Delft3D-FLOW with the critical combination of hurricane parameters, except with a forward speed of 6 knots to determine the bounding case for flood duration.

Details of each Delft3D scenario are summarized in Table 4-15.

4.4.14 Storm Surge Results

4.4.14.1 Probable Maximum Hurricane Parameters for Probable Maximum Storm Surge

After evaluation of the suite of candidate PMH storms, the critical PMH parameters producing the maximum storm surge are:

- A central pressure of 885.54 millibars (26.15 in. Hg).
- A RMW of 19 nmi.
- A storm center at a distance of 0.875 RMW south of PSL.
- A forward speed of 20 knots.
- A track direction of 70 degrees.
- The components of storm surge are presented graphically on Figure 4-27. The critical PMH combination of storm parameters produces a storm surge of EL +18.3 ft-PSL Datum (EL +14.9 ft-NAVD88), not including wave runup. This is the total water level height, including 10 percent exceedance high tide, 30-year sea level rise (0.20 ft), wind and pressure setup (12.36 ft), and wave setup (0.66 ft). These results are presented graphically on Figure 4-27.
- The PMSS inundation area is presented on Figure 4-28. The Units 1 and 2 Powerblock area, the ISFSI, and the FLEX building are not inundated during the PMSS.
- Examining both cases with and without the beach dune, the peak WSEL time series at the PSL reactor observation point is not affected by the presence of a dune breach. The source of flooding is primarily inflows from Big Mud Creek; therefore, the breached dune has a minimal impact on the PSL site flooding.

4.4.15 Sensitivity of Flood Duration to Forward Speed

The flood duration was defined as the time the WSEL exceeded the top of the dike along the PSL intake canal, which is EL +3.219 m-MSL (EL +13 ft-PSL Datum). To determine the bounding case for flood duration, the PMSS was run with forward speeds of 6 knots and 20 knots. A conservative lower bound of 6 knots was selected.

The candidate PMH with longest flood duration is bounded by a hurricane with the following parameters

- A central pressure of 885.54 millibars (26.15 in. Hg)
- A RMW of 19 nmi
- A storm center at a distance of 0.875 RMW south of PSL
- A forward speed of 6 knots
- A track direction of 70 degrees

The final water levels include the 10 percent exceedance high tide and sea level rise to properly calculate the flood duration. As demonstrated on Figure 4-29, the flood duration is bounded by the case of a 6 knot forward speed with a flood duration of 7 hours. The time to peak is 3 hours. The recession time is 4 hours. The slower forward speed was simulated with Delft3D-FLOW only.

4.4.16 Coincident Wind-Wave Runup

Wave runup is evaluated for the spectrum of waves that can potentially impact PSL coincident with the PMSS event. The evaluations follow the guidance provided in ANSI/ANS-2.8-1992 Section 7.4 – Wave Action. Calculations were performed based on methodologies and equations in USACE (USACE 1984b and 2011).

4.4.16.1 Still Water Level for Computing Wave Runup

The SWL is the PMSS water level. The PMSS SWL is EL +18.3 ft-PSL Datum (EL +14.9 ft-NAVD88) and includes the effects of 10 percent exceedance high tide, 30-year sea level rise (0.20 ft), wind and pressure setup (12.36 ft), and wave setup (0.66 ft).

4.4.16.2 Wave Characteristics

The PMH creating the bounding case for PMSS (Section 4.4.14) is used to generate wave characteristics across the PSL site. Figures 4-30 and 4-32 present the significant wave height. The Delft3D model results are presented for the case where a breach of the dune in front of PSL was considered. The sand dune directly in front of PSL was reduced to the elevation at the toe of the dune. The results of significant wave height and peak WSEL were reviewed to determine if the presence of the dune would affect the simulation results.

As demonstrated on Figure 4-30, the dune breach does not affect the significant wave height at the PSL reactor block. Figure 4-30 represents the time of the peak significant wave height. The source of flooding is primarily inflows from Big Mud Creek; therefore, the breached dune has a minimal impact on the PSL site flooding. The values for all wave parameters at maximum storm surge are summarized in Table 4-16.

4.4.16.3 Wave Runup Inputs

Observation Points 1 through 6 (Figure 4-31) were selected for the characterization of varying wave conditions at different locations at and near the site. The primary wave approach direction is from the Atlantic Ocean (and discharge canal). Due to the wave directions, unrefracted wave approach between the northwest and southeast is most likely. The west, southwest, and south sides of PSL, on the side of the intake canal, are very unlikely to experience direct wave attack. Based on the POI configuration on Figure 4-6 and these predominant wave approaches, only Doors 12 (FH-001), 33 (DG-001), and 35 (DG-001) on the eastern side of the Powerblock are vulnerable to direct wave runup. The other doors are shielded sufficiently from direct wave runup by large impeding structures.

The PMSS WAVE results (Figure 4-32) show the wave direction is primarily western. The waves enter the PSL site through the breached dune (Section 4.4.14.1). One hundred percent of the waves then break in front of the PSL site (Figure 4-32). The following parameters from the PMSS surge and wave analysis were used in the calculation of wave runup:

D_1 = depth of water at the point of breaking waves = 3.86 m + 0.305 m = 4.165 m

D_2 = depth of water at PSL site
Peak WSEL = 18.3 ft-PSL Datum < plant elevation = 18.5 ft-PSL Datum

H_b = significant wave height of breaking waves = 0.79 m

λ = wavelength of breaking waves = 7 m

L = runup distance to PSL reactor = 450 m

The maximum possible wave height at the PSL Powerblock is estimated from the peak depth of water 0 m.

4.4.16.4 Wave Runup Results

Due to the configuration of the PSL Powerblock POIs (Figure 4-6) and the mean directions of PMSS wave approach (Section 4.4.16.3), no direct wave runup is expected at the POIs. Overtopping of the steel sheet-piling barrier can occur at the nose of the discharge canal, but this overtopping discharge volume (5.0×10^{-4} $ft^3/s/ft$) is deemed insignificant by USACE (2011).

4.4.17 Probable Maximum Storm Surge Maximum Water Level

The components of the total surge with wave runup can be summarized as follows:

• 10 Percent Exceedance Tide	EL +5.09 ft-PSL Datum (EL +1.74 ft-NAVD88)
• 30-Year Sea Level Rise	+0.20 ft
• Wind and Pressure Setup	+12.36 ft
• Wave Setup	+0.66 ft
• PMSS	EL +18.3 ft-PSL Datum (EL +14.9 ft-NAVD88)

4.5 Seiche

Seiches are standing waves on a body of water whose period is determined by the resonant characteristics of the containing basin. The water body has a set of natural periods of resonance (or modes), called Eigen periods. When external forces are applied, the water body responds by oscillating at its Eigen period until the energy dissipates through mechanisms such as friction or exiting the system. Forces that could potentially drive a seiche include diurnal, meteorological, storm, wave, and seismic forces.

There have been no observed seismic seiches at Indian River. No documentation of seiches is found for other forcing mechanisms; however, evaluations are performed for seiches due to diurnal atmospheric forcing (sea breeze), wind forcing (PMSS), wave/current forcing, seismic forcing, and meteorological forcing at Indian River.

Seiches occur in all bodies of water to some degree but are immeasurable in smaller water bodies due to their small magnitude. To determine if a significant seiche will occur, the Eigen period and the period of the applied forces were analyzed. Eigen periods are intrinsic to a body of water and depend on the geometry of the enclosing basin and the depth of the basin.

The domain considered for the Eigen periods of oscillation is the Indian River Lagoon, from Port Salerno in the south to Ft. Pierce in the north (Figure 4-33). Based on the geometric characteristics of the Indian River (lagoon), the natural oscillation periods of the lagoon are calculated using Merian's two-dimensional formula (Equation 4.16) (Ichinose et al., 2000) for three different water levels: MSL, 10 percent exceedance high tide level, and 10 percent exceedance high tide level plus sea level rise after 30 years. The computed Eigen periods were contrasted with the periods obtained for the wind speeds from meteorological stations with subhourly records.

$$T_{nm} = \frac{2}{\sqrt{gh}}\left[\left(\frac{n}{L_x}\right)^2 + \left(\frac{m}{L_y}\right)^2\right]^{-1/2} \qquad \text{(Equation 4.19)}$$

where:

T_{nm} = Eigen period (s)
L_x = length of basin in x-direction [m]
L_y = length of basin in y-direction [m]
n = number of nodes in the standing wave in x-direction (n = 1, 2, 3, ...)
m = number of nodes in the standing wave in y-direction (m = 1, 2, 3, ...)
g = gravity acceleration (9.806 m/s^2)
h = depth of the body of water (m)

The Indian River Lagoon Eigen periods computed with Merian's formula are shown in Table 4-17.

Wind data were obtained from fourteen meteorological stations and one buoy. Ten of the stations were from NOAA-NCDC (NOAA, 2014a), four stations belong to the Florida Automated Weather Network (FAWN, 2014), and the buoy belongs to the NOAA-National Data Buoy Center (NDBC) (NOAA, 2014b). All of the NOAA stations have at least hourly information; the FAWN stations have subhourly information as do the buoy data. Station locations are shown on Figure 4-34, and the station records are shown on Figure 4-35.

The wind data were processed to establish the characteristics of the wind in terms of speed and direction. As a general rule, the wind blows from the east at between 70 to 110 degrees for all of the stations. There is a second direction peak that is generally at around 300 to 340 degrees, or the northwest direction. That means the preponderant wind directions align somewhat well with the general direction of the Indian River Lagoon. The subhourly data of the FAWN stations and the buoy (NOAA-NDBC) correlate well with the hourly data from NOAA-NCDC stations (Figure 4-36).

The mean wind speeds are in the range of 1.7 to 5 m/s. The maximum values were observed at a buoy 20 nmi to the east of Cape Cañaveral, while the minimum values were observed in the stations located more inland. As is common for wind speed data, the data also appear to follow a Weibull distribution.

To verify any correlation of the Indian River Lagoon response with the forcing meteorological variable in the same frequency, spectral analysis of historical wind data in the region was performed using MATLAB fast Fourier transform (FFT).

An FFT is an algorithm to compute the discrete Fourier transform (DFT) and its inverse, where the result is the DFT of the input function. A Fourier transform converts time (or space) to frequency and vice versa; an FFT rapidly computes such transformations, where the Fourier series decomposes periodic functions or periodic signals into the sum of a (possibly infinite) set of simple oscillating functions, namely sines and cosines with different amplitudes and phases. The amplitude and phase of a sinusoid can be combined into a single complex number, called a Fourier coefficient. The real part is the amplitude of the cosine sub-component; the imaginary part is the amplitude of the sine sub-component. This is based on Euler's equation $e^{i\theta} = \cos\theta + i\sin\theta$, where $i = \sqrt{-1}$. For example, in $a + bi$, a is the amplitude of the cosine sub-component, and b is the amplitude of the sine sub-component. The only requirement of this algorithm is that the number of points in the series be a power of 2.

Wind velocity values from several weather stations (Figure 4-36) were used to determine the period of external wind forcing. Wind data from the weather station from the time frame of 2008 through 2012 were used with a recording interval of 10 minutes. The wind speed data were analyzed to determine the frequency of the dataset through statistical methods. A spectral analysis using the Fourier analysis to determine the frequency outliers of the data was used. The Fourier analysis was applied to the wind speed to generate the frequency and magnitude of the dataset. The FFT frequency was determined from the given sample size of the dataset. The frequency outliers of the FFT method were then converted to the period and compared to the Eigen period. Only the FAWN stations and the buoy have wind data for 15 minute and 10 minute intervals, respectively. For this reason the FFT analysis is only carried out for them. The corresponding Nyquist frequencies, i.e., the highest frequency about which meaningful information can be extracted, are half the sampling rate, i.e., 0.167 hr^{-1} for the FAWN stations and 0.25 hr^{-1} for the buoy. The results are shown on Figures 4-37 and 4-38.

From Figures 4-37 and 4-38, it can be seen that the common periods for the wind speeds at the five stations considered cluster around 0.75, 1.0, 1.25, 1.75, and 2.25 hours. These periods do not coincide with any of the periods presented in Table 4-17; therefore, no seiche is likely to occur in the Indian River Lagoon.

Calculations and observations conclude that seiche is not a threat to PSL.

4.6 Tsunami

This subsection examines the tsunamigenic sources and identifies the probable maximum tsunami (PMT) that could affect the safety-related facilities of PSL. The analytical approach follows the PMT evaluation methodology proposed in NUREG/CR-6966. It evaluates potential tsunamigenic source mechanisms, source parameters, and resulting tsunami propagation from published studies and estimates tsunami water levels at the site based on site-specific numerical model simulation results. Historical tsunami events recorded along the Florida coast are reviewed to support the PMT assessment.

The plant grade is EL +18.5 ft-PSL Datum (EL +15.15 ft-NAVD88). As the plant grade and elevations of SSCs are higher than the maximum water level runup of tsunami events, tsunamis are not expected to pose any hazard to SSCs of PSL, as described in the subsections below.

When referring to surge height, it is important to clarify the usage, as well as the reference point for measuring. The surge height is considered the total water level height, including astronomical tides, wave setup, and, if specified, sea level rise, above the MSL datum unless specifically stated to be in NAVD88 or PSL Datum.

4.6.1 Historical Tsunami Record

Records of historical tsunami runup events along the U.S. Atlantic coast near PSL were obtained from the National Geophysical Data Center (NGDC) tsunami database (NGDC, 2014). The NGDC database contains information on source events and runup elevations for tsunamis worldwide from approximately 2000 B.C. to the present time. The literature search was conducted to examine historical tsunami sources, observed tsunami runups, and potential tsunami sources that have not yet triggered a tsunami but are plausible.

Tsunami sources were characterized according to their specific generation characteristics (i.e., landslides, volcanoes, earthquakes), and each was examined to determine whether it was a credible source for flooding at PSL. A tsunami was considered to be a credible threat to PSL if the runup height could exceed the beach dune crest at PSL (the tsunami runup threshold). However, for conservatism, any source capable of producing tsunami amplitudes greater than 3.28 (1 m) was considered as a tsunami source. This approach allows for the analysis of potential PMT events while eliminating the minor tsunamigenic sources.

The dune crest elevation ($z_{dune\ crest}$) at PSL was used to evaluate whether a tsunamigenic (i.e., capable of producing a tsunami) source could produce a PMT at PSL. If the tsunami runup ($R_{tsunami}$) above the 10 percent exceedance high tide ($z_{10\%\ high\ tide}$) was less than the dune crest elevation ($z_{dune\ crest}$), then the runup could not reach safety-related SSCs at PSL:

$$z_{dune\ crest} > R_{tsunami} + z_{10\%\ high\ tide} \qquad \text{(Equation 4.20)}$$

where:

$z_{dune\ crest}$ = elevation (ft-NAVD88) of the dune crest at PSL
$R_{tsunami}$ = tsunami runup (ft)
$z_{10\%\ high\ tide}$ = elevation (ft-NAVD88) of the 10 percent exceedance high tide

Rearranging Equation 4.20, the critical threshold for tsunami runup ($R_{tsunami}$) was determined:

$$R_{tsunami} < z_{dune\ crest} - z_{10\%\ high\ tide} \qquad \text{(Equation 4.21)}$$

where:

$R_{tsunami}$ = tsunami runup (ft)
$z_{dune\ crest}$ = elevation (ft-NAVD88) of the dune crest at PSL
$z_{10\%\ high\ tide}$ = elevation (ft-NAVD88) of the 10 percent exceedance high tide

4.6.1.1 Summary of Potential Sources for Probable Maximum Tsunami

Three potential probable PMT sources were determined to pose credible danger to PSL (Figure 4-39):

1. An earthquake along the Marques de Pombal Fault near Spain
2. Earthquakes along the Puerto Rico and Hispaniola Trenches in the northern Caribbean
3. Landslides near Cape Fear and Cape Lookout along the U.S. east coast

4.6.2 Tsunami Analysis

Tsunami propagation and the effects of nearshore bathymetric variation at the Florida Atlantic coast were simulated in a two-dimensional computer model. The PMT simulation uses the Delft3D-FLOW computer program (Deltares, 2011c) for most of the analyses. The numerical model developed and calibrated for the PMSS analysis is used for the PMT analysis. The two-dimensional (depth averaged) or three-dimensional nonlinear shallow water equations were solved in Delft3D-FLOW. These equations are derived from the three-dimensional Navier-Stokes equations for incompressible free surface flow.

4.6.2.1 Probable Maximum Tsunami Hazard

The PMT was determined by evaluating the tsunami sources identified in Section 4.6.1.1. The period of record for tsunami events along Florida is approximately 120 years, which results in significant undersampling of the tsunami distribution (USGS, 2008). An alternate approach is to follow the Probabilistic Tsunami Hazard Analysis (PTHA) described by Senior Seismic Hazard Analysis Committee (SSHAC, 1997). Within this method, geologic source parameters driving tsunami sources (earthquake seismic moment, landslide volume, etc.) are evaluated probabilistically based on available literature. If appropriate, these seismic source parameters are then scaled to the appropriate threshold based on the desired annual exceedance probability.

The geologic source parameters are then translated into a deformation of the WSEL (Dababneh et al., 2012). A deterministic model (Delft3D) is then used to evaluate the water surface deformation and resulting tsunami wave at PSL for all potential tsunami sources.

4.6.2.2 Earthquake Probability

The probability of each earthquake event was based on moment magnitude (M_w) frequency plots presented in USGS (2008) and Bird and Kagan (2004). These references present the annual exceedance probability of an earthquake of size M_w occurring along a given fault. As these estimates account for magnitude as well as timing, no further statistical analysis is required.

To scale earthquake parameters to the 10^{-6} threshold, the dimensions of the fault were increased for some scenarios to reach the appropriate M_w value.

4.6.2.3 Landslide Probability

Landslide-generated tsunamis are considered to be roughly one order of magnitude less probable than an earthquake-induced tsunami (USGS, 2008).

USGS (2008) showed that the size distribution of landslides along the U.S. Atlantic coast is best represented by a log-normal distribution. Based on this empirical distribution, the probability of a landslide volume exceeding the value is given as "Source Parameter P."

USGS (2008) suggested that the occurrence of submarine landslides could be correlated with the occurrence of near-field earthquakes. USGS (2008) suggested that submarine landslides could occur for earthquakes of magnitude $M_w \geq 7$. The mean return time of an event of this magnitude is between 600 and 3000 years for the U.S. Atlantic coast.

The probability that a landslide tsunami of volume V occurs was estimated as:

$$Pv = P(E) \cap P(L) = P(E) * P(L) \qquad \text{(Equation 4.22)}$$

where:

Pv = probability that a landslide of Volume V occurs in a given year
P(E) = probability that an earthquake $M_w \geq 7$ occurs in a given year
P(L) = probability that the resulting landslide has a volume V or greater

4.6.2.4 Tsunami Modeling

As recommended in USGS (2008), a numerical model (Delft3D) was used to determine the effects of tsunami sources on WSEL at PSL.

Representation of tsunami-inducing events has been performed through several different methodologies. Dababneh et al. (2012) and Mader (2001a) suggest sources within the model domain may be represented as an instantaneous vertical displacement of the WSEL. NOAA (2007) supports this approach by suggesting that uplift occurring over several tens of seconds may still be approximated with an instantaneous deformation.

To represent tsunami sources outside of the model domain (Section 4.2.4.1), a separate model domain was created of uniform depth. A time series of WSEL was output at the domain boundary. The east boundary of the PSL Delft3D model was then defined by the observed tsunami wave elevation time series. This method is similar to the approach presented in Dababneh et al. (2012) to simulate landslide-induced tsunamis.

Delft3D-FLOW boundary conditions used in all tsunami models were reflective of incoming waves. A true representation of an open ocean boundary would include non-reflective boundaries. As discussed in Deltares (2011b), some reflection of waves cannot be avoided.

To prevent wave reflections from artificially increasing the peak WSEL at PSL, time series boundary conditions for the Lisbon earthquake scenarios as well as the landslide scenarios were trimmed to represent only the initial long period wave.

The first set of analyses used a rectangular water surface displacement with a uniform change in elevation over the entire displacement area. These were considered to be screening runs to determine the critical seismic source zone. The screening tsunami runs were performed with the coarse grid Delft3D-FLOW model. The critical seismic and landslide scenarios were further refined by incorporating near-site bathymetry of PSL.

The critical seismic tsunami source was reevaluated with the analytic formulas presented in Okada (1985). These formulas allow a refined estimate of the ocean floor deformation geometry due to a seismic dislocation. Estimates of the tsunami source depth, length, width, slip, dip, strike, rake, and the tensile component of fault are required. As seismic depth cannot be constrained given existing literature, a sensitivity analysis was performed on depth, holding all other source parameters constant.

4.6.2.5 10 Percent Exceedance High Tide

A tidal adjustment equal to the difference between the ambient WSEL (EL +0 m-MSL) and the 10 percent exceedance high tide level was added to the water level at the end of the simulations to be consistent with the guidelines presented in NRC (2011). The 10 percent exceedance high tide is conservatively defined as 0.53 m-NAVD88 (1.74 ft) at Lake Worth Pier (Station 8722670) (Section 4.2.7.1). The conversion from NAVD88 to MSL at PSL was +0.278 m based on the datum maintained at NOAA Station Vero Beach, Florida (Station 8722125) (NOAA, 2013w). The 10 percent exceedance high tide is therefore given as 0.53 + 0.278 = 0.808 m-MSL at PSL.

4.6.2.6 Earthquake-Induced Tsunami Screening

A literature search was performed to conduct a tsunami screening evaluation. The screening considered near- and far-field earthquake, landslide, and volcano sources. Sources considered to be a credible threat (1 m tsunami wave reaching PSL) were further evaluated with detailed numerical modeling. The results of the screening are presented in Table 4-18.

4.6.2.7 Earthquake-Induced Tsunami Source Parameters

A literature search was conducted for determining earthquake-induced tsunami sources. Mader (2001a) presents a simulation of the 1755 Lisbon tsunami using an ad hoc estimate of a 300 km radius water surface depression of 30 m. More recent studies by Barkan et al. (2009) and Bird and Kagan (2004) provide more scientifically refined estimates of the 1755 Lisbon earthquake dimensions; therefore, the Mader (2001a) estimate was not evaluated. Additionally, this source parameter estimate of the Lisbon earthquake puts the event at less than a 10^{-6} annual probability of occurrence.

Barkan et al. (2009) studied the 1755 Lisbon earthquake and the implications for potential tsunami risk to the U.S. Atlantic coast. A number of hypothetical sources and source parameters were tested to determine the most probable parameters defining the Lisbon earthquake. The paper discusses a sensitivity analysis for source location and fault dimensions, comparing the results to historical records. Research by Barkan et al. (2009) suggested that the bathymetry of the Atlantic Ocean would allow tsunami waves to propagate from Lisbon towards southern Florida efficiently from Earthquake Source 3. Fault parameters used in simulations are presented in USGS (2008) and Barkan et al. (2009). Based on a shear modulus of ($3e^{+10}$ Pascals [Pa]), the seismic M_w of this event would be 8.53 (Equations 4.23 and 4.24). Based on the M_w frequency curve developed by Bird and Kagan (2004), the annual exceedance probability for this event would be approximately $9e^{-6}$.

$$M_0 = \mu AD \qquad \text{(Equation 4.23)}$$

where:
M_0 = scalar seismic moment (Nm)
μ = shear modulus (Pa)
A = area of the rupture (m^2)
D = average displacement (m)

$$M_W = \frac{2}{3} log_{10}(M_0) - 6 \qquad \text{(Equation 4.24)}$$

where:
M_w = moment magnitude
M_0 = scalar seismic moment (Nm)

Lisbon (Magnitude 8.61) presents a hypothetical earthquake event based on the Lisbon event that was scaled in magnitude to create a 10^{-6} annual exceedance probability earthquake event for the Azores-Gibraltar convergence boundary.

USGS (2008) describes an event occurring along the Puerto Rico Trench as having a maximum M_w of 8.85 (USGS, 2008). Based on M_w frequency plots for seismic activity along the Caribbean subduction zone, this would be a $2e^{-4}$ annual exceedance probability event (Bird and Kagan, 2004; USGS, 2008). While this annual exceedance probability exceeds the 10^{-6} annual threshold, USGS (2008) notes that:

> *"...it should be emphasized that such a large slip has never been documented along the Puerto Rico Trench, and the downdip length of the fault rupture is unknown. There is significant uncertainty in scaling average slip with respect to the rupture dimensions."*

Significant uncertainty exists in the estimation of the maximum M_w value for this region and large earthquake events in general. Based on a literature review, the source parameters presented in Table 4-19 are taken to be the probable maximum event for this location.

USGS (2008) describes an event occurring along the Hispaniola Trench as having a maximum M_w of 8.81 (USGS, 2008). Based on M_w frequency plots for seismic activity along the Caribbean subduction zone, this would be approximately a $3e^{-4}$ annual exceedance probability event (Bird and Kagan, 2004; USGS, 2008). As above, significant uncertainty exists among available data on the Hispaniola Trench. Similar to the Puerto Rico Trench, the source parameter values presented are taken as the maximum probable event for this location due to the disparity in estimates of M_{max}.

4.6.2.7.1 1755 Lisbon Earthquake, Magnitude 8.53

The 1755 Lisbon earthquake was simulated with two separate Delft3D-FLOW models because the source was outside of the model domain. A 10 km rectangular grid of uniform depth of 4,800 m was created. The Lisbon earthquake was simulated as a rectangular displacement of the WSEL of 13.1 m (Figure 4-40, elapsed time = 0 minutes). The tsunami was then allowed to propagate to the rectangular domain edge (Figure 4-40, elapsed time = 29 minutes and 1 hour and 39 minutes).

The time series observed at the edge of the simulated domain (Figure 4-41) was input into the Delft3D coarse grid model as a time varying WSEL boundary condition (Figure 4-42).

The results of this modeling exercise generally agree with the results presented by Barkan et al. (2009) who predicted 1 to 2 m wave amplitude along the Florida coast. The PSL site is partially protected by the Bahamas from a tsunami traveling in this direction. The peak wave amplitude at PSL was approximately 0.75 m (2.46 ft) (Figure 4-43).

4.6.2.7.2 1975 Lisbon Earthquake, Magnitude 8.61

The Magnitude 8.61 Lisbon earthquake was simulated with two separate Delft3D-FLOW models because the source was outside of the model domain. A 10 km rectangular grid of uniform depth of 4,800 m was created. The Lisbon earthquake was simulated as a rectangular displacement of the WSEL of 15 m (Figure 4-44, elapsed time = 0 minutes). The tsunami was then allowed to propagate to the rectangular domain edge (Figure 4-44, elapsed time = 19 minutes and 59 minutes).

The tsunami time series (Figure 4-45) generated at the edge of the rectangular grid was introduced onto the coarse grid as the eastern boundary condition (Figure 4-46).

The hypothetical Magnitude 8.61 Lisbon earthquake was simulated to predict the wave amplitude on the Atlantic Ocean near PSL. The PSL site is partially protected by the Bahamas from a tsunami traveling in this direction. The peak wave amplitude at PSL was approximately 0.9 m (2.95 ft) (Figure 4-47).

4.6.2.7.3 Puerto Rico Trench Earthquake

The Puerto Rico Trench was included in the coarse grid Delft3D model; therefore, simulation of an earthquake-induced tsunami resulting from the Puerto Rico Trench involved only one model. The tsunami was simulated as an initial displacement of the WSEL by 10 m (Figure 4-48, elapsed time = 0 minutes). The resulting tsunami wave was significantly reduced by the Bahamas, limiting the wave amplitude at PSL (Figure 4-48, elapsed time = 35 minutes and 195 minutes).

The peak WSEL at PSL was determined to be EL +2.1 m-MSL (EL +6.9 ft-MSL) (Figure 4-49).

4.6.2.7.4 Hispaniola Trench Earthquake

The Hispaniola Trench was included in the coarse grid Delft3D model; therefore, simulation of an earthquake-induced tsunami resulting from the Hispaniola Trench involved only one model. The tsunami was simulated as a 700 by 87.75 km initial displacement of the WSEL by 10 m (Figure 4-50). The resulting tsunami wave was significantly reduced by the Bahamas, limiting the wave amplitude at PSL (Figure 4-50).

The peak WSEL at PSL resulting from the Hispaniola Trench was determined to be EL +3 m-MSL (9.8 ft-MSL) (Figure 4-51).

4.6.2.8 Critical Earthquake Analysis Considering Bed Deformation

Based on the results of the coarse grid screening, the Hispaniola Trench was identified as the most credible seismic tsunamigenic threat to PSL as it resulted in the highest peak surge elevation (surge = 3 m-MSL). The analytic formulas presented in Okada (1985) were used to refine the representation of the ocean bed displacement for the Hispaniola Trench. The equations presented by Okada (1985) were used to determine the initial ocean bed displacement. Estimates of the Hispaniola Trench length (750 km), width (87.75 km), slip (10 m), dip (20 degrees), strike (95 to 102 degrees north), and rake were determined from USGS (2008).

The coarse grid was modified to remove a 1 million km^2 area representing the Hispaniola Trench source grid. The Hispaniola source was included as a 3.33 km resolution grid within the coarse grid coupled through domain decomposition.

The ocean floor deformation was solved with the analytic formulas presented in Okada (1985). The ocean floor displacement was included in the source grid as an initial surface displacement.

A sensitivity analysis of seismic depth was performed on surge elevation at PSL. The coarse grid model was evaluated with source depths of 1 km, 4 km, 8 km, 12 km, 16 km, 20 km, 24 km, 28 km, and 32 km. The critical seismic depth was determined to be 16 km for the Hispaniola Trench.

The sensitivity analysis demonstrates that consideration of the translation of the seismic fault to the ocean floor results in a significant decrease in tsunami wave amplitude (Figure 4-52).

4.6.2.9 Submarine Landslide-Induced Tsunami Screening

4.6.2.9.1 Submarine Landslide-Induced Tsunami Source Parameters

A list of the most probable submarine landslide tsunami sources was compiled. The associated parameters, including the dimensions of the slide and the associated probability, were derived from available literature. Table 4-20 presents the parameters for landslide sources that were evaluated.

As with the earthquake sources, current landslide data are limited by significant undersampling of the landslide distributions (USGS, 2008). Landslide data along the U.S. Atlantic coast are limited to 122 landslide events, 106 of which produced significant slide volumes (≥ 0.001 km^3).

Chaytor et al. (2009) propose that U.S. Atlantic landslides may fit a log-normal distribution with a mean of 6.6 km^3 and standard deviation of 2.27 km^3. To develop a probability of 10^{-6} for the landslide occurrence, the probability of the slide volume must be 0.0006 ($P_v / P(E) = P(O)$).

For a normal distribution, a test statistic (z) value of 3.26 has an associated one-sided probability of 0.0006 (Mendenhall and Sincich, 2006).

$$z = \frac{x-\mu}{\sigma} \qquad \text{(Equation 4.25)}$$

where:
z = standard normal test statistic
x = sample value
μ = mean
σ = standard deviation

$$3.26 = \frac{(x-\ln(6.6))}{\ln(2.27)} \qquad \text{(Equation 4.26)}$$

where:
X sample value = 4.56
Z test statistic = 3.26
Mean = ln(6.6)
Standard deviation = ln(2.27)

The slide volume (V) is defined such that $P(x \geq V) = 0.0009$. The landslide volume for this probability is estimated as ln(V) = 4.56, V = 95 km^3. This best-fit log-normal distribution underestimates the largest slide volumes defined by the empirical distribution. It is therefore not appropriate to use the best-fit log-normal distribution to extrapolate to extreme slide volumes. The primary limitations of fitting extreme slide volumes to a statistical distribution are due to significant undersampling of the true slide distribution and the fact that the maximum possible landslide failure along the U.S. Atlantic margin is currently unknown (USGS, 2008).

4.6.2.9.2 Cape Fear Landslide

Available literature presents varying estimates of the Cape Fear Landslide dimensions. The slide volume presented in USGS (2008) of 200 km^3 was used for the Cape Fear Landslide. The slide dimensions (200 km length by 25 km width by 0.04 km height) estimated in Rodriguez and Paull (2000) were used to determine slide geometry.

The Cape Fear Landslide was simulated with two separate Delft3D-FLOW models because the source was outside of the model domain. A 10 km rectangular grid of uniform depth of 2,000 m was created. The Cape Fear Landslide was simulated as a rectangular displacement of the WSEL of 40 m to represent the initial location of the slide and an increase in the WSEL of 40 m to represent the final location of the slide (Figure 4-53, elapsed time = 0 minutes). The tsunami was then allowed to propagate to the rectangular domain edge (Figure 4-53, elapsed time = 95 minutes and 175 minutes). The reflection following the first wave was artificial and was removed from the time series.

The time series of the Cape Fear Landslide tsunami (Figure 4-54) was introduced as a north boundary condition on the coarse grid Delft3D model (Figure 4-55). The peak wave height of 5 m resulting from the Cape Fear Landslide predicted by Hornbach et al. (2007) demonstrates that this is a conservative representation of the Cape Fear Landslide.

The resulting WSEL at PSL shows a peak of 0.6 m (1.97 ft) on the Atlantic Ocean (Figure 4-56).

4.6.2.9.3 Currituck Landslide

Available literature presents varying estimates of the Currituck Landslide dimensions. The slide volume presented in Locat et al. (2009) of 165 km^3 was used for the Cape Fear Landslide. The slide dimensions (180 km length by 55 km width by 0.02 km height) estimated in Locat et al. (2009) were used to determine slide geometry.

The Currituck Landslide was simulated with two separate Delft3D-FLOW models because the source was outside of the model domain. A 10 km rectangular grid of uniform depth of 2,000 m was created. The Cape Lookout Landslide was simulated as a rectangular displacement of the WSEL of 20 m to represent the initial location of the slide and an increase in the WSEL of 20 m to represent the final location of the slide (Figure 4-57, elapsed time = 0 minutes. The tsunami was then allowed to propagate to the rectangular domain edge (Figure 4-57, elapsed time = 70 minutes and 195 minutes). The reflection following the first wave was artificial and was removed from the time series.

The time series resulting from the Currituck Landslide (Figure 4-58.) was introduced onto the coarse grid Delft3D model as a north boundary condition (Figure 4-59). The peak WSEL of the initial wave form at PSL resulting from the Currituck Landslide was 0.6 m (1.97 ft) (Figure 4-60).

4.6.2.9.4 Tsunami Analysis Considering Near-Site Bathymetry

The worst case seismic (Hispaniola Trench) and landslide scenarios (Cape Fear Landslide) were evaluated further with additional bathymetric resolution around PSL to properly simulate shoaling effects and inundation.

The Hispaniola Trench scenario was run with the four-grid Delft3D-FLOW model (Section 4.2.4.1) to determine if the near-site topography and bathymetry would have a significant effect on tsunami propagation at PSL. Results are presented for a representative grid cell on the Atlantic Ocean at PSL from the fourth grid.

The peak WSEL from the Hispaniola Trench, including near-site bathymetry, was EL +0.65 m-MSL (2.13 ft). To account for the 10 percent exceedance high tide, 0.808 m (2.65 ft) was added to this final result, plus an additional 0.0609 m (0.2 ft) for sea level rise. This represents a conservative approach as losses would occur as the tsunami wave crosses the dune separating PSL from the Atlantic Ocean. The peak WSEL was therefore determined to be EL +1.52 m-MSL (5.0 ft-MSL) (EL +7.44 ft-PSL Datum) (EL +4.09 ft-NAVD88).

The Cape Fear Landslide was defined as the probable maximum landslide tsunami source based on a 10^{-6} annual exceedance probability threshold. The Cape Fear Landslide scenario was run with the five-grid Delft3D-FLOW model to determine if the near-site topography and bathymetry would have a significant effect on tsunami propagation at PSL. The peak WSEL at PSL resulting from the Cape Fear Landslide was 3.76 m-MSL (13.34 ft-MSL) at PSL.

The Cape Fear Landslide PMT did not exceed the dune separating the site from the Atlantic Ocean. The second peak occurring at PSL is the result of a wave packet that forms following the leading tsunami wave. The Cape Fear Landslide peak elevation was conservatively defined as the peak surge elevation from the Atlantic Ocean observation point. Through the simulation, the ambient sea elevation was simulated as EL +0 m-MSL.

To account for the 10 percent exceedance high tide, 0.808 m (2.65 ft) was added to this final result, plus an additional 0.0609 m (0.2 ft) for sea level rise. The peak WSEL was therefore determined to be EL +4.63 m-MSL (15.19 ft-MSL) (EL +17.63 ft-PSL Datum) (EL +14.3 ft-NAVD88).

4.6.3 Summary of Tsunami Analysis Results

The worst case seismic (earthquake) and landslide tsunami sources were determined to be the Hispaniola Trench and the Cape Fear Landslide, respectively. The peak WSEL at PSL, including near-site bathymetry, is presented in Table 4-21.

The PMT source for PSL is the Cape Fear Landslide. The peak WSEL at PSL is EL +17.6 ft-PSL Datum (EL +14.3 ft-NAVD88) (Table 4-21). The PMT inundation for this scenario is presented as Figure 4-61.

4.7 Ice-Induced Flooding

There are no records of ice jams in Florida in the Ice Jam Database of USACE (USACE, 2014). Ice sheet formation, wind-driven ice ridges, and frazil or anchor ice formation are also precluded because extended subfreezing water and daily average air temperatures have not occurred based on the available historical data.

4.8 Channel Diversion and Migration

PSL is located on Hutchinson Island, with the Atlantic Ocean on the east side of the plant and the Indian River (tidal lagoon) on the west side. Hutchinson Island was probably developed as an offshore bar during one of the later interglacial stages of the Pleistocene epoch and was subsequently exposed as sea level dropped in relation to the adjacent land surface. Based on the seismic, geological, topographical, thermal, and hydrological evidences of the region, there is no plausible risk that the safety-related facilities and functions of the plant will be adversely affected by channel diversions as described below.

The site is situated on the southern portion of the Floridian Plateau, a stable carbonate platform on which thick deposits of Cretaceous and Tertiary limestones, dolomites, evaporites, and comparatively small amounts of clastic sediments have accumulated. The Florida peninsula, as it exists today, is the emergent part of this plateau and lies totally within the Coastal Plain physiographic province of eastern North America.

There are no major natural rivers or channels located near PSL. The intake canal receives water directly from the Atlantic Ocean through subaqueous intake water pipes which run under the beach and end at the start of the canal east of State Route A1A. In the unlikely event of blockage of the intake canal or pipes, emergency cooling water is taken from Big Mud Creek through the emergency cooling water canal. According to the UFSAR, this emergency source of water is designed to withstand the design basis earthquake, tornado, and PMH conditions.

Big Mud Creek and the connecting Indian River are saltwater estuaries of the Atlantic Ocean and, therefore, diversion of their cooling water sources is not applicable.

4.9 Wind-Generated Waves

Wind-generated waves are evaluated coincident to the PMSS event. Descriptions of the methodologies and calculation for wind-wave evaluation are provided in Section 4.4.16.

4.10 Flooding-Related Loading

Flooding-related loading was calculated for each of the three flood-causing mechanisms: LIP, PMSS, and PMT. The loading cases considered include hydrostatic, hydrodynamic, wave impact, sediment, debris, and waterborne projectile loading.

4.10.1 Local Intense Precipitation-Related Loading

LIP-related loadings on Powerblock structures were calculated based on the procedures described in American Society of Civil Engineers (ASCE) 7-10 (ASCE, 2010). In general, the LIP water levels are relatively shallow, and flow velocities are relatively slow moving. Hydrostatic and hydrodynamic loadings are relatively low: however, light-duty structures such as doors and stoplogs could be susceptible to damage or leakage under LIP-related loading. Loading conditions were calculated at the POIs (Figure 4-6).

The ASCE 7-10 procedure consists of adding a hydrodynamic load (d_h) to the hydrostatic load (d), or water depth, when the velocities are less than 10 ft/s (which is the case for all the POIs). For simplicity, it was always considered that the velocity is perpendicular to the walls, although the velocity may be in a different direction. A magnifying factor of a = 1.25 is applied to the velocity head. Also per ASCE 7-10, the static water level should be increased by 1 ft.

Thus:

$$d_h = \frac{aV^2}{2g} \qquad \text{(Equation 4.27)}$$

$$h = d + 1ft \qquad \text{(Equation 4.28)}$$

$$H = h + d_h \qquad \text{(Equation 4.29)}$$

The average pressure is then computed:

$$p = \frac{\gamma H}{2} = \frac{\gamma(h + d_h)}{2} \qquad \text{(Equation 4.30)}$$

The total force on the wall/door is:

$$F = p \cdot A = p \cdot H \cdot B \qquad \text{(Equation 4.31)}$$

where:
B = width of the wall/door
F = force in lbs

In the present case, the results are presented per unit width, so $B = 1$ and F is expressed in lb/ft.

LIP loading diagrams are presented on Figure 4-62. Resultant loads are presented on Figure 4-63 and summarized in Table 4-22.

4.10.2 Probable Maximum Storm Surge Related Loading

Analyses were performed to estimate the hydrostatic and hydrodynamic pressure distributions due to storm surge and wind waves at various locations around the site. The key parameters in the analysis are the PMSS SWL, wavelength, wave height, and wave runup. The PMSS loading is calculated using methods presented by USACE (2011).

Loading was considered at the following plant locations:

- SSCs within the power block;
- FLEX Equipment Storage Building (FESB);
- Independent spent fuel storage installation (ISFSI) pad; and
- Non-safety related structures south of the power block.

Each location presents a unique configuration for evaluation of the loads. USACE (2011) presents schematic cases for the evaluation of loads based on specific configurations. The USACE case that most closely resembles each PSL location was used for evaluating the loading. These cases include:

1. The stillwater elevation is at or higher than the grade level of the toe of structure, and the wave breaks before reaching the structure (ISFSI and FESB);

2. The stillwater elevation is higher than the grade level of the toe of structure, and the unbroken wave directly impacts the structure (non-safety related buildings south of the power block);

3. The stillwater elevation is lower than the toe of structure and the wave breaks before reaching the structure and runs up to the structure (Power Block; however, wave runup does not reach the power block structures).

Note that the FESB has a grade elevation of approximately 15.65 ft-NAVD88, thus the PMSS water level is below grade. Therefore, waves will likely break on the FESB slope, and the FESB would satisfy Case 3 criteria. However, because the Case 3 equations are limited to relatively flat slopes (in the range of $0.01 < \tan \beta < 0.1$) and do not have an associated loading diagram, the Case 1 equations were deemed applicable for the FESB.

The maximum forces from these waves occur on a vertical rigid wall. The total pressure distribution on a vertical wall consists of two time varying components: the hydrostatic pressure component due to the instantaneous water depth at the wall; and the dynamic pressure component due to the accelerations of the water particles (USACE, 2011).

4.10.2.1 Hydrostatic Loading

The hydrostatic pressure increases in proportion to depth measured from the surface due to the increasing weight of fluid that is exerted downward from the force above. The hydrostatic pressure varies from zero at a height (H_w) above the SWL to a maximum at the base of the wall:

$$p_s = \gamma_w(h_s + H_w) \qquad \text{(Equation 4.32)}$$

where:
p_s = hydrostatic pressure (pounds per square foot) (lb/ft^2)
γ_w = specific (unit) weight of water = 64 lb/ft^3
h_s = still water depth (ft)
H_w= incident wave height (ft)

The hydrostatic force per horizontal unit length (F_s) is calculated as:

$$F_s = \frac{1}{2}\gamma_w(h_s + H_w)^2 \qquad \text{(Equation 4.33)}$$

The wave height at the wall (H_w) is calculated as:

$$H_w = \left(0.2 + 0.58\frac{h_s}{h_b}\right)H_b \qquad \text{(Equation 4.34)}$$

where:
h_b = water depth at wave break (ft)
H_b = breaking wave height (ft)

Shallow water wave breaking is a depth limited process, allowing the determination of the breaking wave height. A common breaker index, γ_b, of approximately 0.8 (0.78 is also commonly quoted) is used.

$$\frac{H_b}{h_b} = 0.8 = \gamma_b \qquad \text{(Equation 4.35)}$$

By definition, the hydrostatic force acts horizontally at a distance, h_r, above the floor per the following equation (Munsen et al., 2006):

$$h_r = \frac{1}{3}(h_s + H_w) \qquad \text{(Equation 4.36)}$$

The resultant loading conditions, including pressure distributions, resultant forces, and load application diagrams, are provided in Table 4-23a. Associated parameters are provided in Table 4-23b.

4.10.2.2 Hydrodynamic Loading at ISFSI and FESB

The hydrodynamic pressure, due to acceleration of the water particles, is estimated based on Hiroi's wave pressure formula. Hiroi's wave pressure formula is intended for use in relatively shallow water where breaking waves are the governing factor and are conservatively represented by a uniform pressure distribution, as shown in Table 4-23a. The pressure extends to the elevation of 1.25 H_w above the SWL. The hydrodynamic pressure (p_d) is represented by the following equation:

$$p_d = 1.5\gamma_w H_w \qquad \text{(Equation 4.37)}$$

where:

p_d = hydrodynamic pressure, lb/ft^2
γ_w = seawater unit weight = 64 lb/ft^3
H_w = incident wave height, ft

The hydrodynamic force per horizontal unit length (F_d) is calculated as follows (d'Angremond and van Roode, 2004):

$$F_d = p_d(h_s + 1.25H_w) \qquad \text{(Equation 4.38)}$$

The hydrodynamic flow is assumed to be uniform; therefore, the resultant force will act at the centroid of the projected area (Kim, 2010).

The resultant force is defined as the sum of the hydrostatic and hydrodynamic forces. Due to the resulting total force's irregular pressure distribution, shown in Table 4-23a, the height at which the total force acts is calculated using the following equation:

$$\bar{y}_{total} = \frac{\sum F_i \bar{y}_i}{\sum F_i} \qquad \text{(Equation 4.39)}$$

where:

$\bar{y}_{total}$ = height at which F_{total} acts ($F_{total} = F_s + F_d$)
$\bar{y}_i$ = height at which individual forces act
F_i = individual forces

The elevation of the resultant force is determined by adding the height of the resultant force to the site elevation at each location.

The resultant loading conditions, including pressure distributions, resultant forces, and load application diagrams, are provided in Table 4-23a. Associated parameters are provided in Table 4-23c.

4.10.2.3 Hydrodynamic Loading at Structures South of Power Block

The hydrodynamic loads for the area south of the power block were determined by using the Sainflou formula for non-breaking/partially breaking waves. The Sainflou formula for conditions under wave crest and wave trough were derived theoretically for the case of regular waves and a vertical wall. This formula cannot be applied in cases where wave breaking and/or overtopping take place.

$$p_1 = (p_2 + \gamma_w h_s)\frac{H_w + \delta_0}{h_s + H_w + \delta_0} \qquad \text{(Equation 4.40)}$$

$$p_2 = \frac{\gamma_w H_w}{\cosh\left(\frac{2\pi h_s}{L}\right)} \qquad \text{(Equation 4.41)}$$

$$\delta_0 = \frac{\pi H_w{}^2}{L}\coth\frac{2\pi h_s}{L} \qquad \text{(Equation 4.42)}$$

$$p_3 = \gamma_w(H_w - \delta_0) \qquad \text{(Equation 4.43)}$$

where:

H_w = Wave height, ft,
p_1 = Wave pressure at the still water level (corresponding to the wave crest), lb/ft^2,
p_2 = Wave pressure at the bottom of the body of water, lb/ft^2,

p_3 = Wave pressure at still water level, corresponding to wave trough, lb/ft^2,
δ_0 = Vertical shift in the wave crest = $(\pi H^2/L)\coth(\frac{2\pi h_s}{L})$, ft,
h_s = Water depth at the foot of the structure, ft,
L = Wavelength, ft, and
γ_w = specific weight of sea water, 64 lb/ft^3.

The negative pressure, p_3, of the Sainflou formula was not considered for the flood walls or buildings because it reduces the stress on the wall when the wave is pulling (rather than pushing) on the building or structure. The hydrodynamic force per horizontal unit length (F_d) for the Sainflou pressure distribution was conservatively approximated as the maximum pressure (p_1) acting over the entire height of the pressure column as shown in the loading diagram in Table 4-23a. Associated parameters are provided in Table 4-23d.

4.10.2.4 Probable Maximum Storm Surge Debris and Waterborne Projectile Loading

Debris loading and waterborne projectile loading conditions were calculated based on procedures presented in ASCE 7-10. NRC guidance (NRC, 2013a) states that:

- Debris loads on SSCs important to safety should be considered.
- The methodologies described in ASCE 7-10 (with consideration of local conditions) are acceptable to the NRC staff.

The methods of ASCE 7-10 were used to calculate impact loads. Per ASCE 7-10, impact loads are those that result from logs and other objects striking buildings, structures, or parts thereof.

ASCE 7-10 presents a method for calculation of the magnitude of impact loads as follows:

$$F = \frac{\pi W V_b C_I C_0 C_D C_B R_{max}}{2g\Delta t} \quad \text{(Equation 4.44)}$$

where:

F = impact force, lbs
W = debris weight, lbs
V_b = velocity of object (equal to velocity of storm surge water), ft/s
g = acceleration due to gravity = 32.2 ft/s^2
Δt = impact duration time, s = 0.03 second
C_I = importance coefficient = 1.3
C_o = orientation coefficient = 0.8
C_D = depth coefficient = 1.0
C_B = blockage coefficient = 1.0
R_{max} = response ratio for half-sine impulsive load = 0.8

An orientation coefficient (C_o) of 0.8 was selected for this analysis as a direct (head-on) impact from debris is unlikely. ASCE 7-10 recommends using an orientation coefficient of 0.8 because, even in laboratory testing, direct impact from debris is rare without any obstructions.

The debris impact force will act up to the maximum water height (i.e., $h_s + 1.25H_w$).

The pressure (P) is determined by dividing the force by the area of the impact:

$$P = F \div A \quad \text{(Equation 4.45)}$$

where:
P = pressure due to impact force (lb/ft^2)
A = 1.23 ft^2 for 15 inch diameter log weighing 2,000 lbs

Per ASCE 7-10, a 1,000 lb object can be considered a reasonable average for floodborne debris. This represents a reasonable weight for trees, logs, and other large woody debris that are the most common forms of damaging debris nationwide. This weight corresponds to a log approximately 30 ft long and just under 1 ft in diameter. This reference also notes that regional or local conditions should be considered to determine the debris object weight. PSL is located in a coastal area. Guidance in ASCE 7-10 states that debris weights in coastal areas generally fall into three classes as follows:

1. In the Pacific Northwest, a 4,000 lb debris weight is typical due to the large trees and logs in this area.
2. In other coastal areas where piers and large pilings are available locally, debris weights may range from 1,000 to 2,000 lbs.
3. In other coastal areas where large logs and pilings are not expected, debris will likely be derived from failed decks, steps, and building components and will likely average less than 500 lbs.

Based on the recommendation for coastal areas, a debris object weight of 2,000 lbs was used to determine the maximum impact load. To account for the larger weight, a slightly larger diameter of 15 inches was used to calculate the resulting pressure.

The maximum average current velocity during the PMSS is 6.23 ft/s. Per ASCE 7-10, the debris will travel at the velocity of the flood waters.

The resultant loading conditions are provided in Table 4-23a.

4.10.2.5 Probable Maximum Storm Surge Sediment Loading

The maximum sediment deposition is bounded by the PMSS level (i.e., 14.9 ft-NAVD88).

The vertical soil pressure was determined using the following equation from Das (2011):

$$\sigma_{vertical} = \gamma_{buoyant} z \quad \text{(Equation 4.46)}$$

where:
$\sigma_{vertical}$ = vertical soil pressure (lb/ft^2),
$\gamma_{bouyant}$ = buoyant unit weight of soil = wet unit weight minus seawater unit weight (lb/ft^3)
z = depth of soil/sediment (ft)

The lateral (horizontal) pressure of the soil/sediment was determined using the following equations from Das (2011):

$$\sigma_{horizontal} = K_o \sigma_{vertical} \quad \text{(Equation 4.47)}$$

$$K_o = 1 - \sin \phi \quad \text{(Equation 4.48)}$$

where:

K_o = coefficient of at-rest earth pressure

ϕ = angle of internal friction = 29 degrees for very loose sand (Winterkorn and Fang, 1975)

The resultant force per unit length (P_o) due to sediment loading on an SSC wall acts at 1/3 the height (z) as determined by the following equation (Peck et al., 1974):

$$P_o = \frac{1}{2} K_o \gamma z^2 \quad \text{(Equation 4.49)}$$

The resultant loading conditions are provided in Table 4-23a. Associated parameters are provided in Table 4-23e.

4.10.3 Probable Maximum Tsunami Related Loading

PMT-related loading was calculated based on guidance in JLD-ISG-2012-06 (NRC, 2013a) and NUREG/CR-6966 (NRC, 2009) and the methods of the USACE Coastal Engineering Manual (USACE, 2011). Note that the peak PMT water level does not reach the Powerblock; therefore, SSCs within the Powerblock are protected against PMT-related loading due to their physical positioning. The intake would be susceptible to the effects of PMT as would nonsafety-related structures at or below the PMT water level.

4.10.3.1 Probable Maximum Tsunami - Hydrostatic and Hydrodynamic Loads

Hydrostatic force per unit width on a wall that is not overtopped by the tsunami waves is given by NRC (2009):

$$F_S = \frac{1}{2} \rho g \left(h + \frac{v^2}{2g}\right)^2 \quad \text{(Equation 4.50)}$$

where:

F_s = hydrostatic force per unit width of wall (lb/ft)

ρ = density of seawater at 68°F = 1.985 slugs/ft^3

g = acceleration due to gravity (32.2 ft/s^2)

v = velocity normal to the structure (ft/s)

h = water depth (ft)

The hydrostatic force acts horizontally at a distance, h_r, above the floor or ground per the following equation:

$$h_r = \frac{1}{3}\left(h + \frac{v^2}{2g}\right) \quad \text{(Equation 4.51)}$$

The hydrostatic pressure, p_s, is determined as (Finnemore and Franzini, 2002):

$$p_{s=}\ \rho g(h + \frac{v^2}{2g}) \qquad \text{(Equation 4.52)}$$

The hydrodynamic force was determined per the following equation (NRC, 2009):

$$F_d = \frac{1}{2}\rho C_d A v^2 \qquad \text{(Equation 4.53)}$$

where:

F_d = hydrodynamic (i.e., drag) force (lbs)
ρ = density of seawater at 68°F = 1.985 slugs/ft^3
v = velocity normal to the structure (ft/s)
A = projected area of the structure on a plane normal to the flow direction (ft^2)
C_d = coefficient of drag

The hydrodynamic flow is assumed to be uniform; therefore, the resultant force (F_d) will act at the centroid of the projected area (Kim, 2010):

$$h_r = \frac{1}{2}h \qquad \text{(Equation 4.54)}$$

where:

h_r = resultant force height (ft)
h = water depth (ft)

The resultant force, F_{total}, is defined as the sum of the hydrostatic and hydrodynamic forces. Due to the resulting total force's irregular pressure distribution, as seen in Table 4-24a, the height at which the total force acts is calculated using the following equation:

$$\bar{y}_{total} = \frac{\sum F_i \bar{y}_i}{\sum F_i} \qquad \text{(Equation 4.55)}$$

where:

$\bar{y}_{total}$ = height at which F_{total} acts
$\bar{y}_i$ = height at which individual forces act
F_i = individual forces

The elevation of the resultant force is determined by adding the height of the resultant force to the site grade at each location.

The resultant hydrostatic and hydrodynamic pressure distributions are provided in Table 4-24a. Associated parameters are provided in Tables 4-24b and 4-24c.

4.10.3.2 Probable Maximum Tsunami – Debris and Waterborne Projectiles

JLD-ISG-2012-06 requires that the effects from debris and waterborne projectiles be considered when tsunami flood levels impinge on flood protection or safety-related SSCs. JLD-ISG-2012-06 refers to NUREG/CR-6966 for additional details on debris and waterborne projectile forces.

NUREG/CR-6966 presents a method for calculation of the magnitude of impact loads as follows:

$$F = m\left(\frac{v}{t}\right) \quad \text{(Equation 4.56)}$$

where:
F = impact force (lbs)
m = mass of impacting projectile (slugs)
v = velocity of object (equal to velocity of PMT water) (ft/s)
t = impact duration time (seconds) (varies for different construction materials)

The debris impact force will act at the maximum water height.

The pressure (P) is determined by dividing the force by the area of the impact:

$$P = F \div A \quad \text{(Equation 4.57)}$$

where:
P = Pressure due to impact force (lb/ft^2)
A = 1.23 ft^2 for 15 inch diameter log weighing 2,000 lbs (Section 4.10.2.2).

The maximum average current velocity during the PMT is 9.2 ft/s. The debris will travel at the velocity of the flood waters.

The maximum sediment deposition is bounded by the maximum PMT water level (i.e., 14.3 ft-NAVD88).

The lateral (horizontal) pressure of the sediment was determined using the following equation from Peck et al. (1974):

$$P_h = k_o \gamma z \quad \text{(Equation 4.58)}$$

where:
k_0 = coefficient of at-rest earth pressure = $1 - \sin \emptyset$
$\emptyset$ = angle of internal friction = 29 degrees for very loose sand (Winterkorn and Fang, 1975)
γ = buoyant unit weight of soil = wet unit weight minus seawater unit weight (lb/ft^3)
z = depth of soil/sediment (ft)

The resultant force per unit length (P_o) due to sediment loading on an SSC wall acts at 1/3 the height (z) as determined by the following equation (Peck et al., 1974):

$$P_o = \frac{1}{2} k_o \gamma h^2 \quad \text{(Equation 4.59)}$$

The resultant loading conditions, including pressure distributions, resultant forces, and load application diagrams, are provided in Table 4-24a.

4.11 Debris and Sedimentation

The potential for fouling of PSL's safety water intake structure and equipment due to debris and sedimentation is evaluated based on postulated extreme events, major windstorm events, and non-flood related mechanisms. Because of PSL's coastal setting, marine fouling also has the potential to affect the water intake structure and equipment. Although some of the debris and sedimentation mechanisms considered are not strictly flood related, the mechanisms are intrinsically related to plant's potential vulnerability to loss of safety-related cooling water.

4.11.1 Summary of Debris and Sedimentation Mechanisms

Fouling from debris can occur from:

- Current-borne loading of debris
- Episodic large-volume debris transport from extreme events (tsunamis, windstorms, hurricane surges)

Fouling from sedimentation can occur from:

- Sediment transport loading in cooling water canals
- Episodic large-volume sediment transport from extreme events (tsunamis, hurricane surges)
- Sediment transport loading in ocean currents
- Fouling from non-flood related mechanisms can occur from:
- Microorganisms/biological blooms
- Macroorganisms (barnacles, mussels, jellyfish)
- Seaweed

4.11.1.1 Postulated Extreme Events

Postulated extreme events include tsunamis and windstorms. Major tsunamis of critical magnitude have not been observed in South Florida; thus, PSL has never been affected by a tsunami.

Tsunamis have the capability of carrying large volumes of debris and sediment. Generally, a tsunami picks up debris and sediment as it moves inland, and additional debris and sediment transport potential occurs during the receding wave. The PMT peak surge level is EL +14.3 ft-NAVD88 (Section 4.6); thus, there is potential for a tsunami surge to reach the plant intake.

4.11.1.2 Major Windstorm Events

Windstorm events (tropical storms and hurricanes) can carry windborne and waterborne debris. Major windstorms have occurred frequently at PSL. Recent significant hurricanes include Hurricanes Frances (2004), Jeanne (2004), Wilma (2005), and Irene (2011). Hurricanes Frances and Jeanne caused significant damage on Hutchinson Island, including effects at PSL. Subsequently, the intake channel was armored to prevent the recurrence of embankment erosion (Section 2.3.6).

4.11.1.3 Non-Flood Related Mechanisms

Non-flood related mechanisms can result in debris and sedimentation accumulation. In general, for plants with ocean intakes, these mechanisms can include current-borne sediment and debris as well as marine fouling. PSL does experience problems with current-borne sediment during extreme storm events and certain current-borne marine life such as seaweed, jellyfish, and biological blooms because of the intake configuration where the intake velocity caps are elevated off the ocean bed. Thus, floating debris and minimal near-beach shifting sediments do reach the intake. Numerous hurricanes have been experienced in the plant's life, and the ICW pumps which are the critical equipment at the intake structure have remained operable. Florida Power & Light (FPL) has a procedure (1-NOP-21.15 [FPL, 2014c]) in place to maintain the cooling canals, which includes sediment monitoring and associated maintenance.

4.11.2 Sedimentation and Debris Protection

As described above, the intake is susceptible to sedimentation and debris accumulation during postulated extreme storm events and certain current-borne marine life. The intake structure is equipped with features such as multiple submerged debris nets, coarse screens, and fine traveling water screens that prevent floating debris from adversely affecting the ICW pumps. However, there are no quantitative evaluations of the impact load capacities of the intake structure nets and screens in the CLB.

PSL has effectively managed sedimentation and debris from historical extreme windstorms or from other causal or chronic mechanisms associated with non-flood related mechanisms. Furthermore, PSL has a monitoring and maintenance program to prevent debris and sedimentation impacts in the cooling water and intake systems (1-NOP-21.15 [FPL, 2014c]).

4.12 Low Water Considerations

In an oceanside setting, potential low water events can be induced by tidal fluctuations, negative storm surges, and tsunami drawdowns. Low water events can potentially impact a safety-related cooling system by depleting the available water supply or exposing pump systems to detrimental air intake conditions. The normal, and emergency, source of cooling water for the plant is the Atlantic Ocean. This source of cooling water is not affected by low flow in rivers and streams.

4.12.1 Probable Maximum Tsunami Induced Low Water

The minimum WSEL from the Hispaniola Trench was -0.56 m. To account for the 10 percent exceedance low tide, -0.84 m (-2.76 ft) was added to this final result. The minimum WSEL was therefore determined to be EL -1.40 m-MSL (-4.59 ft) (EL -2.15 ft-PSL Datum) (EL -5.50 ft-NAVD88).

The minimum WSEL from the Cape Fear Landslide was -2.19 m (-7.18 ft). To account for the 10 percent exceedance low tide, -0.84 m (-2.76 ft) was added to this final result. The minimum WSEL was therefore determined to be EL -3.03 m-MSL (-9.94 ft) (EL -7.50 ft-PSL Datum) (EL -10.85 ft-NAVD88).

The PMT low water results are summarized in Table 4-25.

4.12.2 Probable Maximum Storm Induced Low Water

The low water-inducing PMH is the probable maximum storm (PMS) that will cause the lowest drawdown at PSL. The PMS low water duration is defined as the duration of drawdown below the 10 percent exceedance low tide (EL -3.67 ft-NAVD88 [-0.32 ft-PSL Datum]). For a hurricane at landfall, where the hurricane wind field is rotating counterclockwise, the negative surge (or low water) will typically dominate in the coastal area on the left of the translating storm track, then increase towards the storm center, and then have a positive surge in the coastal area to the right of the storm track (IHE, 2003).

In determining the low water caused by the PMH, it is assumed the PMH resulting in the PMSS will be the storm which will also contribute to the low water at PSL. The exception is that the hurricane track is offset such that the maximum winds to the left of the hurricane center induce a negative surge at PSL. The following conservative assumptions were also made:

a) The PMH does not weaken (wind speed decrease or central pressure increase) once the hurricane makes landfall.

b) No sea level rise is present as this would yield a higher initial WSEL.

c) No waves are coupled with the model so that no wave setup is present.

A sensitivity of the hurricane's landfall location is performed through seven simulations with a storm track ranging from +2.25 times the RMW to +3.25 times the RMW. The landfall location was defined such that a landfall location of 0 indicates the hurricane passes 1 RMW south of PSL. To further illustrate the convention, a landfall location of +1 RMW indicates that the hurricane eye passes over PSL.

The timing of the maximum negative storm surge is synchronized with the incoming low tide in the Delft3D model. If the peak of a hurricane's negative storm surge arrives at high tide, the water level will be higher than if it arrives at low tide. To account for this possibility, the model runs are synchronized such that the peak of the negative storm surge arrives at low tide. The difference between the ambient WSEL at landfall of the PMH and the calculated 90 percent exceedance low tide is then calculated. The difference is accounted for by adding it to the final negative surge (as a negative number) in the model result.

The storm with the following attributes produced the low water at PSL:

- A central pressure of 885.54 millibars (26.15 in. Hg)
- An RMW of 19 nmi
- A storm center at a distance of 2 RMW north of PSL
- A forward speed of 6 knots
- A track direction of 70 degrees

The components of the PMS low water elevation at PSL can be summarized as follows:

Atlantic Ocean

Initial Condition (Antecedent Water Level)	EL +0.0 ft-MSL
10 Percent Exceedance Low Tide	-2.758 ft-MSL
Wind Setdown	-2.85 ft
PMS Low Water Elevation	EL -5.608 ft-MSL

$$\text{Probable Maximum Storm Low Water at PSL} = \begin{Bmatrix} -5.61 \text{ ft MSL} \\ -3.17 \text{ ft PSL Datum} \\ -6.52 \text{ ft NAVD88} \end{Bmatrix}$$

Big Mud Creek

Initial Condition (Antecedent Water Level)	EL +0.0 ft-MSL
10 Percent Exceedance Low Tide	-2.758 ft-MSL
Wind Setdown	-1.36 ft
PMS Low Water Elevation	EL -4.118 ft-MSL

$$\text{Probable Maximum Storm Low Water at PSL} = \begin{Bmatrix} -4.12 \text{ ft MSL} \\ -1.68 \text{ ft PSL Datum} \\ -5.03 \text{ ft NAVD88} \end{Bmatrix}$$

This hurricane combination results in a low water duration of 5.5 hours below EL -3.67 ft-NAVD88 at the Atlantic Ocean side of PSL and 14.5 hours in Big Mud Creek. Refer to Figure 4-64 for the bounding PMS low water time series.

4.13 Combined Events Flooding

Combined events flooding was evaluated in accordance with NUREG/CR-7046 (after ANSI/ANS-2.8-1992). The critical PMF combination event for PSL was determined.

The combination flooding analysis was performed according to the following guidelines:

1. NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America" (NRC, 2011).

2. ANSI/ANS-2.8-1992, "Determining Design Basis Flooding at Power Reactor Sites" (ANS, 1992).

PSL is a "shore" location on an "open or semi-enclosed body of water." For this location, combined events flooding involving surges, seiches, tsunamis, and tides might produce maximum flood levels. Because PMP flooding of streams and rivers cannot affect the site, combined events involving PMF will have no impact on PSL. Also, because there are no dam failure-related flooding hazards, dam flooding combinations are precluded from analysis. Thus, the applicable combined events flood hazards are:

- Storm surge-related combination:
  - PMSS and seiche with wind-wave activity
  - Antecedent 10 percent exceedance high tide

- Tsunami related combination:
  - PMT runup
  - Antecedent 10 percent exceedance high tide

These combinations are the same as those analyzed in Section 4.4 and Section 4.6 for PMSS and PMT, respectively. Refer to those sections for the descriptions of the analyses.

5.0 COMPARISON WITH CURRENT DESIGN BASIS

5.1 Local Intense Precipitation

For the CLB (Section 3.1), the PSL site was evaluated for a PMP of 32 inches and 47.1 inches during a 6-hour and 24-hour period, respectively, over a 10-square mile area. The roof leaders are designed for a rainfall intensity of 6 inches per hour. Short periods of more intense rainfall would result in water running off the edges of roofs with no adverse effects to safety-related equipment. There is no water buildup on the roofs in excess of 1'-6" high parapet. The CLB states that none of the above conditions adversely affect the structures or safety-related equipment. Effects of flooding at door locations around the Powerblock were not evaluated.

The site-specific LIP analyzed for the flooding hazards reevaluation was determined to be 19.4 inches of precipitation in a 60-minute period. A FLO-2D model was used to evaluate the LIP and determine the depth of water at any point on the PSL site. Site survey data were used to construct the model, and four LIP distributions were evaluated.

The results for the LIP temporal distributions evaluated are similar in terms of the maximum values for both depth and velocity. However, the time distribution of these quantities is significantly affected. In general, it was found that the first quartile and fourth quartile temporal distribution (Section 4.1.1) hyetographs produced the highest water depths. Some locations in the vicinity of the main buildings of Units 1 and 2 have water depths greater than 2 ft.

The reevaluated LIP water depths exceed the CLB conditions at the Powerblock.

5.2 Riverine (Rivers and Streams) Flooding

It is concluded that PSL is not affected by flooding from streams, rivers, or canals.

5.3 Dam Breaches and Failure Flooding

It is concluded that PSL is not affected by flooding from dam breaches or failures.

5.4 Storm Surge

The CLB PMSS WSEL at the Powerblock is EL +13.85 ft-NAVD88 (EL +17.2 ft-PSL Datum), which is well below the flood protection level of EL +16.15 ft-NAVD88 (EL +19.5 ft-PSL Datum).

The reevaluated PMSS WSEL at the Powerblock is EL +14.9 ft-NAVD88 (EL +18.3 ft-PSL Datum) with 30-year sea level rise. At the present time, the reevaluated PMSS WSEL is EL +14.7 ft-NAVD88 (EL +18.1 ft-PSL Datum) (without future sea level rise). It was determined that no wave runup is present at the PSL Powerblock POIs.

The reevaluated PMSS peak water level exceeds the CLB peak water level; however, the water surface is still below the Powerblock grade elevation (EL +18.5 ft-PSL Datum) and the flood protection level (EL +19.5 ft-PSL Datum) with reduced margin.

5.5 Seiche

It is concluded that PSL is not affected by seiche flooding (as an independent mechanism) or by seiche flooding coincident with the PMSS.

5.6 Tsunami Flooding

The CLB excluded tsunami events based upon historical regional data.

The reevaluation determined the worst case seismic (earthquake) and landslide tsunami sources were the Hispaniola Trench and the Cape Fear Landslide. The flooding hazards reevaluation determined the PMT source for PSL is the Cape Fear Landslide. The peak WSEL at PSL is EL +17.6 ft-PSL Datum (EL +14.3 ft-NAVD88) (Table 4-21).

The PMT peak WSEL exceeds the CLB; however, the water surface is still below the Powerblock grade elevation (EL +18.5 ft-PSL Datum) and the flood protection level (EL +19.5 ft-PSL Datum).

5.7 Ice-Induced Flooding

PSL is not affected by ice-induced flooding.

5.8 Channel Migration or Diversion

PSL is not affected by channel migration or diversion.

5.9 Wind-Generated Waves

The CLB considers the effects of wave runup for the maximum postulated surge level of EL +13.85 ft-NAVD88 (EL +17.2 ft-PSL Datum), the maximum water elevation is EL +15.45 ft-NAVD88 (EL +18.8 ft-PSL Datum) except for waves from the east over eroded areas (dunes and mangroves), which propagate up the discharge canal approaching the nose where Unit 1 and Unit 2 canals join, where a maximum water elevation (surge with runup) of EL +24.65 ft-NAVD88 (EL +28 ft-PSL Datum) is postulated. The discharge canal nose area is protected by a steel sheet-piling barrier with its top at EL +18.65 ft-NAVD88 (EL +22 ft-PSL Datum). During the peak surge water level of EL +13.85 ft-NAVD88 (EL +17.2 ft-PSL Datum), the refracted wave will break on the slope in front of the sheet piling and result in a wave runup of about 11 ft on a hypothetical extension of the slope of the canal nose. Overtopping is expected and the resultant flood water behind the barrier that will drain on grade. The temporary flooding around the nose is of no concern since there are no Category I structures located in that part of the plant site.

The flooding reevaluation determined that the maximum wave runup at the discharge canal would result in overtopping of the steel sheet-piling barrier at the nose of the discharge canal, but this overtopping discharge volume (5.0×10^{-4} $ft^3/s/ft$) is deemed insignificant by USACE (2011).

The reevaluated wave runup analyses resulted in the same conclusion as the CLB; the Powerblock is protected from wave runup by the discharge canal steel sheet-piling barrier.

5.10 Flooding-Related Loading

The CLB does not consider hydrodynamic loading; however, the UFSAR considers subsurface hydrostatic loading. According to the UFSAR, all Seismic Category I structures were designed for hydrostatic loading. The intake channel is equipped with screens and netting for protection against damage from debris and waterborne projectiles. However, the structural capacity of the screens and netting is not quantified.

5.10.1 Local Intense Precipitation Related Loading

The CLB does not consider loading due to LIP; therefore, the reevaluated LIP loading conditions exceed the license basis. LIP loading needs to be considered in the Integrated Assessment.

Flooding of the Powerblock will only occur during an LIP event. The potential debris generation caused by the LIP event will be from unsecured materials located inside the plant Powerblock. Procedurally controlled housekeeping practices (MA-AA-100-1008 [FPL, 2014a] and 0005753 [FPL, 2014d]) minimize the amount of material/debris that can be moved by LIP runoff. In addition, the flow velocities inside the Powerblock are low, minimizing the ability for waterborne projectiles to adversely affect plant flood protection features.

5.10.2 PMSS-Related Loading

During the PMSS event, structures located below the PMSS at or below EL +14.9 ft-NAVD88 (EL +18.3 ft-PSL Datum) will be subject to hydrostatic, hydrodynamic, debris impact, waterborne projectile, and sediment loads. It is understood that nonsafety-related structures below EL +14.9 ft-NAVD88 will not require protection against the PMSS event.

In the CLB, waterborne projectiles and debris are not considered; however, the maximum water elevation resulting from a storm surge with wind-generated waves reaching EL +14.75 ft-NAVD88 (EL +18.1 ft-PSL Datum) at the Powerblock; therefore, a waterborne projectile is not a credible hazard for safety-related structures inside the Powerblock with a flood protection level at EL +16.15 ft-NAVD88 (EL +19.5 ft-PSL Datum).

Safety-related SSCs potentially affected by PMSS are:

- UHS barrier
- Intake structure and appurtenances

The ICW structures (located at EL +13.15 ft-NAVD88 [EL +16.5 ft-PSL Datum]) are designed for both seismic loads and tornado-generated missiles since they house the safety-related ICW pumps located at EL +18.65 ft-NAVD88 (EL +22 ft-PSL Datum). The ICW pumps draw cooling water from a submerged inlet located between EL -14.85 ft and EL -32.85 ft-NAVD88 (EL -11.5 ft and EL -29.5 ft-PSL Datum) on the west side of the structures. The cooling water enters the structure through the submerged inlet then travels through a series of debris removal structures consisting of coarse screen guides and traveling screens. The coarse screen guides are constructed out of 4"x1/2" steel bars welded at 3-1/8" on center with a W8 sub-frame and have some capacity to absorb impact loads from submerged debris. However, the structural capacity is not quantified. The cooling water inlet is submerged approximately 2 ft below the minimum intake channel water level and is protected by a 2-ft thick concrete wall which has been designed for both seismic loads and tornado-generated missiles (NEE, 2014c).

The reevaluated PMSS event, with wave runup EL +14.9 ft-NAVD88 (EL +18.3 ft-PSL Datum), does not cause a credible flooding hazard for safety-related structures inside the plant island (EL +19.5 ft-PSL Datum).

The loadings prescribed herein may not be bounding or directly applicable for other locations and structures; further consideration should be evaluated independently on a case-by-case basis.

5.10.3 Probable Maximum Tsunami Related Loading

During the PMT event, structures below the PMT EL +14.3 ft-NAVD88 (EL +17.6 ft-PSL Datum) will be subject to hydrostatic, hydrodynamic, debris impact, waterborne projectile, and sediment loads. It is understood that nonsafety-related structures below EL +14.3 ft-NAVD88 will not require protection against the PMT event.

In the CLB, tsunami-related loading is not considered.

The maximum PMT WSEL is EL +14.3 ft-NAVD88 (EL +17.6 ft-PSL Datum) at the Powerblock; therefore, a PMT loading is not a credible hazard for safety-related structures inside the Powerblock with a flood protection level at EL +16.15 ft-NAVD88 (EL +19.5 ft-PSL Datum).

Safety-related SSCs potentially affected by PMT are:

- UHS barrier
- Intake structure and appurtenances

These SSCs are described above in Section 5.10.2, and PMT would be applied similar to the described PMSS effects.

The loadings prescribed herein may not be bounding or directly applicable for other locations and structures; further consideration should be evaluated independently on a case-by-case basis.

5.11 Debris and Sedimentation

The CLB evaluated erosion by analyzing wave conditions during a stalled or looping hurricane. The CLB analysis evaluated wave-induced erosion around the PSL generating station. This analysis included historical beach erosion in the vicinity of the site, laboratory test results which simulate wave erosion during a hurricane event, and conservative methods for estimating the quantity of erosion.

The CLB did not evaluate sedimentation resulting from the erosion and deposition of the dunes and other loose sediment. Debris and sedimentation accumulation resulting from a beyond design basis external event (BDBEE) is expected to have the largest impact on the east side (ocean side) of the plant site due to wave runup. Because the PSL Powerblock grade is located above the highest probable WSEL, there would be no threat of debris and sedimentation that could impact safety-related structures in that area. The additional physical attributes of the parking lot earth berms, State Route A1A roadway, hardened slopes on the intake and discharge canals, and vehicle security barrier were conservatively excluded from the CLB analysis. Each of these features would help to prevent sedimentation and debris accumulation from reaching other plant areas; however, these attributes have not been quantified for these evaluations.

The largest threat related to the plant's potential vulnerability to debris and sedimentation is loss of safety-related cooling water. However, debris and sedimentation accumulation having the potential to affect the water intake structure and associated equipment is protected by a series of mechanisms that include:

- Robust marine wildlife nets that span across the intake canal.
- Coarse and fine screens that prevent submerged debris from adversely affecting the ICW pumps and the ability to provide an alternate cooling water source from Big Mud Creek through the UHS barrier. The screens have some capacity to absorb impact loads from submerged debris; however, the structural capacity is not quantified.

5.12 Low Water Considerations

The normal, and emergency, source of cooling water for the plant is the Atlantic Ocean. This source of cooling water is not affected by low flow in rivers and streams. In the unlikely event of blockage of the intake canal or pipes, emergency cooling water is taken from Big Mud Creek through the emergency cooling water canal via the UHS barrier. This emergency source of water is designed to withstand the design basis earthquake, tornado, and CLB PMH conditions.

The CLB determined that an extreme low tide of EL -3.0 ft-PSL Datum in the Indian River Lagoon can be expected to occur at the plant site. The intake structure bottom elevation is EL -31 ft-PSL Datum. The circulating water pump suction elevation is EL -16 ft-PSL Datum. The ICW pump suction elevation is EL -18.5 ft-PSL Datum.

The minimum submergence is 6 ft for the circulating water pumps. For the ICW pumps, the minimum submergence is 4 ft for 14,500 gpm flow. Therefore, the minimum water level which will sustain the required cooling water flow (14,500 gpm) is EL -14.5 ft-PSL Datum. Since the lowest water level in the intake canal with all circulating water pumps operating would be EL -6 ft-PSL Datum, and EL -9 ft-PSL Datum considering 3 ft additional drawdown from PMH maximum wind, the intake water pumps are assured sufficient submergence under the CLB conditions.

Low water due to tsunamis was not evaluated in the CLB.

For the reevaluated PMSS event, the low water level on the Atlantic Ocean is EL -3.17 ft-PSL Datum, and the low water at Big Mud Creek (UHS barrier) is EL -1.68 ft-PSL Datum.

For the reevaluated PMT event, the low water level on the Atlantic Ocean is EL -7.50 ft-PSL Datum, and the low water at Big Mud Creek (UHS barrier) is EL -1.71 ft-PSL Datum.

The reevaluated low water level (due to the PMT) exceeds the CLB; however, the intake is safe to a low water level of EL -14.5 ft-PSL Datum.

5.13 Combined Events

As described in Section 4.14, the combined flooding effects related to storm surge and tsunami flooding apply to PS, and are addressed in Sections 4.5 and 4.6, respectively. Therefore, the relevant comparisons to the CLB are provided in Sections 5.5 and 5.6.

5.14 Summary of Results

The reevaluation of the flooding hazards for PSL has concluded that some flood mechanisms and effects exceed the CLB. The flooding mechanism results for the CLB and the flooding hazards reevaluation are summarized in the table below.

Summary of Flood Mechanisms		
Mechanism	**Current License Basis (CLB)**	**Flood Hazard Reevaluation (FHR)**
Local Intense Precipitation (LIP)	32 in./6-hr. 38.7 in./12-hr 47.1 in./24-hr. 51.8 in./48-hr 55.7 in./72-hr [10 square-mile storm area] (1-hr duration not provided)	19.4 in/1-hr. with 4 Quartile Temporal Distributions [1 square-mile storm area]
LIP Flooding Depth	Surface Flooding not calculated; Only Rooftop accumulation calculated	*Maximum Flood Depth* 3.16 ft [Unit 1](1) 2.07 ft [Unit 2](2) Refer to Table 4-2 for all Points of Interest
Probable Maximum Flood in Rivers	n/a	n/a
Dam Breaches and Failures	n/a	n/a
PMSS (without wave runup)	Astronomical High Tide (4.6 ft-PSL Datum) Initial Rise (1.5 ft) Pressure Setup (3.8 ft) <u>+ Wind Setup (7.3 ft)</u> EL +17.2 ft-PSL Datum	10% Exceedance High Tide (5.09 ft-PSL Datum) 30 year Sea Level Rise (0.2 ft) Pressure and Wind Setup (12.36 ft) <u>+ Wave Setup (0.66 ft)</u> EL +18.3 ft-PSL Datum [Powerblock]
PMSS + Wave Runup and Combined Effects	*Wave runup evaluated at points near the Powerblock* EL +18.1 ft-PSL Datum [plant island southeast corner] EL +18.5 ft-PSL Datum [up south discharge canal] EL +18.8 ft-PSL Datum [Northern Unit 1] EL +28.0 ft-PSL Datum [nose of discharge canal]	*Wave runup evaluated at points of interest (doors) at the Powerblock* *(waves dissipate before reaching the Powerblock; i.e., 0 ft runup)* EL +18.3 ft-PSL Datum [Powerblock]
Seiche	Not Calculated	No Seiche-related flooding will occur
Tsunami	Not Calculated	10%Exceedance High Tide (5.09 ft-PSL Datum) 30 year Sea Level Rise (0.2 ft) <u>+ Tsunami Surge (12.33 ft)</u> Max Runup EL +17.62 ft-PSL Datum [from Cape Fear Landslide Source]
Ice Effects	n/a	n/a
Low Water Effects	*PMSS* -3.0 ft-PSL Datum [Big Mud Creek] -3.0 ft-PSL Datum [Atlantic Ocean]	*PMSS* -1.68 ft-PSL Datum [Big Mud Creek] -3.17 ft-PSL Datum [Atlantic Ocean] *Tsunami* -2.20 ft-PSL Datum [Hispaniola Trench – Atlantic Ocean] -1.71 ft-PSL Datum [Hispaniola Trench – Big Mud Creek -7.50 ft-PSL Datum [Cape Fear Landslide – Atlantic Ocean] -1.64 ft-PSL Datum [Cape Fear Landslide – Big Mud Creek]

n/a = not applicable

1. RAB and FHB.
2. Outside entry to H&V plenum CTMT Purge/H2 Purge Suction.

6.0 INTERIM EVALUATION AND ACTIONS

This section identifies interim actions to be taken before the Integrated Assessment is completed. It identifies the items to be addressed in the Integrated Assessment and the rationale for doing so.

6.1 Local Intense Precipitation

The new LIP analysis is for a duration of one hour, and the maximum depth of accumulated water in the Powerblock area is 3.16 ft. Interim evaluations will be performed to determine the potential effects of LIP-related flooding into buildings containing SSCs. This interim evaluation will model the time series hydrologic and hydraulic conditions at points of entry to the buildings and determine the influx and accumulation of water in the buildings. The evaluation will consider the passive, active, and institutional flood protection features. The evaluation for potential internal flooding from LIP was entered in the CAP. If it is determined that SSCs can be affected by LIP flooding, an interim measures plan will be formulated and implemented. It is not known at this time if, or which, SSCs could be potentially affected.

The results of the LIP analysis will be delivered to NRC upon completion. Subsequently, the interim flooding protection measures plan will be delivered upon availability.

This hazard will be addressed in the Integrated Assessment because the reevaluated levels exceed the CLB.

6.2 Riverine (Rivers and Streams) Flooding

No interim measures are required because riverine flooding does not apply to PSL. Therefore, this hazard will not be addressed in the Integrated Assessment.

6.3 Dam Breaches and Failure Flooding

No interim measures are required because dam breach-related flooding does not apply to PSL. Therefore, this hazard will not be addressed in the Integrated Assessment.

6.4 Storm Surge

No interim measures are required since the PMSS levels for this hazard would not adversely affect critical SSCs. However, since the reevaluation determined that the flood levels exceed the CLB, the effects of storm surge will be addressed in the Integrated Assessment.

6.5 Seiche

No interim measures are required because seiches were determined not to occur on the Indian River Lagoon. Therefore, this hazard will not be addressed in the Integrated Assessment.

6.6 Tsunami

No interim measures are required since the flooding levels for this hazard would not affect critical SSCs. The CLB excluded tsunami events based upon historical regional data; therefore, this hazard will be addressed in the Integrated Assessment.

6.7 Ice-Induced Flooding

No interim measures are required because ice-induced flooding is not a hazard at PSL. Therefore, this hazard will not be addressed in the Integrated Assessment.

6.8 Channel Diversion and Migration

No interim measures are required because channel diversion and migration are not issues at PSL. Therefore, this hazard will not be addressed in the Integrated Assessment.

6.9 Wind-Generated Waves

See Section 6.4.

6.10 Flooding-Related Loading

6.10.1 Local Intense Precipitation Related Loading

The CLB does not consider loading due to LIP; therefore, the reevaluated LIP loading conditions exceed the license basis and further action is necessary to evaluate the effects of LIP-related loading conditions. Because the loadings are relatively small, robust structures such as concrete buildings would not need to be evaluated. However, less robust structures such as doors and stoplogs need to be evaluated in the Integrated Assessment.

6.10.2 Probable Maximum Storm Surge Related Loading

The storm surge reevaluation determined an SWL of EL +18.3 ft-PSL Datum with a wave runup to EL +18.3 ft-PSL Datum (waves dissipate before reaching the Powerblock). This compares to the CLB of EL +17.2 ft-PSL Datum and 18.8 ft-PSL Datum for still water and wave runup, respectively.

The CLB is exceeded, but the new levels are below the physical level of protection for critical plant equipment. The reevaluation includes a sea level rise of 0.20 ft for the remainder of the current license. The available physical margin is 1.2 ft (19.5 ft – 18.3 ft = 1.2 ft) for still water and wave runup.

No interim measures are required since the flooding levels for this hazard would not affect critical SSCs (Section 5.10.2). The CLB does not consider hydrodynamic loading; however, the UFSAR considers subsurface hydrostatic loading. Therefore, these hazards will be addressed in the Integrated Assessment.

The intake channel is equipped with screens and netting for protection against damage from debris and waterborne projectiles, and no immediate action is necessary. However, the structural capacities of the screens and netting are not quantified; thus, they will be analyzed in the Integrated Assessment.

The cooling water inlet is submerged approximately 2 ft below the minimum intake channel water level. Floating debris may not be a concern since the impact location (2-ft thick concrete wall) has been designed for both seismic loads and tornado-generated missiles (NEE, 2014c). Because the concrete wall is designed to satisfy safety-related loading criteria, it is not considered to be an imminent concern; however, an evaluation will be performed in the Integrated Assessment to ensure that the structure can meet the PMSS loading conditions.

The UHS barrier is classified as a safety-related structure. Because this structure is designed to satisfy safety-related loading criteria, it is not considered to be an imminent concern; however, evaluations should be performed in the Integrated Assessment to ensure that the structure can meet the PMSS loading conditions.

The discharge canal sheet-piling barrier will be evaluated for PMSS loading in the Integrated Assessment.

6.10.3 Probable Maximum Tsunami Related Loading

The tsunami maximum wave runup evaluation determined a surge elevation of EL +17.62 ft-PSL Datum. The CLB did not evaluate a tsunami event. However, the available physical margin of 1.88 ft (19.5 ft – 17.62 ft = 1.88 ft) remains during the event; therefore, the tsunami would not affect critical SSCs.

No interim measures are required since the flooding levels for this hazard would not affect critical SSCs (Section 5.10.3). The CLB does not consider hydrodynamic loading; however, the UFSAR considers subsurface hydrostatic loading. Therefore, these hazards will be addressed in the Integrated Assessment.

The intake channel is equipped with screens and netting for protection against damage from debris and waterborne projectiles, and no immediate action is necessary. However, the structural capacities of the screens and netting are not quantified; thus, they will be analyzed during the Integrated Assessment.

The ICW structures are protected with a concrete wall which is designed to satisfy safety-related loading criteria and is not considered to be an imminent concern; however, an evaluation will be performed in the Integrated Assessment to ensure that the structure can meet the PMT loading conditions.

The UHS barrier is classified as a safety-related structure. Because this structure is designed to satisfy safety-related loading criteria, it is not considered to be an imminent concern; however, evaluations will be performed in the Integrated Assessment to ensure that the structure can meet the PMT loading conditions.

The intake and its appurtenances will need to be evaluated in the Integrated Assessment.

The reevaluated PMT event does not cause a credible flooding hazard for safety-related structures inside the plant island (EL +19.5 ft-PSL Datum).

The discharge canal sheet-piling barrier will be evaluated for PMT loading in the Integrated Assessment.

6.11 Debris and Sedimentation

No interim measures are required since the erosion hazard is conservatively analyzed in the CLB. Note that no credit was taken for physical modifications that have been made to the site that would improve the site's resistance to erosion and sedimentation. Further review of the CLB analysis will be performed in the Integrated Assessment to ensure it remains valid under the reevaluated flooding conditions.

6.12 Low Water Considerations

No interim measures are required because the reevaluated low water level is higher than the minimum low water operating level. Because the reevaluated low water elevation is lower than the CLB, this hazard will be addressed in the Integrated Assessment.

6.13 Combined Events Flooding

There are no interim measures related to combination flooding events other than those previously described in Sections 6.4 (PMSS) and 6.6 (PMT).

7.0 ADDITIONAL ACTIONS

There are no additional actions identified as of the date of this submittal.

8.0 REFERENCES

ANS, 1992, American National Standards Institute/American Nuclear Society (ANSI/ANS), "Determining Design Basis Flooding at Power Reactor Sites," ANSI/ANS-2.8-1992, La Grange Park, Illinois, July 28, 1992.

AOML, 2013, Atlantic Oceanographic & Meteorological Laboratory (AOML), "JEANNE Post Storm Wind Analysis," Available Online: http://www.aoml.noaa.gov/hrd/Storm_pages/jeanne2004/wind.html, Accessed on December 9, 2013.

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

Barkan et al., 2009, Barkan, R., U.S. ten Brink, and J. Lin, "Far field tsunami simulations of the 1755 Lisbon earthquake: Implications for tsunami hazard to the U.S. East Coast and the Caribbean," Marine Geology, Volume 264, pp. 109-122, 2009.

Battjes and Janssen, 1978, Battjes, J.A. and J.P.F.M. Janssen, "Energy Loss and Set-up due to Breaking of Random Waves," Proceedings of 16th International Conference on Coastal Engineering, pp. 569-587, American Society of Civil Engineers, New York, 1978.

Beven and Cobb, 2004, Beven, J. and H.D. Cobb III, National Hurricane Center, "Tropical Cyclone Report Hurricane Isabel 6-19 September 2003," Website, http://www.nhc.noaa.gov/2003isabel.shtml?, Revised January 16, 2004.

Bird and Kagan, 2004, Bird, P. and Y. Kagan, "Plate-tectonic analysis of shallow seismicity: apparent boundary width, beta-value, corner magnitude, coupled lithosphere thickness, and coupling in 7 tectonic settings," Bulletin of the Seismological Society of America, Vol. 94, pp. 2380-2399.

Blake et al., 2007, Blake, E.S., E.N. Rappaport, and C.W. Landsea, "The Deadliest, Costliest, and Most Intense United States Tropical Cyclones from 1851 to 2006 (and Other Frequently Requested Hurricane Facts)," Technical Memorandum NWS TPC-5, National Weather Service (NWS), National Hurricane Center, National Oceanic and Atmospheric Administration (NOAA), April 2007.

Blake et al., 2011, Blake, E.S., E.J. Gibney, and C.W. Landsea, "The Deadliest, Costliest, and Most Intense United States Tropical Cyclones from 1851 to 2010 (and Other Frequently Requested Hurricane Facts)," Technical Memorandum NWS TPC-6, National Weather Service (NWS), National Hurricane Center, National Oceanic and Atmospheric Administration (NOAA), August 2011.

Booij et al., 1999, Booij, N., L.H. Holthuijsen, and R. Ris, "A third-generation wave model for coastal regions, Part I, Model Description and Validation," Journal of Geophysical Research, Volume 104 (C4), pp. 7649-7666, 1999.

Cavaleri and Malanotte-Rizzoli, 1981, Cavaleri, L. and P. Malanotte-Rizzoli, "Wind Wave Prediction in Shallow Water: Theory and Applications," Journal of Geophysical Research, Volume 86, No. C11, 10, pp. 10,961-10,973, 1981.

Chaytor et al., 2009, Chaytor J.D., U.S. ten Brink, A.R. Aolow, and B.D. Andrews, "Size Distribution of Submarine Landslides along the US Atlantic Margin," Marine Geology 264, pp.16-27, 2009.

Churchill et al., 1995, Churchill, D.D., S.H. Houston, and N.A. Bond, "The Daytona Beach Wave of 3-4 July 1992: A Shallow-Water Gravity Wave Forced by a Propagating Squall Line," Bulletin of the American Meteorological Society, Volume 76, No. 1, pp. 21-32, January 1995.

Dababneh et al., 2012, Dababneh, J., B.K. Ferguson, and D.J. Barton, "Probable Maximum Tsunami Along the Dutch Coastline," Presented at ISOPE, Rhodes, Greece, June 17-22, 2012.

d'Angremond and van Roode, 2004, K. d'Angremond and F.C. van Roode, "Breakwaters and Closure Dams," Delft University Press, 2004.

Das, 2011, Das, B.M., "Principles of Foundation Engineering," Seventh Edition, PWS-Kent Publishing Company, 2011.

Dean and Dalrymple, 2007, Dean, R.G. and R.A. Dalrymple, Water Wave Mechanics for Engineers and Scientists, World Scientific, 363 pp.

Deltares, 2011a, Deltares, "Delft3D-RGFGRID, Generation and manipulation of curvilinear grids for Delft3D-FLOW and Delft3D-WAVE, User Manual Version 4.00, Revision 15423," 2600 MH Delft, The Netherlands, May 18, 2011.

Deltares, 2011b, Deltares, "Delft3D-QUICKIN, Generation and manipulation of grid-related parameters such as bathymetry, initial conditions and roughness, User Manual Version 4.00, Revision 15423," 2600 MH Delft, The Netherlands, May 18, 2011.

Deltares, 2011c, Deltares, "Delft3D-FLOW, Simulation of multi-dimensional hydrodynamic flows and transport phenomena, including sediments, User Manual Version 3.15, Revision 18392," 2600 MH Delft, The Netherlands, September 7, 2011.

Deltares, 2011d, Deltares, Delft3D Hydro-Morphodynamics and Water Quality Computer Program, Version 4.00.01, 2600 MH Delft, The Netherlands, 2011.

Deltares, 2011e, Deltares, "DELFT3D-WAVE, Simulation of short-crested waves with SWAN, User Manual Version 3.04, Revision 15779," 2600 MH Delft, The Netherlands, May 18, 2011.

Deltares, 2012, Deltares, "International Hydrographic Office (IHO) Tide Stations," Data assembled by Maarten van Ormondt at Deltares, Available at: http://opendap.deltares.nl/static/deltares/delftdashboard/toolboxes/TideStations/iho.nc, Accessed on December 9, 2013.

Dietrich et al., 2013, Dietrich, J.C., M. Zijlema, P.E. Allier, L.H. Holthuijsen, N. Booij, J.D. Meixner, J.K. Proft, C.N. Dawson, C.J. Bender, A. Naimaster, J.M. Smith, and J.J. Westerink, "Limiters for Spectral Propagation Velocities in SWAN," Ocean Modeling, Volume 70, pp. 85-102, DOI: 10.1016/j.ocemod.2012.11.005.

Egbert and Erofeeva, 2002, Egbert, Gary D. and Svetlana Y. Erofeeva, "Efficient Inverse Modeling of Barotropic Ocean Tides," Journal of Atmospheric and Oceanic Technology, Volume 19, Issue 2, pp. 183-204, 2002.

Egbert et al., 1994, Egbert, Gary D., Andrew F. Bennett, and Michael G.G. Foreman, "TOPEX/POSEIDON Tides Estimated Using a Global Inverse Model," Journal of Geophysical Research, Volume 99, No. C12, pp. 24, 821-852, December 1994.

ESRI, 2014a, Environmental Systems Research Institute (ESRI), World Imagery Map Service, Updated October 2014, http://services.arcgisonline/arcgis/services, Accessed August 2013 to October 2014.

ESRI, 2014b, Environmental Systems Research Institute (ESRI), World Street Map Service, Updated September 2014, http://services.arcgisonline/arcgis/services, Accessed August 2013 to October 2014.

ESRI, 2014c, Environmental Systems Research Institute (ESRI), World Topography Map Service, Updated October 2014, http://services.arcgisonline/arcgis/services, Accessed August 2013 to October 2014.

ESRI, 2014d, Environmental Systems Research Institute (ESRI), Ocean Basemap Service, Updated March 2013, http://services.arcgisonline/arcgis/services, Accessed August 2013 to October 2014.

FAWN, 2014, Florida Automated Weather Network (FAWN), ftp://agrofawn-prod01.osg.ufl.edu/fawnpub/data/15_minute_obs/, Accessed on May 13, 2014.

Fine et al., 2005, Fine, I.V., A.B. Rabinovich, B.D. Bornhold, R.E. Thomson, and E.A. Kulikov, "The Grand Banks landslide-generated tsunami of November 18, 1929: preliminary analysis and numerical modeling," Marine Geology, Volume 215, pp. 45-57, 2005.

Finnemore and Franzini, 2002, E. John Finnemore and Joseph B. Franzini, "Fluid Mechanics with Engineering Applications," Pearson Prentice Hall, 2002.

FLO-2D, 2012, FLO-2D Software, Inc. (FLO-2D), "FLO-2D Reference Manual," Updated September 1, 2012.

FLO-2D, 2013, FLO-2D Software, Inc. (FLO-2D), FLO-2D Pro Model Software, vPRO Build 13.11.06.

FPL, 1994, Florida Power and Light (FPL), "St. Lucie Units 1 and 2, Docket 50-335 and 50-389, NRC Generic Letter 88-20 Supplement 4, Individual Plant Examination of External Events for Severe Accidents Vulnerabilities Report," L-94-318, December 15, 1994.

FPL, 2012, Florida Power and Light (FPL), "Flooding Walkdown Report in Response to the 50.54(f) Information Request Regarding Near-Term Task Force Recommendation 2.3: Flooding for the St. Lucie Plant," FPL060-PR-001, Rev. 0, November 26, 2012, ADAMS Accession No. ML12335A201 and ML12335A202.

FPL, 2014a, Florida Power and Light (FPL), "Station Housekeeping and Material Control, Procedure No. MA-AA-100-1008, Rev 6," January 27, 2014.

FPL, 2014b, Florida Power and Light (FPL), "Flooding Walkdown Report in Response to the 50.54(f) Information Request Regarding Near-Term Task Force Recommendation 2.3: Flooding for the St. Lucie Plant," FPL060-PR-001, Rev. 1, September 23, 2014.

FPL, 2014c, Florida Power and Light (FPL), "Intake Intrusion Monitoring and Mitigation, Procedure No. 1-NOP-21.15, Rev 9," September 12, 2014.

FPL, 2014d, Florida Power and Light (FPL), "Severe Weather Preparations, Procedure No. 0005753, Rev 75," September 29, 2014.

Franklin, 2008, Franklin, J.L., National Hurricane Center, "Tropical Cyclone Report Hurricane Dean (AL042007), 13-23 August 2007," Website, http://www.nhc.noaa.gov/pdf/TCR-AL042007_Dean.pdf, Revised April 7, 2008.

GEBCO, 2008, General Bathymetric Chart of the Oceans (GEBCO), Gridded Bathymetric Data: GEBCO_08 Grid (2008), released 2008, GEBCO Website, http://www.gebco.net/data_and_products/gridded_bathymetry_data/, Accessed August 2012.

GEBCO, 2014, General Bathymetric Chart of the Oceans (GEBCO), Frequently Asked Questions: GEBCO's grids and vertical datum, GEBCO Website, http://www.gebco.net/general_interest/faq/, updated June 11, 2014.

Geist and Parsons, 2009, Geist, E.L. and T. Parsons, "Assessment of source probabilities for potential tsunamis affecting the U.S. Atlantic coast," Marine Geology, Volume 264, pp. 98-108, March 11, 2009.

Giardino et al., 2010, Giardino, A., J. van der Werf, and M. van Ormondt, "Simulating Coastal Morphodynamics with Delft3D: Case Study Egmond aan Zee," Deltares Delft Hydraulics, 2010.

Gisler et al., 2006, Gisler, G., R. Weaver, and M.L. Gittings, "Sage Calculations of the Tsunami Threat from La Palma," Science of Tsunami Hazards, Volume 24, No. 4, pp. 288-301, 2006.

Guiney and Lawrence, 2000, Guiney, J.L. and M.B. Lawrence, National Hurricane Center, "Preliminary Report Hurricane Mitch 22 October – 05 November 1998," Website, http://www.nhc.noaa.gov/1998mitch.html, Revised May 4, 2000.

Haflidason et al., 2004, Haflidason, H., H.P. Sejrup, A. Nygard, J. Mienert, P. Bryn, R. Lien, C.F. Forsberg, K. Berg, and D. Masson, "The Storegga Slide: architecture, geometry and slide development," Marine Geology, Volume 213, pp. 201-234, 2004.

Hasegawa and Kanamori, 1987, Hasegawa, H.S. and H. Kanamori, "Source Mechanism of the Magnitude 7.2 Grand Banks Earthquake of November 1929: Double Couple or Submarine Landslide?" Bulletin of the Seismological Society of America, Volume 77, No. 6, pp. 1984-2004, 1987.

Hasselmann et al., 1973, Hasselmann, K., T.P. Barnett, E. Bouws, H. Carlson, D.E. Cartwright, K. Enke, J.A. Ewing, H. Gienapp, D.E. Hasselmann, P. Kruseman, A. Meerburg, P. Müller, D.J. Olbers, K. Richter, W. Sell, and H. Walden, "Measurements of Wind-Wave Growth and Swell Decay During the Joint North Sea Wave Project (JONSWAP)," Deutsches Hydrographisches Institute, Reihe A8, NR. 12, Hamburg, 1973.

Hasselmann et al., 1985, Hasselmann, S., K. Hasselmann, J.H. Allender, and T.P. Barnett, "Computations and Parameterizations of the Nonlinear Energy Transfer in a Gravity-Wave Spectrum, Part II, Parameterizations of the Nonlinear Energy Transfer for Application in Wave Models," Journal of Physical Oceanography, Volume 15, pp. 1378-1391, 1985.

Herbert, 1987, Herbert, Dave, "An Estimate of the Effective Horizontal Eddy Viscosity in the Gulf Stream Due to Internal Waves," American Meteorological Society, Volume 17, October 1987, pp. 1837-1841, 1987.

H.M.S. Weather Bureau, 1968, U.S. Department of Commerce, E.S.S.A., Memorandum HUR 7-97, "Interim Report – Meteorological Characteristics of the Probable Maximum Hurricane, Atlantic and Gulf Coasts of the United States," H.M.S. Weather Bureau, May 7, 1968.

Hornbach et al., 2007, Hornbach, M.J., L. Lavier, and C. Ruppel, "Triggering Mechanism and Tsunamogenic Potential of the Cape Fear Slide Complex, U.S. Atlantic Coastal Margin," Geochemistry, Geophysics, and Geosystems, Volume 8, No. 10, 2007.

ICAO, 2005, International Civil Aviation Organization (ICAO), "Needs for Upgrading the United States Tsunami Warning System in the Caribbean and East Coast," CAR/SAR IP/01.

Ichinose et al., 2000, Ichinose, G.A., J.G. Anderson, K. Satake, R.A. Schweickert, and M.M. Lahren, "The potential hazard from tsunami and seiche waves generated by large earthquakes within Lake Tahoe, California-Nevada," Geophysical Research Letters, Vol. 27, Issue 8, pp. 1203-1216, 2000.

IHE, 2003, Vu Thi Thu Thuy, International Institute for Infrastructural, Hydraulic and Environmental Engineering (IHE Delft), "Storm Surge Modelling for Vietnam's Coast," M.Sc. Thesis, April 2003.

IOC, 2006, Intergovernmental Oceanographic Commission (IOC), United Nations Educational Scientific, and Cultural Organization (UNESCO) and International Tsunami Information Centre, "Tsunami Glossary," January 2006.

IPCC, 2007, Intergovernmental Panel on Climate Change (IPCC), "Climate Change 2007: Synthesis Report," Core Writing Team, R.K. Pachauri and A. Reisinger (eds.), Geneva, Switzerland, 2007.

IPET, 2009, Interagency Performance Evaluation Task Force (IPET), "Performance evaluation of the New Orleans and southeast Louisiana hurricane protection system, Volume VIII – Engineering and Operational Risk and Reliability Analysis (Including Appendix 8)," U.S. Army Corps of Engineers (USACE), Washington, D.C., 2009.

Johnson, 1996, Johnson, A.C., "Seismic moment assessment of earthquakes in stable continental regions – III. New Madrid 1811-1812, Charleston 1886 and Lisbon 1755," Geophysical Journal International, Volume 126, pp. 314-344.

Kim, 2010, Young C. Kim, "Handbook of Coastal and Ocean Engineering," edited by Young C. Kim, World Scientific Publishing Company, 2010.

Knabb et al., 2006a, Knabb, R.D., J.R. Rhome and D.P. Brown, National Hurricane Center, "Tropical Cyclone Report Hurricane Katrina 23-30 August 2005," Website, http://www.nhc.noaa.gov/pdf/TCR-AL122005_Katrina.pdf, Updated August 10, 2006.

Knabb et al., 2006b, Knabb, R.D. et al., National Hurricane Center, "Tropical Cyclone Report Hurricane Rita 18-26 September 2005," Website, http://www.nhc.noaa.gov/pdf/TCR-AL182005_Rita.pdf, Updated August 14, 2006.

Knight, 2006, Knight, B., "Model Predictions of Gulf and Southern Atlantic Coast Tsunami Impacts from a Distribution of Sources," Science of Tsunami Hazards, Volume 24, No. 5, pp. 304-312, 2006.

Kojima, 2005, Kojima, Satoshi, "Optimization of an Unstructured Finite Element Mesh for Tide and Storm Surge Modeling Applications in the Western North Atlantic Ocean," M.Sc. Thesis, University of Central Florida, 2005.

Komen et al., 1984, Komen, G.J., S. Hasselmann, and K. Hasselmann, "On the Existence of A Fully Developed Wind-Sea Spectrum," Journal of Physical Oceanography, Volume 14, pp. 1271-1285, August 1984.

Komen et al., 1994, Komen, G.J., L. Cavaleri, M. Donelan, K. Hasselmann, S. Hasselmann, and P.A.E.M. Janssen, "Dynamics and Modelling of Ocean Waves," Cambridge University Press, New York, 1994.

Locat et al., 2009, Locat, J., L. Homa, U. ten Brink, D. Twichell, E. Geist and M. Sansoucy, "Geomorphology, Stability and Mobility of the Currituck Slide," Marine Geology, Vol. 264, pp. 28–40, 2009.

Mader, 2001a, Mader, C.L., "Modeling the 1755 Lisbon Tsunami," Science of Tsunami Hazards, Volume 19, pp. 93-98, 2001.

Mader, 2001b, Mader, C.L., "Modeling the LaPalma Landslide Tsunami," Science of Tsunami Hazards, Volume 19, pp. 150-170, 2001.

Mayfield, 1995, Mayfield, M., National Hurricane Center, "Preliminary Report Hurricane Opal 29 November 1995," Website, http://www.nhc.noaa.gov/1995opal.html, November 29, 1995.

Munson et al., 2006, Munson, B., D. Young, and T. Okiishi, "Fundamentals of Fluid Mechanics Fifth Edition," John Wiley & Sons, Inc., 2006.

NASA, 2010, National Aeronautics and Space Administration (NASA), "Air Properties Definitions," Available at: https://www.grc.nasa.gov/WWW/Wright/airplane/airprop508.html, Accessed on January 1, 2013, Page last modified on March 24, 2010.

Nash and Sutcliffe, 1970, J.E. Nash and J.V. Sutcliffe, "River flow forecasting through conceptual models part 1 – A discussion of principles," Journal of Hydrology, Vol. 10, Issue 3, pp. 282-290, 1970.

NEE, 1975a, NextEra Energy (NEE), St. Lucie Nuclear Power Plant (PSL), "Preliminary Safety Analysis Report Supplement 1 to Appendix 2H Response to NRC Request for Additional Information on Stalled Hurricanes – For St. Lucie Plant Unit No. 2," Revision 37, August 22, 1975.

NEE, 1975b, NextEra Energy (NEE), St. Lucie Nuclear Power Plant (PSL), "Preliminary Safety Analysis Report Supplement 2 to Appendix 2H Response to NRC Request for Additional Information on Stalled Hurricanes – For St. Lucie Plant Unit No. 2," Revision 41, October 31, 1975.

NEE, 2012, NextEra Energy (NEE), "FPL/St. Lucie Plant's 90 Day Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding Flooding Aspects of Recommendations 2.1 and 2.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident," L-2012-241, June 11, 2012.

NEE, 2013a, NextEra Energy (NEE), "Hurricane Season Preparation," St. Lucie Plant (PSL) Administrative Procedure ADM-04.01, Revision 27, September 13, 2013.

NEE, 2013b, NextEra Energy (NEE), St. Lucie Nuclear Power Plant (PSL), "Updated Final Safety Analysis Report, Unit 2," Amendment 21, May 23, 2013.

NEE, 2014a, NextEra Energy (NEE), "Severe Weather Preparations," St. Lucie Plant (PSL) Administrative Procedure 0005753, Revision 74, July 1, 2014.

NEE, 2014b, NextEra Energy (NEE), "Fukushima Flooding Re-evaluation Design Information Transmittal," DIT-001, St. Lucie Plant (PSL), August 18, 2014, *Description of Information: Plant Datum, Bathymetric Survey for Big Mud Creek, and signed copy of the Final PSL Topographic Survey Report.*

NEE, 2014c, NextEra Energy (NEE), "Fukushima Flooding Re-evaluation Design Information Transmittal," DIT-002, St. Lucie Plant (PSL), August 19, 2014, *Description of Information: PSL FSAR Signed Cover Page, Safety Classification of the Inlet Canal Weir (Ultimate Heat Sink Dam) and PSL Drawings 2998-G-644 and 8770-G-644.*

NEE, 2014d, NextEra Energy (NEE), "Fukushima Flooding Re-evaluation Design Information Transmittal," DIT-003, St. Lucie Plant (PSL), August 27, 2014, *Description of Information: Points-of-Interest Documentation and operations actions for the Ultimate Heat Sink.*

NEE, 2014e, NextEra Energy (NEE), St. Lucie Nuclear Power Plant (PSL), "Updated Final Safety Analysis Report, Unit 1," Amendment 26, May 9, 2014.

NEI, 2012, Nuclear Energy Institute (NEI), "Guidelines for Performing Verification Walkdowns of Plant Flood Protection Features," NEI 12-07, Rev. 0, May 2012, ADAMS Accession No. ML12144A401.

NOAA, 1967, National Oceanic and Atmospheric Administration (NOAA), National Ocean Survey (NOS) Hydrographic Survey: H08955, created 1967, National Geophysical Data Center (NGDC): Bathymetry & Digital Elevation Models Website, http://maps.ngdc.noaa.gov/viewers/bathymetry/, Accessed July 2013.

NOAA, 1978, National Oceanic and Atmospheric Administration (NOAA), "Hydrometeorological Report No. 51, Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Washington, D.C., June 1978.

NOAA, 1979, National Oceanic and Atmospheric Administration (NOAA), National Ocean Survey (NOS) Hydrographic Surveys: H05027, H05028, H05031, H05032, H05047, H08783, H08839, and H08957, created 1979, National Geophysical Data Center (NGDC): Bathymetry & Digital Elevation Models Website, http://maps.ngdc.noaa.gov/viewers/bathymetry/, Accessed July 2013.

NOAA, 1982, National Oceanic and Atmospheric Administration (NOAA), National Weather Service (NWS), "Hydrometeorological Report No. 52, Application of Probable Maximum Precipitation Estimates – Unit^{ed} States East of the 105th Meridian," Washington, D.C., August 1982.

NOAA, 1986, National Oceanic Atmospheric Administration (NOAA), National Ocean Service (NOS), U.S. Department of Commerce, "The National Geodetic Survey Gravity Network," NOAA Technical Report NOS 121 NGS 39, Rockville, Maryland, December 1986.

NOAA, 1999, National Oceanic Atmospheric Administration (NOAA), "Preliminary Report Hurricane Floyd, 7 – 17 September, 1999," National Hurricane Center, Available at: http://www.nhc.noaa.gov/1999floyd.html, Accessed on December 9, 2013.

NOAA, 2000, National Oceanic Atmospheric Administration (NOAA), "Effects of Hurricane Floyd on Water Levels Data Report," NOAA Technical Report NOA CO-OPS 027.

NOAA, 2002, National Oceanic and Atmospheric Administration (NOAA), National Ocean Survey (NOS) Hydrographic Survey: H08958, created 2002, National Geophysical Data Center (NGDC): Bathymetry & Digital Elevation Models Website, http://maps.ngdc.noaa.gov/viewers/bathymetry/, Accessed July 2013.

NOAA, 2004, National Oceanic and Atmospheric Administration (NOAA), National Ocean Survey (NOS) Hydrographic Surveys: H01513A, H01513B, H01570, H01571, H04914, H01419A, H01419B, H08956, H08954, H08955, and H08959, created 2004, National Geophysical Data Center (NGDC): Bathymetry & Digital Elevation Models Website, http://maps.ngdc.noaa.gov/viewers/bathymetry/, Accessed July 2013.

NOAA, 2004a, National Oceanic Atmospheric Administration (NOAA), "Tropical Cyclone Report Hurricane Frances 25 August – 8 September 2004," National Hurricane Center, Available at: http://www.nhc.noaa.gov/2004frances.shtml, Page last modified June 11, 2005, Accessed on December 9, 2013.

NOAA, 2004b, National Oceanic Atmospheric Administration (NOAA), "Tropical Cyclone Report Hurricane Jeanne 13 – 28 September 2004," National Hurricane Center, Available at: <http://www.nhc.noaa.gov/2004jeanne.shtml>, Page last modified on March 15, 2005, Accessed on December 9, 2013.

NOAA, 2004c, National Oceanic Atmospheric Administration (NOAA), "Hurricane Frances Preliminary Water Levels Report," National Ocean Service (NOS) Center for Operational Oceanographic Products and Services, September 2004.

NOAA, 2004d, National Oceanic Atmospheric Administration (NOAA), "Hurricane Jeanne Preliminary Water Levels Report," National Ocean Service (NOS) Center for Operational Oceanographic Products and Services, October 2004.

NOAA, 2005, National Oceanic and Atmospheric Administration (NOAA), National Ocean Survey (NOS) Hydrographic Survey: H08713, created 2005, National Geophysical Data Center (NGDC): Bathymetry & Digital Elevation Models Website, http://maps.ngdc.noaa.gov/viewers/bathymetry/, Accessed July 2013.

NOAA, 2006, National Oceanic and Atmospheric Administration (NOAA), National Hurricane Center, FAQ/State of the Science: Atlantic Hurricane & Climate, National Hurricane Center Website, http://hurricanes.noaa.gov/pdf/hurricanes-and-climate-change-09-2006.pdf, updated December 12, 2006.

NOAA, 2007, National Oceanic and Atmospheric Administration (NOAA), "Scientific and Technical Issues in Tsunami Hazard Assessment of Nuclear Power Plant Sites," NOAA Technical Memorandum OAR PMEL-136, Science Review Working Group, Pacific Marine Laboratory, May 2007.

NOAA, 2011a, National Oceanic Atmospheric Administration (NOAA), "Tropical Cyclone Report Hurricane Irene (AL092011) 21-28 August 2011," National Hurricane Center.

NOAA, 2011b, National Oceanic Atmospheric Administration (NOAA), "NOAA Water Level and Meteorological Data Report Hurricane Irene," National Ocean Service (NOS) Center for Operational Oceanographic Products and Services, October 2011.

NOAA, 2011d, National Oceanic Atmospheric Administration (NOAA), Gica, E., "Development of a Tsunami Forecast Model for Palm Beach, Florida, USA," NOAA Report, May 24, 2011.

NOAA, 2012, National Oceanic and Atmospheric Administration (NOAA), Hurricane Research Division, Atlantic Oceanographic & Meteorological Laboratory, Wind Field Analysis Data, Website, http://www.aoml.noaa.gov/hrd/data_sub/wind.html, Updated through 2012.

NOAA, 2013a, National Oceanic Atmospheric Administration (NOAA), "Tides and Currents," Available Online: http://tidesandcurrents.noaa.gov/, Accessed on December 9, 2013.

NOAA, 2013b, National Oceanic Atmospheric Administration (NOAA), "National Data Buoy Center," Available Online: http://www.ndbc.noaa.gov/, Accessed on December 9, 2013.

NOAA, 2013c, National Oceanic Atmospheric Administration (NOAA), "Booklet Chart St Lucie Inlet to Fort Meyers and Lake Okeechobee NOAA Chart 11428," Available Online: http://ocsdata.ncd.noaa.gov/BookletChart/11428_BookletChart.pdf, Accessed on December 19, 2013.

NOAA, 2013d, National Oceanic Atmospheric Administration (NOAA), Sanja Perica, Deborah Martin, Sandra Pavlovic, Ishani Roy, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Michael Yekta, and Geoffrey Bonnin, NOAA Atlas 14, Volume 9, Version 2, Precipitation Frequency Analysis of the United States, Southeastern States (Alabama, Arkansas, Florida, Georgia, Louisiana, Mississippi), NOAA, National Weather Service (NWS), Silver Spring, Maryland, 2013.

NOAA, 2013e, National Oceanic Atmospheric Administration (NOAA), "Tidal Datums – NOAA Tides & Currents," Revised October 15, 2013, Available Online: http://tidesandcurrents.noaa.gov/datum_options.html, Accessed on November 15, 2013.

NOAA, 2013f, National Oceanic Atmospheric Administration (NOAA), "Frequently Asked Questions – NOAA Tides & Currents," Revised October 15, 2013, Available Online: http://www.tidesandcurrents.noaa.gov/est/faq, Accessed on November 15, 2013.

NOAA, 2013g, National Oceanic Atmospheric Administration (NOAA), "Sea Level Trends – NOAA Tides & Currents," Webpage last revised October 15, 2013, Available Online: http://tidesandcurrents.noaa.gov/sltrends/, Accessed on November 15, 2013.

NOAA, 2013h, National Oceanic Atmospheric Administration (NOAA), "Historic Tide Data, Monthly Means, Gage 8723214, Virginia Key, Florida," Available at: http://tidesandcurrents.noaa.gov/data_menu.shtml?bdate=19300411&edate=20130412&wl_sensor_hist=W5&relative=&datum=7&unit=1&shift=g&stn=8723214+Virginia+Key%2C+FL&type=Historic+Tide+Data&format=View+Data, Webpage last revised on October 15, 2013, Accessed on November 15, 2013.

NOAA, 2013i, National Oceanic Atmospheric Administration (NOAA), "Historic Tide Data, Monthly Means, Gage 8722670, Lake Worth Pier, Florida," Available at: http://tidesandcurrents.noaa.gov/data_menu.shtml?bdate=19300411&edate=20130412&wl_sens_hist=W5&relative=&datum=7&unit=1&shift=g&stn=8722670+Lake+Worth+Pier%2C+FL&type=Historic+Tide+Data&format=View+Data, Webpage last revised on October 15, 2013, Accessed on November 15, 2013.

NOAA, 2013j, National Oceanic Atmospheric Administration (NOAA), "Historic Tide Data, Monthly Means, Gage 8721604, Trident Pier, Florida," Available at: http://tidesandcurrents.noaa.gov/data_menu.shtml?bdate=20130411&edate=20130412&wl_sensor_hist=W5&relative=&datum=7&unit=1&shift=g&stn=8721604+Trident+Pier%2C+FL&type=Historic+Tide+Data&format=View+Data, Webpage last revised on October 15, 2013, Accessed on November 15, 2013.

NOAA, 2013k, National Oceanic Atmospheric Administration (NOAA), "Historic Tide Data, Monthly Means, Gage 8720030, Fernandina Beach, Florida," Available at: http://tidesandcurrents.noaa.gov/data_menu.shtml?bdate=18500419&edate=20130420&wl_sensor_hist=W5&relative=&datum=7&unit=1&shift=g&stn=8720030+Fernandina+Beach%2C+FL&type=Historic+Tide+Data&format=View+Data, Webpage last revised on October 15, 2013, Accessed on November 15, 2013.

NOAA, 2013l, National Oceanic Atmospheric Administration (NOAA), Coastal Services Center (CSC), "Historical Hurricane Tracks," Available at: http://www.csc.noaa.gov/hurricanes/, Accessed on September 4, 2013.

NOAA, 2013m, National Oceanic Atmospheric Administration (NOAA), "8720030 Fernandina Beach, FL Data Inventory – NOAA Tides and Currents," Available at: http://tidesandcurrents.noaa.gov/inventory.html?id=8720030, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013n, National Oceanic Atmospheric Administration (NOAA), "8724580 Key West, FL Data Inventory – NOAA Tides and Currents," Available at: http://tidesandcurrents.noaa.gov/inventory.html?id=8724580, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013o, National Oceanic Atmospheric Administration (NOAA), "8720218 Mayport (Bar Pilots Dock), FL Data Inventory – NOAA Tides and Currents," Available at: http://tidesandcurrents.noaa.gov/inventory.html?id=8720218, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013p, National Oceanic Atmospheric Administration (NOAA), "8721604 Trident Pier, FL Data Inventory – NOAA Tides and Currents," Available at: http://tidesandcurrents.noaa.gov/inventory.html?id=8721604, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013q, National Oceanic Atmospheric Administration (NOAA), "8723970 Vaca Key, FL Data Inventory – NOAA Tides and Currents," Available at: http://tidesandcurrents.noaa.gov/inventory.html?id=8723970, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013r, National Oceanic Atmospheric Administration (NOAA), "8723214 Virginia Key, FL Data Inventory – NOAA Tides and Currents," Available at: http://tidesandcurrents.noaa.gov/inventory.html?id=8723214, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013s, National Oceanic Atmospheric Administration (NOAA), "8722670 Lake Worth Pier, FL Data Inventory – NOAA Tides and Currents," Available at: http://tidesandcurrents.noaa.gov/inventory.html?id=8722670, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013t, National Oceanic Atmospheric Administration (NOAA), "9755371 San Juan, PR Data Inventory – NOAA Tides and Currents," Available at: http://tidesandcurrents.noaa.gov/inventory.html?id=9755371, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013u, National Oceanic Atmospheric Administration (NOAA), "Saffir-Simpson Hurricane Wind Scale," Available at: http://www.nhc.noaa.gov/aboutsshws.php, Webpage last revised on May 24, 2013, Accessed on December 9, 2013.

NOAA, 2013v, National Oceanic Atmospheric Administration (NOAA), "Historical Hurricane Tracks," Available at: http://csc.noaa.gov/hurricanes/, Accessed on December 9, 2013.

NOAA, 2013w, National Oceanic Atmospheric Administration (NOAA), "Datum – NOAA Tides and Currents (Vero Beach, FL)," http://tidesandcurrents.noaa.gov/datums.html?id=8722125, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013x, National Oceanic Atmospheric Administration (NOAA), "Datums – NOAA Tides and Currents (Fernandina Beach, FL)," Available Online: http://tidesandcurrents.noaa.gov/datums.html?id=8720030, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013y, National Oceanic Atmospheric Administration (NOAA), "Datums – NOAA Tides and Currents (Mayport (Bar Pilots Dock), FL)," Available Online: http://tidesandcurrents.noaa.gov/datums.html?id=8720218, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013z, National Oceanic Atmospheric Administration (NOAA), "Datums – NOAA Tides and Currents (Augustine Beach, FL)," Available Online: http://tidesandcurrents.noaa.gov/datums.html?id=8720587, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013aa, National Oceanic Atmospheric Administration (NOAA), "Datums – NOAA Tides and Currents (Trident Pier, FL)," Available Online: http://tidesandcurrents.noaa.gov/datums.html?id=8721604, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013ab, National Oceanic Atmospheric Administration (NOAA), "Datums – NOAA Tides and Currents (Lake Worth Pier, FL)," Available Online: http://tidesandcurrents.noaa.gov/datums.html?id=8722670, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013ac, National Oceanic Atmospheric Administration (NOAA), "Datums – NOAA Tides and Currents (Virginia Key, FL)," Available Online: http://tidesandcurrents.noaa.gov/datums.html?id=8723214, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013ad, National Oceanic Atmospheric Administration (NOAA), "Datums – NOAA Tides and Currents (San Juan, PR)," Available Online: http://tidesandcurrents.noaa.gov/datums.html?id=9755371, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2013ae, National Oceanic Atmospheric Administration (NOAA), "8720587 St. Augustine, FL Data Inventory – NOAA Tides and Currents," Available at: http://tidesandcurrents.noaa.gov/inventory.html?id=8720587, Webpage last revised on October 15, 2013, Accessed on December 9, 2013.

NOAA, 2014a, National Oceanic and Atmospheric Administration, National Data Buoy Center (NOAA-NDBC) – Station 41009-CANAVERAL 20 nm East of Cape Canaveral, Available at: http://www.ndbc.noaa.gov/station_history.php?station=41009, Accessed on May 13, 2014.

NOAA, 2014b, National Oceanic and Atmospheric Administration, National Climatic Data Center (NOAA-NCDC), Available at: https://gis.ncdc.noaa.gov/map/viewer/#app=cdo, Accessed on May 13, 2014.

NOAA, 2014c, National Oceanographic and Atmospheric Administration (NOAA), National Geophysical Data Center/World Data Service (NGDC/WDS): Global Historical Tsunami Database, National Geophysical Data Center, Available at: http://ngdc.noaa.gov/hazard/tsu_db.shtml, Accessed May 2014.

NRC, 2005, "Operability Determinations & Functionality Assessments for Resolution of Degraded or Nonconforming Conditions Adverse to Quality or Safety." NRC Inspection Manual. Part 9900: Technical Guidance. Regulatory Issues Summary 2005-20, Revision 1, September 26, 2005.

NRC, 2007, U.S. Nuclear Regulatory Commission (NRC), "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," NUREG-0800 (Formally issued as NUREG-75/087), Washington, D.C., Revision 3, March 2007.

NRC, 2009, U.S. Nuclear Regulatory Commission (NRC), "Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America – Final Report," NUREG/CR-6966, March 2009, ADAMS Accession No. ML091590193.

NRC, 2011, U.S. Nuclear Regulatory Commission (NRC), "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," NUREG/CR-7046, Washington, D.C., November 2011, ADAMS Accession No. ML11321A195.

NRC, 2012a, U.S. Nuclear Regulatory Commission (NRC), "Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3 and 9.3, of the Near-Term Task Force Review of Insights From the Fukushima Dai-ichi Accident," March 12, 2012, ADAMS Accession No. ML12053A340.

NRC, 2012b, U.S. Nuclear Regulatory Commission (NRC), "Endorsement of Nuclear Energy Institute (NEI) 12-07 'Guidelines for Performing Verification Walkdowns of Plant Flood Protection Features'," May 31, 2012, ADAMS Accession No. ML12144A142.

NRC, 2013a, U.S. Nuclear Regulatory Commission (NRC), "Guidance for Performing a Tsunami, Surge and Seiche Flooding Safety Analysis Revision 0," Japan Lessons-Learned Project Directorate Interim Staff Guidance, JLD-ISG-2012-06, January 4, 2013, ADAMS Accession No. ML12314A412.

NRC, 2013b, U.S. Nuclear Regulatory Commission (NRC), "Guidance for Assessment of Flooding Hazards Due to Dam Failure," Japan Lessons-Learned Project Directorate Interim Staff Guidance, JLD-ISG-2013-01, Revision 0, July 29, 2013, ADAMS Accession No. ML13151A153.

NRC, 2013c, U.S. Nuclear Regulatory Commission (NRC), "Operating Nuclear Power Reactors by Location or Name," Available at: http://www.nrc.gov/info-finder/reactor/index.html, Accessed on June 13, 2013.

NWS, 1979, National Weather Service (NWS), Technical Report 23 (NWS23), "Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coast of the United States," U.S. Department of Commerce, National Oceanic Atmospheric Administration (NOAA), September 1979.

NWS, 1987, National Weather Service (NWS), Technical Report 38 (NWS38), "Hurricane Climatology for the Atlantic and Gulf Coasts of the United States," U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA), April 1987.

NWS, 2004, National Weather Service (NWS), "Tropical Cyclone Report: Hurricane Jeanne, 13-28 September 2004," January 7, 2005.

Okada, 1985, "Surface Deformation Due to Shear and Tensile Faults in a Half-Space," Bulletin of the Seismological Society of America, Vol. 75, No. 4, pp. 1135–1154, 1985.

OSU, 2012, Oregon State University (OSU), "The OSU TOPEX/Poseidon Global Inverse Solution TPXO," Available at: http://volkov.oce.orst.edu/tides/global.html, Accessed on December 26, 2012.

Pasch et al., 2006a, Pasch, R.J. et al., National Hurricane Center, "Tropical Cyclone Report Hurricane Wilma 15-25 October 2005," Website, http://www.nhc.noaa.gov/pdf/TCR- AL252005_Wilma.pdf, Updated September 28, 2006.

Pasch et al., 2006b, Pasch, R.J. et al., National Hurricane Center, "Preliminary Report Hurricane Floyd 7-17 November 1999," Website, http://www.nhc.noaa.gov/1999floyd.html, November 18, 1999.

Peck et al., 1974, Peck, Ralph B., Walter E. Hanson, and Thomas H. Thornburn, "Foundation Engineering," John Wiley & Sons, 1974.

Provost and Lyard, 2003, Le Provost, Christian, and Florent Lyard, "The impact of ocean bottom morphology on the modelling of the long gravity waves, from tides and tsunami to climate," Charting the Secret World of the Ocean Floor, The GEBCO Project 1903-2003.

Rappaport, 2005, Rappaport, E., National Hurricane Center, "Preliminary Report Hurricane Andrew 16-28 August 1992," Website, http://www.nhc.noaa.gov/1992andrew.html, Updated December 10, 1993, Addendum 7, February 2005.

Resio and Westerink, 2008, Resio, D.T. and J.J. Westerink, "Modeling the Physics of Storm Surge, Physics Today," Volume 61, Issue 9, pp. 33-38, 2008.

Rodriguez and Paull, 2000, Rodriguez N.M. and C.K. Paull, "14C Dating of Sediment of the Uppermost Cape Fear Slide Plain: Constraints of the Timing of this Massive Submarine Landslide," Proceedings of the Ocean Drilling Program, Scientific Results, Vol. 164.

SFWMD, 2007, South Florida Water Management District (SFWMD), "2007 Martin/St. Lucie County 10-ft DEM in NAVD 1988," Release Version 1, SFWMD Website, http://my.sfwmd.gov/gisapps/sfwmdxwebdc/dataview.asp?, Accessed June 2013.

Southern Resource Mapping, Inc., 2013, Southern Resource Mapping, Inc., "St. Lucie Nuclear Plant Topographic Survey," July 2013.

SSHAC, 1997, Senior Seismic Hazard Analysis Committee (SSHAC), "Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts: Appendices (NUREG/CR-6372, Volume 2)," Available Online: http://www.nrc.gov/reading-rm/doc-collections/nuregs/contract/cr6372/vol2/index.html, Accessed on February 26, 2014.

Stewart, 2005, Stewart, S.R., National Hurricane Center, "Tropical Cyclone Report Hurricane Ivan 2-24 September 2004," Website, http://www.nhc.noaa.gov/2004ivan.shtml?, Updated May 27, 2005.

Strauss and Tomlinson, 2009, Strauss, D. and R. Tomlinson, "Modeling transitions between barred beach states on a straight coast," Proceedings of Coastal Dynamics 2009: Impacts of Human Activities on Dynamic Coastal Processes, Singapore: World Scientific Publishing, pp. 1-11, 2009.

Tappin, 2010, Tappin, D.R., "Mass Transport Events and their Tsunami Hazard," Advances in Natural and Technological Hazards Research, Vol. 28, pp. 667–684, 2010.

Twichell et al., 2009, Twichell, D.C., J.D. Chaytor, U.S. ten Brink, and B. Buczkowski, "Morphology of late Quaternary submarine landslides along the U.S. Atlantic continental margin," Marine Geology, Volume 264, pp. 4-15, 2009.

USACE, 1984a, U.S. Army Corps of Engineers (USACE), "HMR-52 Probable Maximum Storm (Eastern United States) Generalized Computer Program User's Manual," CPD-46, March 1984, Revised April 1987.

USACE, 1984b, U.S. Army Corps of Engineers (USACE), "Shore Protection Manual, Volumes 1 and 2," 1984.

USACE, 2011, U.S. Army Corps of Engineers (USACE), "Coastal Engineering Manual - Parts I-VI," EM 1110-2-1100, Change 3, September 2011.

USACE, 2012, U.S. Army Corps of Engineers (USACE), "Statistical Analysis and Storm Sampling Approach for Lakes Michigan and St. Clair," ERDC/CHL TR-12-19, September 2012.

USACE, 2014, U.S. Army Corps of Engineers (USACE), "CRREL's Ice Engineering Web Site," Available at: http://icejams.crrel.usace.army.mil/, Accessed October 2014.

USDC, 1953, U.S. Department of Commerce (USDC), Coast and Geodetic Survey, "Density of Sea Water at Tide Stations – Atlantic Coast – North and South America," Special Publication No. 279 (Formerly No. DW-1), Revised 1953 Edition, U.S. Government Printing Office, Washington D.C., 1953.

USDC, 1971, U.S. Department of Commerce (USDC), Coast and Geodetic Survey, "Manual of Harmonic Analysis and Prediction of Tides," Special Publication No. 98, Revised 1940 Edition, October 1971 Reprint.

USGS, 2008, U.S. Geological Survey (USGS), Atlantic and Gulf of Mexico Tsunami Hazard Assessment Group (AGMTHAG), "Evaluation of Tsunami Sources with the Potential to Impact the U.S. Atlantic and Gulf Coasts - A Report to the Nuclear Regulatory Commission: U.S. Geological Survey Administrative Report."

USGS, 2009, U.S. Geological Survey (USGS), "1/3-Arc Second National Elevation Dataset (NED)," Published 2009, NED Bounding Coordinates 80.2930° to 80.6820° West to 25.2740° to 25.7890° North, USGS National View Website, http://viewer.nationalmap.gov/viewer, Accessed August 2012.

USGS, 2013, U.S. Geological Survey (USGS), "Forecasting the Impact of Storm Waves and Sea-Level Rise on Midway Atoll and Laysan Island within the Papahānaumokuākea Marine National Monument—

A Comparison of Passive Versus Dynamic Inundation Models," Open-File Report 2013-1069, Available Online: http://pubs.usgs.gov/of/2013/1069/of2013-1069.pdf, Accessed on January 16, 2014.

Vatvani et al., 2012, Vatvani, D., N.C. Zweers, M. van Ormondt, A.J. Smale, H. de Vries, and V.K. Makin, "Storm Surge and Wave Simulations in the Gulf of Mexico Using a Consistent Drag Relation for Atmospheric and Storm Surge Models," Natural Hazards and Earth System Sciences, 12, pp. 2399-2410, 2012.

Vickery et al., 2009, Vickery, Peter J., Forrest J. Masters, Mark D. Powell, and Dhiraj Wadhera, "Hurricane Hazard Modeling: The Past, Present, and Future," Journal of Wind Engineering and Industrial Aerodynamics, doi:10.1016/j.jweia.2009.05.005.

Walton, 2007, Walton, Todd L., "Projected sea level rise in Florida," Ocean Engineering, Volume 34, pp. 1832 – 1840, 2007.

Wang and Manausa, 2005a, Wang R, and M. Manausa, "Hurricane Frances Characteristics and Storm Tide Evaluation," Florida Department of Environmental Protection, Bureau of Beaches and Coastal Systems, 2005.

Wang and Manausa, 2005b, Wang R, and M. Manausa, "Hurricane Jeanne Characteristics and Storm Tide Evaluation," Florida Department of Environmental Protection, Bureau of Beaches and Coastal Systems, 2005

Ward and Day, 2001, Ward, S.N. and S. Day, "Cumbre Vieja Volcano – Potential collapse and tsunami at La Palma, Canary Islands," Geophysical Research Letters, Volume 28, No. 17, pp. 3397-3400, 2001.

Willoughby et al., 1989, Willoughby, H.E. et al., "Notes and Correspondence, A record Minimum Sea Level Pressure Observed in Hurricane Gilbert," Monthly Weather Review, Volume 117, pp. 2824–2825, Website, http://journals.ametsoc.org/doi/abs/10.1175/1520-0493%281989%29117%3C2824%3AARMSLP%3E2.0.CO%3B2, February 26, 1989 and June 3, 1989.

Wilmott et al., 2012, Wilmott, C., S. Robeson, and K. Matsuura, "A refined index of model performance," International Journal of Climatology 32, pp. 2088–2094, 2012.

Winterkorn and Fang, 1975, Hans F. Winterkorn and Hsai-Yang Fang, "Foundation Engineering Handbook," Van Nostrand Reinhold Company, 1975.

WMO, 1998, World Meteorological Organization (WMO), "Guide to Wave Analysis and Forecasting," WMO No. 702, 1998.