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2 SITE CHARACTERISTICS

2.4 Hydrologic Engineering

To ensure that a nuclear power plant or plants can be designed, constructed, and safely operated on the applicant's (PSEG) proposed site (i.e., PSEG Site) and in compliance with U.S. Nuclear Regulatory Commission (NRC) regulations, the staff evaluated the hydrologic characteristics of the site and surrounding vicinity that may affect the safety of a potential nuclear power plant at the site. These site characteristics describe the potential for flooding due to precipitation, riverine processes (runoff, dam breach discharge, channel blockage or diversion), coastal effects (storm surges and tsunamis), and combined events (e.g., from coincident wind waves). In addition, the staff reviewed the maximum elevation of surface water during floods and combined events, associated static and dynamic characteristics, minimum water-surface elevation during low-water events, maximum elevation of groundwater, and the characteristic ability of the site to attenuate a postulated accidental release of radiological material into surface water and groundwater. The surface water hydrologic site characteristics determine the design-basis flood for the proposed PSEG Site, and provide the basis for determining whether flood protection will be required. The groundwater hydrologic site characteristics determine the design-basis groundwater loadings and provide the basis for radiological dose analysis for a potential receptor from the postulated accidental release of radioactive liquid effluents in surface and ground waters.

The staff prepared Sections 2.4.1 through 2.4.14 herein in accordance with the review procedures described in NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," Sections 2.4.1 through 2.4.14, using information presented in the applicant's Site Safety Analysis Report (SSAR), Revision 3, Section 2.4, "Hydrologic Engineering," which references responses to staff requests for additional information (RAIs), and generally available reference materials (e.g., those cited in applicable sections of NUREG-0800).

2.4.1 Hydrologic Description

The applicant provided information on the radioactive liquid effluents that would be generated as a normal byproduct of nuclear power operations. These radioactive materials will be collected, processed, stored, and discharged in a controlled manner to the local environment. The proposed facility will have the ability to handle these radiological effluents in a manner that minimizes radioactive releases to the environment and maintains exposure to the public during normal plant operation, anticipated operational occurrences (AOO), and maintenance at levels that are as low as is reasonably achievable (ALARA).

2.4.1.1 *Introduction*

The PSEG Site is located on a tidally influenced reach of the Delaware River 83.7 km (52 mi) north of the mouth of Delaware Bay (Figure 2.4.1-1). SSAR Section 2.4.1 provides an overview of the hydrologic characteristics and phenomena that have the potential to affect the plant design basis of a reactor technology to be determined within the plant parameter envelope (PPE) at the combined license (COL) application stage. Designs under consideration within the PPE are discussed in Section 2.4.1.4.1 of this report.

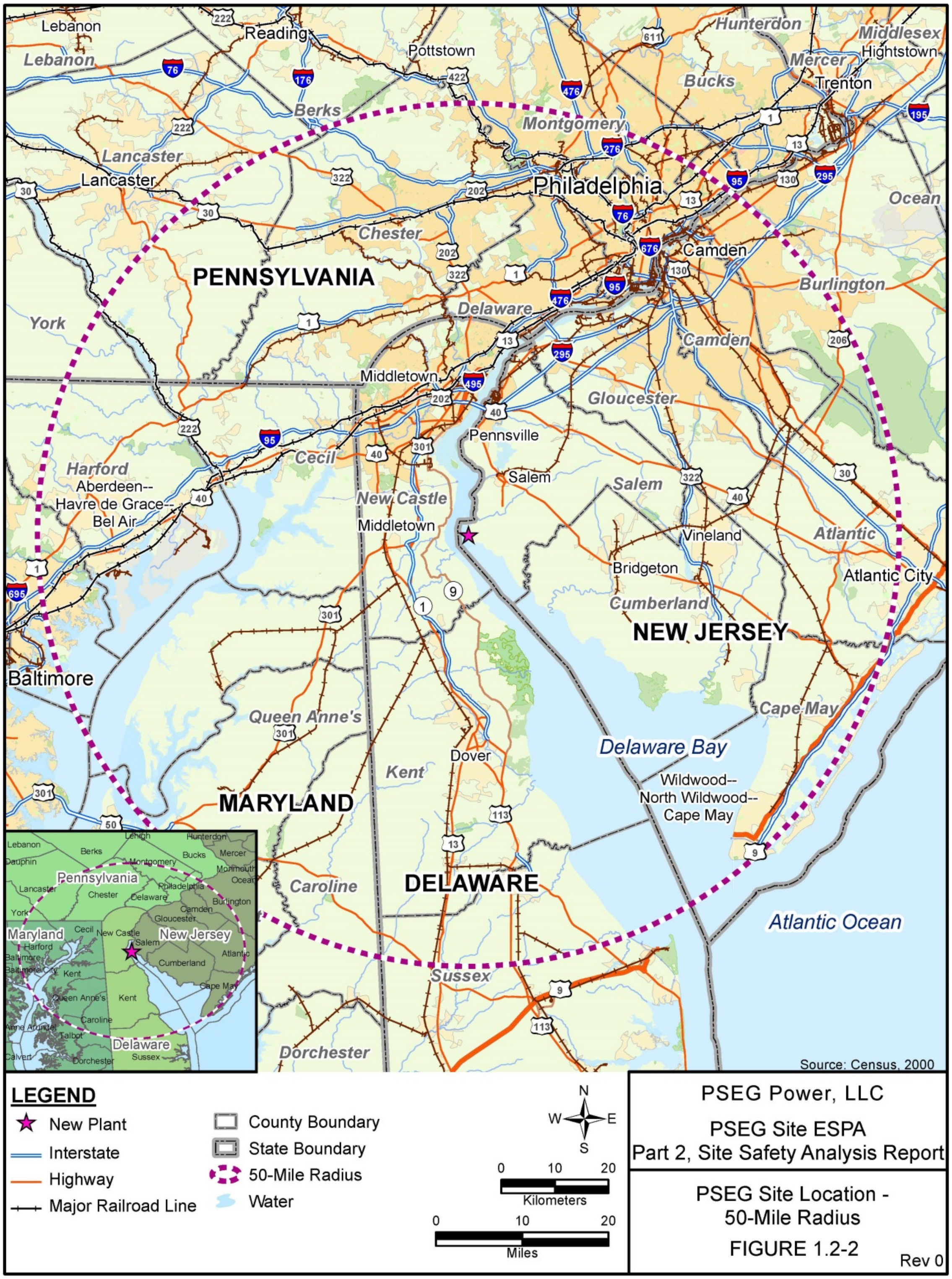

Figure 2.4.1-1 PSEG Site Region (from SSAR Revision 3, Figure 1.2-2)

The hydrologic description of the PSEG Site includes the interface of the plant with the hydrosphere, hydrological causal mechanisms, surface and groundwater uses, hydrologic data, and alternate conceptual models. The staff review discusses the following specific areas: (1) interface of the plant with the hydrosphere, including descriptions of site location, major hydrologic features in the site vicinity, surface water and groundwater-related characteristics, and the proposed water supply to the plant; (2) hydrological causal mechanisms that may require special plant design bases or operating limitations with regard to floods and water supply requirements; (3) current and likely future surface and groundwater uses by the plant and water users in the vicinity of the site that may impact safety of the plant; (4) available spatial and temporal data relevant for the site review; (5) alternate conceptual models of the hydrology of the site that reasonably bound hydrological conditions at the site; (6) potential effects of seismic and nonseismic data on the postulated design bases and how they relate to the hydrology in the vicinity of the site and the site region; and (7) any additional information requirements prescribed within the "Contents of Application" sections of the applicable subparts to Title 10 of the *Code of Federal Regulations* (10 CFR) Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants."

2.4.1.2 *Summary of Application*

In SSAR Section 2.4.1, the applicant described the site and all safety-related elevations, structures, and systems from the standpoint of hydrologic considerations and provided a discussion of proposed changes to natural drainage features. Since a technology has not been selected proposed changes to existing grade, a site grading plan and a drainage design will be evaluated at the COL stage.

2.4.1.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the hydrologic description, and the associated acceptance criteria, are specified in NUREG–0800, Section 2.4.1.

The applicable regulatory requirements for identifying the site location and describing the site hydrosphere are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), "Contents of applications," as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, "Reactor Site Criteria," as it relates to identifying and evaluating hydrologic features of the site. The requirements to consider physical site characteristics in site evaluations are specified in 10 CFR 100.20(c).

The staff also used the appropriate sections of the following regulatory guides (RGs) for the acceptance criteria identified in NUREG–0800, Section 2.4.1:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed.

- RG 1.29, "Seismic Design Classification," as it relates to those structures, systems, and components (SSCs) intended to protect against the effects of flooding.

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.

- RG 1.102, “Flood Protection for Nuclear Power Plants,” as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.1.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.1 and confirmed that the information in the application adequately and acceptably addresses the required information and components related to the site’s hydrologic description. On the basis of its review, the staff confirmed that the information contained in the application addresses the required information related to this section under Docket 52-043. The staff’s technical evaluation of the information including the applicant’s responses to RAIs will be documented in the staff’s Final Safety Evaluation Report (FSER) for the ESP.

The staff conducted a site audit on February 15 - 16, 2011, in accordance with the guidance provided in NUREG-0800, Section 2.4.1 to review information provided by the applicant. The staff used information from this site visit, United States Geological Survey (USGS) topographic maps, topographic maps of the site provided by the applicant, available studies and references, and independent reviews to verify the hydrologic description provided by the applicant. The following sections describe the staff’s evaluation of the technical information submitted by the applicant.

2.4.1.4.1 Site and Facilities

Information Submitted by the Applicant

The applicant’s proposed plant location is north of the Hope Creek Generating Station (HCGS) lying mostly within the current property boundary. The applicant developed an agreement in principle with the U.S. Army Corps of Engineers (USACE) to acquire an additional 85 acres immediately to the north of the HCGS for the proposed facility. Although a specific reactor technology has not been selected for construction at the PSEG Site, designs under consideration inclusive of the PPE are as follows:

- Single Unit U.S. Evolutionary Power Reactor (U.S. EPR)

- Single Unit Advanced Boiling Water Reactor (ABWR)

- Single Unit U.S. Advanced Pressurized-Water Reactor (US-APWR)

- Dual Unit Advanced Passive 1000 (AP1000)

The applicant described the site hydrology and the principal plant structures with the constraints of the PPE for the associated design elevations, and presented maps showing drainage patterns for existing conditions. The Delaware River will be used for circulating water system makeup water and plant turbine cooling systems. The minimum surface water elevation for the ultimate heat sink (UHS) makeup water intake is -4.85 meters (m) (-15.9 feet (ft)) North American Vertical Datum (NAVD) 1988 (SSAR Section 2.4.11).

The design basis flood (DBF) level is 9.78 m (32.1 feet (ft)) NAVD88 as described in SSAR Section 2.4.5, while the proposed site grade is 11.25 m (36.9 ft) NAVD88. The intake structure will be designed at the COL stage with flood protection features to withstand the DBF and associated effects as required by the selected technology. At the COL stage, a site grading plan and drainage system will be designed to route runoff from probable maximum precipitation (PMP) into swales and pipes draining toward the Delaware River. The staff is tracking the applicant's evaluation of PMP at the COL stage via **COL Action Item 2.4-1** and flood protection at the COL stage via **COL Action Item 2.4-2** (See Sections 2.4.2.4.3 and 2.4.10.4, respectively, of this report).

The Staff's Technical Evaluation

Initially, the staff determined that reference elevations in the SSAR, Revision 0, referred to multiple elevation datum and temporal information**.** In RAI 25, Question 02.04.01-2, the staff requested that the applicant provide consistent elevation information and datum conversion procedures, temporal information and gaging station identification. In a June 23, 2011, response to RAI 25, Question 02.04.01-2, the applicant committed to modify and correct text and tables in SSAR Section 2.4. The staff confirmed that the corrections were incorporated into Revision 1 of the ESP application (May 21, 2012). Elevations reported in SSAR Section 2.4, Revision 1, were converted into NAVD88 datum consistently. Some components of hydrologic events such as storm surge and wave height are customarily expressed in feet, which need not be referenced to a geographic datum. The staff considers RAI 25, Question 02.04.01-2 resolved.

Based on a review of the material presented by the applicant in SSAR Section 2.4.1, the staff's observations of the PSEG Site during the February 2011 site audit, and the applicant's response to the RAIs discussed above, the staff finds that the applicant has adequately considered the hydrologic characteristics of the ESP site within this section.

2.4.1.4.2 Hydrosphere

This section describes the hydrology in the vicinity of the proposed site, including rivers and streams, lakes and reservoirs, coastal regions, and surface water and groundwater uses.

Information Submitted by the Applicant

The applicant described the local and regional hydrology surrounding the PSEG Site. As stated in SSAR Section 2.4.1.1, the applicant's descriptions of hydrologic characteristics were taken from publicly available maps and data published by the USGS, National Oceanic and Atmospheric Administration (NOAA), Natural Resources Conservation Service (NRCS), USACE, and/or appropriate State agencies and the Delaware River Basin Commission (DRBC).

The proposed PSEG Site is located on Artificial Island on the east bank of the Delaware River in Lower Alloways Creek Township, Salem County, New Jersey (NJ). The Delaware River has a drainage area of approximately 35,224 square kilometers (km^2) (13,600 square miles (mi^2)) and is the largest undammed river east of the Mississippi River. The river basin includes portions of Delaware, Maryland, New Jersey, New York, and Pennsylvania and crosses five physiographic provinces: the Coastal Plain, Piedmont, New England, Valley and Ridge, and the Appalachian Plateau. The total drainage area upstream of the PSEG Site is 29,785 km^2 (11,500 mi^2).

The site is located 52 river miles (RM) upstream (i.e., at RM 52), from the mouth of Delaware Bay. The proposed finished new plant grade is 11.25 m (36.9 ft) NAVD88, which is 1.47 m

(4.8 ft) above the DBF (9.78 m (32.1 ft) NAVD88) based on storm surge as described in SSAR Section 2.4.5. As discussed in Section 2.4.2 herein, tidal action and storm surge is the primary influence on the DBF. Under normal conditions, the tidal flow ranges from 11,327 cubic meters (m^3) (400,000 cubic feet (ft^3)) per second to 13,366 m^3 (472,000 ft^3) per second while freshwater flow at the PSEG Site is approximately 425 m^3 (15,000 ft^3) per second (Reference 2.4.1-1).

Average annual precipitation in the Delaware River basin ranges from 127 centimeters (cm) (50 inches (in.)) in the upper basin to 107 cm (42 in.) in the lower basins near the PSEG Site and is generally evenly distributed over the basin throughout the year (Reference 2.4.1-2).

As the Delaware River is the primary source of water for operation for the PSEG plant, the applicant stated that the safety-related intake structure for the selected reactor technology will be designed to operate during the lowest water conditions, which is assumed coincident with a 20-year low flow in the Delaware River at Trenton and 90 percent exceedance low tide, which would result in an extreme and temporary low water level of -4.85 m (-15.9 ft) NAVD88.

The Staff's Technical Evaluation

The staff reviewed the completeness of the hydrologic data and watershed characteristics, and made several spot checks to confirm the accuracy of specific data, such as basin physiography, precipitation, tidal surges, peak flood flows and historical water surface elevations in the Delaware River. As noted in studies of the Delaware River (Reference 2.4.1-1), tidal flow is approximately 30 times greater than fresh water flow at the PSEG Site under average conditions increasing to approximately 290 times greater near the Delaware Bay entrance (Reference 2.4.1-3). During the site visit and audit in 2011, the staff identified and confirmed various site characteristics that were considered in flood analyses at the site and finds the applicant's evaluation adequate.

Based on a review of the material presented by the applicant in SSAR Section 2.4.1, the staff's observations of the PSEG Site during the February 15-16, 2011, site visit and audit, and the staff's independent review of published data and reports, the staff finds that the applicant has adequately considered the hydrosphere near the PSEG Site.

2.4.1.4.3 Hydrologic Casual Mechanisms

Information Submitted by the Applicant

The Delaware River is the only surface water body of any significance that could affect the site. The Delaware River has a drainage area of 35,224 km² (13,600 mi²) and is undammed along the entire course of its main stem. The transition between the head of the Delaware Bay and the mouth of the river occurs at RM 48, 6.48 km (4 mi) downstream from the PSEG Site. At the PSEG Site, the Delaware River is subject to tidal influence from the mouth of the Delaware River to the upstream limit of the estuary, which is defined by RM 134 in Trenton, NJ. Historical records indicate that the highest flood events recorded near the mouth of the Delaware River and within Delaware Bay are caused by storm surge associated with hurricanes. Wave run up due to tsunamis is far less likely to affect the PSEG Site as there have been few recorded Atlantic coast incidents of significant run up due to tsunamis.

Tides enter Delaware Bay from the Atlantic Ocean and propagate upstream. The tide of the Delaware Estuary is semidiurnal in character. There are two high waters and two low waters in a tidal day, with comparatively little diurnal inequality. The Reedy Point station (RM 58.6) is the tidal gauge station nearest the PSEG Site. The mean tide range at this location is 1.63 m

(5.34 ft), indicating a significant influence of tide on river flow. NOAA tidal gauge stations are used to calibrate hydraulic models for the tidally influenced sections of the Delaware River and Delaware Bay.

There are 24 reservoirs on Delaware River tributaries in the Delaware River Basin (Figure 2.4.1-2 of this report). Of these, nine reservoirs are dedicated for water supply, two generate hydropower, three are dedicated for flood loss reduction, and one is solely for flow augmentation. The remaining nine reservoirs are multipurpose, providing water for a combination of water supply, flow augmentation, and flood loss reduction. Dedicated water supply reservoirs fill during the winter and spring months to ensure water supply during dry months. Multipurpose reservoirs and those dedicated to flood reduction maintain year-round flood storage voids to mitigate flooding. Flow management of the Delaware River is accomplished through coordinated releases from major reservoirs on its tributaries as overseen by the DRBC.

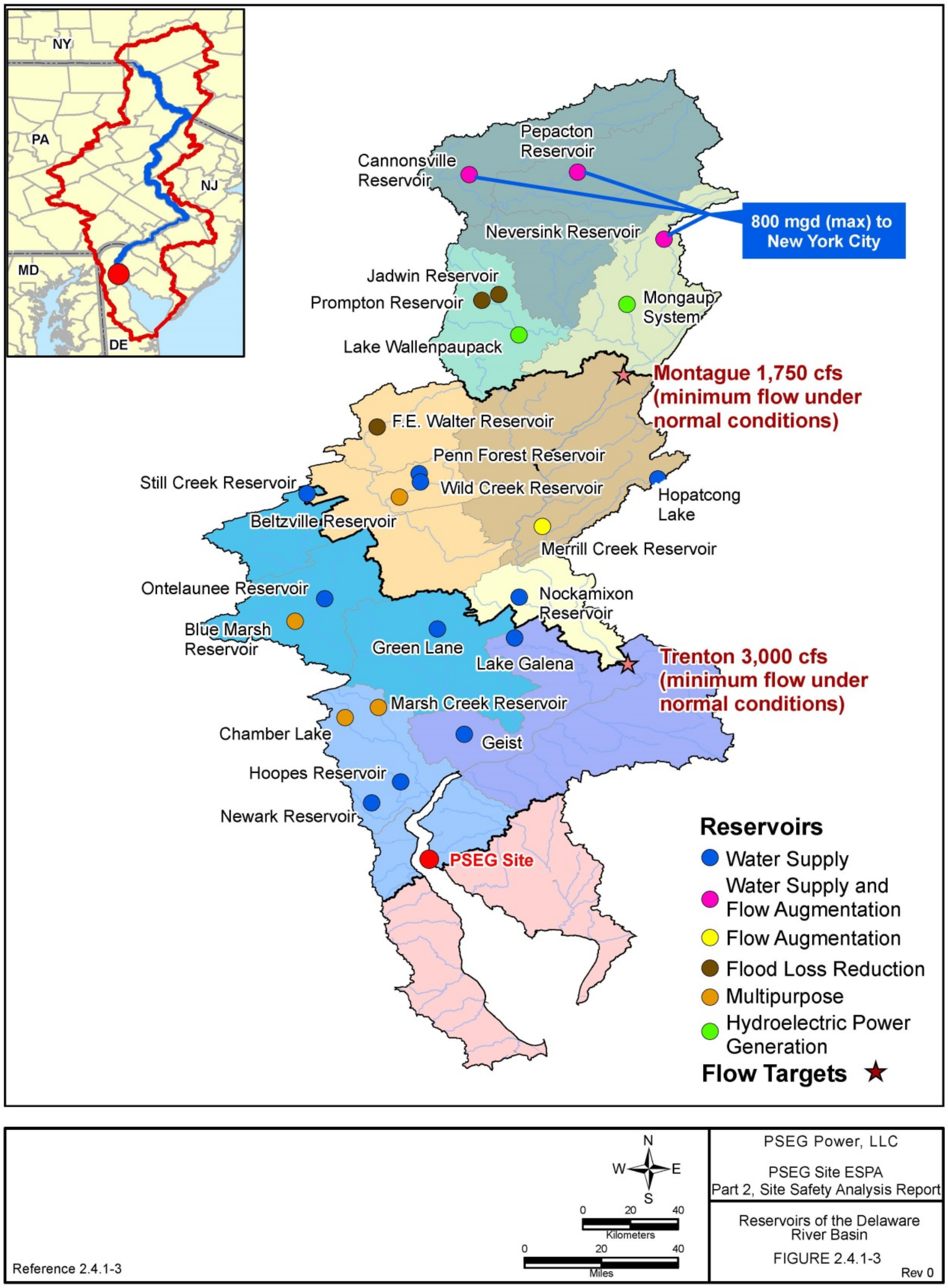

Figure 2.4.1-2 Reservoirs in the Delaware River Basin (from SSAR Revision 3, Figure 2.4.1-3)

The Staff's Technical Evaluation

The staff reviewed the information provided by the applicant and performed an independent review of the applicant's information. The staff also visited the site and verified the disposition and elevation of the Delaware River, surrounding creeks, and hydrologic features. The staff supplemented this information with other publicly available sources of data. Based on this review, the staff concludes that the information and data provided are adequate.

Specific discussions of the effects of various hydrologic phenomena such as storm surge, tsunamis, floods, dam failures, ice effects, and groundwater levels are included in Sections 2.4.2 through 2.4.14 herein.

2.4.1.4.4 Surface and Groundwater Uses

Information Submitted by the Applicant

The Delaware River is a primary source of water for industry and municipalities, a receiving body for effluent, a resource for power generation, and a location for recreational activities. The DRBC authorizes Delaware River surface-water withdrawals for industrial and public water supply purposes in Delaware, Pennsylvania and New Jersey. The majority of surface-water users are located upstream of the PSEG Site. The primary surface-water users of the Delaware River are industrial, power, commercial, and water supply. Instream use of the Delaware River includes port traffic, barge traffic, fishing, boating, and other recreational activities.

The applicant addressed groundwater in SSAR Section 2.4.12 and summarized groundwater users in the SSAR. The staff's reviews of the information submitted by the applicant are located in Section 2.4.12 herein.

The applicant described the current and past surface-water use of the Delaware River. This information about water use was presented and summarized in the SSAR.

The Staff's Technical Evaluation

The staff reviewed the information provided by the applicant and performed an independent review of the applicant's information. The staff also visited the site and verified the location of important water users. The staff supplemented this information with other publicly available sources of data. Based on this review, the staff concludes that the information and data provided are adequate.

2.4.1.5 *Post Early Site Permit Activities*

There are no post ESP activities related to this section.

2.4.1.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the PPE, and that there is no outstanding information required to be addressed in the SSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description to

allow the staff to evaluate, as documented in Section 2.4.1 of this report, whether the applicant has met the relevant requirements of 10 CFR 52.17(a)(1) and 10 CFR Part 100 with respect to determining the acceptability of the site. The staff finds that the applicant has provided sufficient information to satisfy the requirements of 10 CFR Part 52 and 10 CFR Part 100.

2.4.2 Floods

2.4.2.1 *Introduction*

SSAR Section 2.4.2 discusses historical flooding at the proposed site and in the region of the site. The information summarizes and identifies the individual types of flood-producing phenomena, and combinations of flood-producing phenomena, considered in establishing the flood design basis for safety-related plant features.

Section 2.4.2 herein provides a review of the specific areas as follows: (1) local flooding on the site and drainage design; (2) stream flooding; (3) surges; (4) seiches; (5) tsunami; (6) dam failures; (7) flooding caused by landslides; (8) effects of ice formation on water bodies; (9) combined event criteria; (10) other site-related evaluation criteria; and (11) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.2.2 *Summary of Application*

In SSAR Section 2.4.2, the applicant addresses the information related to site-specific and regional flood causal mechanisms.

2.4.2.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the identification of floods and flood design considerations, and the associated acceptance criteria, are specified in NUREG–0800, Section 2.4.2.

The applicable regulatory requirements for identifying probable maximum flooding on streams and rivers are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

The staff also used the appropriate sections of the following regulatory guides for the acceptance criteria identified in NUREG–0800, Section 2.4.1:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed.

- RG 1.29, “Seismic Design Classification,” as it relates to those SSCs intended to protect against the effects of flooding.

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.

- RG 1.102, “Flood Protection for Nuclear Power Plants,” as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.2.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.2 and confirmed that the information in the application addresses the required information related to site floods. The staff’s technical review of this application included an independent review of the applicant’s information in the SSAR and in the applicant’s responses to staff RAIs. The staff supplemented this information with other publicly available sources of data. The review areas included:

- Stream flooding

- Surges and seiches

- Tsunamis

- Dam failures

- Effects of ice formation in water bodies

- Channel diversions

- Combined events criteria

- Consideration of other site-related evaluation criteria

In addition to the systematic review of information provided by the applicant, the staff visited the site and verified the location and elevation of important streams and hydrologic features. The staff reviewed the available information in SSAR, Revision 0, and concluded that the SSAR did not provide sufficient detail describing the techniques and methodology used for surface water modeling and resulting flood levels at the site. Therefore, in RAI 25, Question 02.04.01-1, the staff requested that the applicant provide detailed descriptions of the methods used, the simulation input files, and a description of the simulation scenarios so the staff could verify that the parameters were reasonable given the resulting runoff flow paths and flood levels. The staff’s review of the information contained in the SSAR is discussed below.

2.4.2.4.1 Flood History

<u>*Information Submitted by the Applicant*</u>

The applicant provided extensive analyses and computations to determine maximum flow rates and flood levels. Detailed information was provided regarding input parameters and flood computations associated with the following:

- Stream and river flooding
- Dam failures (on Delaware River Tributaries)
- Surge and seiche flooding
- Tsunamis
- Effects of ice formation in water bodies
- Channel diversions
- Combined events criteria

Flooding due to underwater landslides was evaluated within the tsunami scenarios by the applicant. Due to a lack of a technology-specific grading plan and site drainage system design, less detailed information was provided for local flooding on the site and drainage based on the PMP. Once a final technology is selected with an associated grading plan and drainage design, PMP peak flows and water levels will be re-analyzed to establish the maximum water surface elevation near plant safety-related SSCs.

The Staff's Technical Evaluation

In a June 23, 2011, response to RAI 25, Questions 02.04.01-1 and 02.04.01-2 (See Section 2.4.1.4.1 of this report), the applicant provided detailed maps, surface water input and output files, and a user information guide clearly describing the information and methods used for surface water modeling including river flooding, dam failures on tributaries to the Delaware River, tidal induced flooding and low water, and channel diversions with combined event scenarios. The staff finds the information submitted adequate for the staff's evaluation and considers RAI 25, Questions 02.04.01-1 and 02.04.01-2 resolved.

The staff reviewed the flood history information provided by the applicant in SSAR Section 2.4.2 and finds that the information provided is sufficient to establish the history of flooding at and near the PSEG Site.

2.4.2.4.2 Flood Design Considerations

Information Submitted by the Applicant

The applicant noted that the highest recorded flood events near the mouth of the Delaware River and in the vicinity of the site are produced by storm surge with southeast to northwest moving hurricanes producing the more severe surge levels.

The Trenton, NJ gauge is the last downstream gauge at which discharge values for the Delaware River are determined based solely on freshwater discharge. In selecting the initial base flow for flood event scenarios, the applicant used discharge measurements from the Trenton gauge for a June 2006 flood event. The applicant selected this discharge measurement of greater than 6,372 m^3 (225,000 ft^3) per second over two earlier (1905 and 1955), discharge measurements of greater than 9,314 m^3 (329,000 ft^3) per second, for the following reasons: (a) the 2006 event had more recent and accurate records; (b) a substantial reservoir capacity was added, including flood control reservoirs, since the earlier events; (c) it best represented the current configuration of the basin; and (d) the relatively uniform rainfall

totals over the basin during the 2006 event were conducive to subbasin calibration. Major river flood events have little impact on gauge levels at the PSEG Site due to the wide and open marine connection of the Delaware River adjacent to the site.

Tsunamis (SSAR Section 2.4.6) along the Atlantic Coast are rare in the historical record; the most recent tsunami was recorded in 1929 at Atlantic City with amplitude of 0.67 m (2.2 ft). In SSAR Section 2.4.7, the applicant reviewed the USACE Cold Regions Research and Engineering Laboratory (CRREL) Ice Jam Database and noted no ice jams causing flooding downstream of the PSEG Site, and evaluated the ice jam flooding potential. Combinatory dam break flooding potential is evaluated by the applicant in SSAR Section 2.4.4. The flooding scenarios investigated for the site include the following:

- Flooding due to PMP on the site (SSAR Section 2.4.2)
- Probable maximum flood (PMF) on rivers and streams (SSAR Section 2.4.3)
- Potential dam failures (SSAR Section 2.4.4)
- Maximum surge and seiche flooding (SSAR Section 2.4.5)
- Probable maximum tsunami (PMT) (SSAR Section 2.4.6)
- Ice effect flooding (SSAR Section 2.4.7)
- Channel diversions (SSAR Section 2.4.9)

The applicant's evaluation of the above flooding scenarios confirmed that the DBF for the new plant of 9.78 m (32.1 ft) NAVD88 is associated with storm surge. As applicable to the design at the COL stage, the applicant will design a safety-related intake structure to withstand the DBF and associated effects (See Section 2.4.10 of this report). All safety-related SSCs at a site grade elevation or higher will have adequate safety margins relative to the DBF for the given PPE.

The Staff's Technical Evaluation

The applicant's assertion that the strong tidal nature of the Delaware River adjacent to the site precludes significant impacts to safety-related SSCs from rainfall/runoff scenarios in the river basin is reasonable given the physiography of the area and the wide and open marine connection of the Delaware River at the PSEG Site. The applicant's choice of a flood flow magnitude slightly less than two earlier, larger events in the interest of a more complete and accurate record results in an appropriately conservative representation of the physical system for surface water modeling. The staff reviewed the historical record for flooding and the CRREL Ice Jam Database and verified that no ice jams downstream of the PSEG Site have caused flooding. The staff finds that the applicant's conclusion of the DBF due to storm surge as the bounding event is consistent with the historical record and physiography of the Delaware River basin. The staff finds the applicant's evaluation of flood design considerations adequate.

Based on a review of the applicant's information contained in the SSAR, the staff finds that the applicant appropriately considered flood-causing phenomena and their combinations that are relevant for the PSEG Site.

2.4.2.4.3 Local Intense Precipitation

Information Submitted by the Applicant

To determine the potential effects of flooding, it is very important to select an appropriately conservative rainfall event on which to base the hydrologic designs. Further, the staff considers that the selection of a design flood event should not be based on the extrapolation of limited historical flood data, due to the unknown level of accuracy associated with such an extrapolation. The applicant utilized the PMP event, computed by deterministic methods (rather than statistical methods) and based on site-specific hydrometeorological characteristics. The PMP has been defined as the most severe reasonably possible rainfall event that could occur as a result of a combination of the most severe meteorological conditions occurring over a watershed. No recurrence interval is normally assigned to the PMP; however, the staff has concluded that the probability of such an event being equaled or exceeded during the plant lifetime is very low. Accordingly, the PMP is considered by the staff to provide an acceptable design basis. The staff considers that use of the PMP meets requirements of 10 CFR 52.17, "Contents of Applications; Technical Information in Final Safety Analysis Report," and provides sufficient margins to account for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

Prior to determining the runoff, the flooding analysis requires the determination of PMP amounts for the specific site location. Techniques for determining the PMP have been developed for the United States by Federal agencies in the form of hydrometeorological reports for specific regions. These techniques are widely used and provide straightforward procedures with minimal variability.

For the PSEG Site, PMP values and rainfall distributions were estimated by the applicant using reports prepared by NOAA, including Hydrometeorological Report No. 51 (HMR-51) (Reference 2.4.2-1) and Hydrometeorological Report No. 52 (HMR-52) (Reference 2.4.2-2). Using these reports, a 1-hour PMP of 46.73 cm (18.4 in.) was used by the applicant as a basis for estimating the PMF for each of the subbasins affected. (The PMF is a hypothetical flood that is considered to be the most severe reasonably possible flood, based on comprehensive hydrometeorological application of the PMP and other hydrologic factors favorable for peak runoff.) The staff reviewed HMR-51, HMR-52, and the procedures for estimating PMP values for several different durations, and concluded that the PMP amounts are acceptable for the subbasin drainage areas.

Due to the lack of a specific technology, a site grading plan and storm water management system necessary to establish the maximum site water surface elevation due to the PMP have not been determined. The applicant has stated that the local PMP event will not affect new plant safety features; however, a detailed PMP analysis to determine the maximum site water level cannot be performed until a specific technology is selected.

The Staff's Technical Evaluation

The staff reviewed NOAA HMR-51 (Reference 2.4.2-1) and HMR-52 (Reference 2.4.2-2) to verify the applicant's determination of the PSEG Site PMP event. The staff finds that the applicant's deterministic method of defining the PMP event for the site reasonable and the associated analysis adequate; however, the site grading plan and storm water management system will be specific to the reactor technology to be selected by the COL applicant. Accordingly, the staff has identified **COL Action Item 2.4-1** to address this item.

Based on a review of the applicant's information in the SSAR, the staff finds that the applicant has appropriately considered flood-causing phenomena related to local intense precipitation for the PSEG Site.

2.4.2.4.4 Infiltration Losses

Information Submitted by the Applicant

The applicant used the NRCS curve number method (Reference 2.4.2-3) commonly used to estimate runoff. This method incorporates the effects of soil, surface vegetation and land management into a representative value for each of the subbasins. To maximize runoff, initial soil moisture was saturated by using the highest antecedent moisture condition (AMC) III, for the respective curve number prior to the PMP event. The applicant used these subbasin curve numbers as initial conditions to estimate peak discharge resulting from the PMP event. Weighted by subbasin areas, curve numbers were calculated for each of the subbasins to derive initial soil moisture conditions.

The Staff's Technical Evaluation

Using AMC III is an acceptable and reasonable method of initializing soil saturation to limit infiltration losses prior to a PMP event. Curve number values are between 0 and 100 with values near 100 characteristic of impervious basins (i.e., maximum runoff). Weighted by subbasin areas, curve numbers were calculated for each of the subbasins and resulted in a relatively high overall average value of 90.5 to provide conservative estimates of peak discharge relative to the PMP event. The staff finds the applicant's analysis adequate.

The staff agreed that the flood causing phenomena associated with infiltration losses considered by the applicant are appropriate for the PSEG Site. Based on a review of the applicant's information in the SSAR, the staff finds that the applicant has appropriately considered flood-causing phenomena related to infiltration losses for the PSEG Site.

2.4.2.4.5 Time of Concentration

Information Submitted by the Applicant

The time of concentration is the amount of time required for runoff to reach the outlet of a drainage basin from the most remote point in that basin. The peak runoff for a given drainage basin is inversely proportional to the time of concentration. If the time of concentration is assumed to be smaller, the peak discharge will be larger. Times of concentration and/or lag times are typically computed using empirical relationships such as those developed by Federal agencies.

The applicant estimated times of concentration and lag times for the various subbasins using methods recommended by NRCS and presented in the USDA, "Urban Hydrology for Small Watersheds," Technical Release (TR) 55 Manual (Reference 2.4.3-4).

The Staff's Technical Evaluation

These methods are generally accepted in hydrologic engineering practice and are considered by the staff to be appropriate and adequate for estimating times of concentration at the PSEG Site. The staff finds the applicant's analysis adequate.

The staff agreed that the flood causing phenomena associated with time of concentration considered by the applicant are appropriate for the PSEG Site. Based on a review of the applicant's information in the SSAR, the staff finds that the applicant has appropriately considered flood-causing phenomena related to time of concentration for the PSEG Site.

2.4.2.4.6 Rainfall Distributions

Information Submitted by the Applicant

A typical PMP value is derived for periods of about 1 hour. If the time of concentration is less than 1 hour, it is necessary to extrapolate the data presented in the various hydrometeorological reports to shorter time periods. For example, the applicant used distributions recommended in NOAA, Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian, Hydrometeorological Report (HMR)-52 (Reference 2.4.2-2) and determined the 5-minute PMP to be about 15.5 cm (6.1 in.), with a resulting rainfall intensity of 1.85 m (72.8 in.) per hour.

The Staff's Technical Evaluation

Based on a review of the applicant's assumptions, input parameters, and calculations, the staff finds that the computed peak rainfall amounts (and resulting intensities) for various short time periods are reasonable and conservative and, are therefore adequate.

The staff agreed that the flood causing phenomena associated with rainfall distributions considered by the applicant are appropriate for the PSEG Site. Based on a review of the applicant's information in the SSAR, the staff finds that the applicant has appropriately considered flood-causing phenomena related to rainfall distributions for the PSEG Site.

2.4.2.4.7 Computation of Peak Flood Discharges and PMF Water Levels

Information Submitted by the Applicant

Various methods can be used to determine peak PMF flows and water levels, depending on the location of the feature, the drainage area, and other factors. Peak flows and water levels generated by the PMP/PMF within the PSEG Site were determined by the applicant using the Hydrologic Engineering Centers-Hydrologic Modeling System (HEC-HMS) (Reference 2.4.2-5) model developed by the USACE. The software is widely used by many Federal agencies (and others) for various hydrologic analyses and is considered by the staff to be an adequate model.

Due to the lack of a specific technology, a site grading plan and storm water management system necessary to establish the maximum site water surface elevation due to the PMP have not been developed. A detailed PMP analysis to determine the maximum site water level given a site-specific drainage system will be performed by the applicant and reviewed by staff at the COL stage after a reactor technology is selected.

The Staff's Technical Evaluation

Although the staff concluded that the models and procedures outlined above are acceptable, the staff needed additional information and the applicant's calculations to determine the acceptability of the computed peak flood flows and water levels. The applicant addressed the staff's needs in a June 23, 2011, response to RAI 25, Questions 02.04.02-1 and 02.04.02-2 (See Section 2.4.1.4.1 of this report). The staff reviewed digital modeling files, explanations and

calculations submitted by the applicant concerning the flooding caused by the probable maximum precipitation at the site. The approach to the modeling appears to be appropriate but a final evaluation of the PMP impacts to site-specific flooding cannot be made until the detailed modeling is conducted based on an actual site design and storm water management system. The staff finds the applicant's analysis adequate. The staff considers RAI 25, Questions 02.04.02-1 and 02.04.02-2, resolved, noting that PMP impacts will be developed by the applicant at the COL stage after a specific reactor technology is selected. The staff is tracking the applicant's evaluation of PMP and flood protection at the COL stage via **COL Action Item 2.4-1** (See Section 2.4.2.4.3 of this report).

2.4.2.5 *Post Early Site Permit Activities*

The staff will review the applicant's modeling incorporating site-specific grading plans and storm water management system design features to determine site-specific PMP flooding, identified as **COL Action Item 2.4-1.**

2.4.2.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the PPE, and that there is no outstanding information required to be addressed in the SSAR related to this section as related to the application.

As set forth above, the applicant presented and substantiated information to establish the site description. The staff reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description to allow the staff to evaluate, as documented in Section 2.4.2 herein, whether the applicant has met the relevant requirements of 10 CFR 52.17(a)(1) and 10 CFR Part 100 with respect to determining the acceptability of the site. In conclusion, the applicant has provided sufficient information for satisfying the requirements of 10 CFR Part 52 and 10 CFR Part 100. The COL applicant will address **COL Action Item 2.4-1**.

2.4.3 Probable Maximum Flood on Streams and Rivers

2.4.3.1 *Introduction*

SSAR Section 2.4.3 describes the hydrological site characteristics affecting any potential hazard to the plant's safety-related facilities as a result of the effect of the PMF on streams and rivers, and combinations of flood-producing phenomena.

Section 2.4.3 herein provides a review of the following specific areas: (1) design basis for flooding in streams and rivers; (2) design basis for site drainage; (3) consideration of other site-related evaluation criteria; and (4) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.3.2 *Summary of Application*

In SSAR Section 2.4.3, the applicant addresses the information about site-specific PMFs on streams and rivers.

2.4.3.3 *Regulatory Basis*

The relevant requirements of NRC regulations for identifying the PMF on streams and rivers, and the associated acceptance criteria, are specified in NUREG–0800, Section 2.4.3.

The applicable regulatory requirements for identifying the PMF on streams and rivers are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirements to consider physical site characteristics in site evaluations are specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used the appropriate sections of the following regulatory guides for the acceptance criteria identified in NUREG-0800, Section 2.4.3:

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants,” as it relates to providing high assurance that the water sources relied on for the sink will be available where needed.

- RG 1.29, “Seismic Design Classification,” as it relates to those SSCs intended to protect against the effects of flooding.

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.

- RG 1.102, “Flood Protection for Nuclear Power Plants,” as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.3.4 *Technical Evaluation*

The staff reviewed SSAR Section 2.4.3 and confirmed that the information contained in the application addresses the relevant information related to this section. In addition to the systematic review of information provided by the applicant, the staff also visited the site, verified the location and elevation of important streams and hydrologic features, and supplemented this information with other publicly available sources of data. The review topics included the following:

- Design basis for flooding in streams and rivers

- Combined events criteria

- Design basis for site drainage

- Effects of sediment erosion and deposition

- Consideration of other site-related evaluation criteria

In the initial review of the information provided in SSAR Section 2.4.3 and the information gathered during the site visit, the staff determined that the information in SSAR, Revision 0, for the methods of analysis related to identification of the effects of probable maximum flooding on streams and rivers was not sufficiently detailed and substantiated for the staff to assess drainage patterns and independently confirm maximum water levels associated with the PMF. As discussed below, the applicant subsequently provided sufficient and substantiated information regarding the effects of probable maximum flooding on streams and rivers.

Due to the wide and open marine connection of the Delaware River at the PSEG Site, tidal influences and storm surge are the primary drivers in determining the design-basis flood rather than PMF due to precipitation-induced riverine flooding. As discussed previously in Section 2.4.2.5 of this report, potential PMP impacts to site surface-water drainage systems will be evaluated at the COL stage after a reactor technology is selected.

2.4.3.4.1 Design Bases for Flooding in Streams and Rivers

<u>Information Submitted by the Applicant</u>

The applicant used three methods to determine the PMF: two methods simulate flood levels from two different PMP events, and the third method determines the flood level from the Approximate Method from RG 1.59, “Design Basis Floods for Nuclear Power Plants.” Two PMP events were developed using HMR-51 and HMR-52 (References 2.4.3-1 and 2.4.3-2, respectively): the first was designed to yield maximum rainfall over the Delaware Basin and the second was designed to yield more intense rainfall close to, and upstream of the PSEG Site. Of these two PMP events, the one resulting in the highest simulated water level at the PSEG Site was selected. Alternatively, the Approximate Method from RG 1.59, Appendix B, was used to determine a PMF. The analysis producing the highest water level at the plant was selected as the PMF.

After establishing the PMF, the applicant selected the following two combined events (Alternative I and Alternative II) based on American National Standards Institute/American Nuclear Society (ANSI/ANS-2.8-1992), “Determining Design Basis Flooding at Power Reactor Sites” (Reference 2.4.3-3) to arrive at a design-basis flood:

Alternative I

- One-half PMF or 500-year flood (whichever is less)

- Surge and seiche from the worst regional hurricane or windstorm including wind-waves

- 10 percent exceedance high tide

Alternative II

- PMF

- 25-year surge and seiche with wind-waves

- 10 percent exceedance high tide

HEC-HMS (Reference 2.3.4-4) and HEC-RAS (Reference 2.4.3-5) were used to simulate the PMF. Based on PMP results, HEC-HMS was used to calculate the PMF discharge to the Delaware River from the watershed that was then applied to the HEC-RAS model, which simulates Delaware River processes and routes the subbasin runoff to ultimately determine the maximum water level at the PSEG Site. Inputs to the HEC-RAS model included the 10 percent exceedance high tide, and surge and seiche. Wind-waves were calculated based on the USACE Coastal Engineering Manual (Reference 2.4.3-6).

The Staff's Technical Evaluation

The staff reviewed the applicant's design basis methodology for determination of flooding on streams and rivers. The applicant utilized the PMP event computed by deterministic methods (rather than statistical methods), and based on site-specific hydrometeorological characteristics. The staff reviewed HMR-51, HMR-52, and the procedures for estimating PMP. No recurrence interval is normally assigned to the PMP; however, the staff finds that such an event being equaled or exceeded during the plant lifetime is unlikely. The staff considers that use of the PMP meets the requirements to provide a sufficient margin to account for the limited accuracy, quantity, and period of time in which the historical data have been accumulated, and provides an acceptable design basis.

For conservatism, the applicant applied two PMP events as a basis for surface-water modeling to determine the maximum water level at the site from each event. An alternative analysis to determine discharge was performed consistent with the Approximate Method of RG 1.59. For additional conservatism, additional flooding mechanisms for two scenarios were added to the PMF calculated from the surface-water simulations including surge and seiche from the worst regional hurricane or windstorm, and wind-wave activity. The staff considers the bases methodology developed by the applicant adequate. Components of the methodology are described in further detail below.

2.4.3.4.2 Basin Discharge

Information Submitted by the Applicant

The applicant-derived PMP estimates were from HMR-51 isohyetal maps (Reference 2.4.3-1) and applied in a temporal and spatial pattern over the Delaware River basin based on procedures in HMR-52 (Reference 2.4.3-2) to yield maximum runoff. The applicant modeled two PMP events to maximize rainfall throughout the Delaware River Basin, and to yield more intense rainfall close to and upstream of the PSEG Site. The basin wide event that produced the greatest PMP event was determined as a 38,850 km^2 (15,000 mi^2) storm centered over Doylestown, PA and oriented at 222 degrees azimuthal (deg) to produce maximum total rainfall. The upper basin storm close to and upstream of the PSEG Site that produced the greatest PMP was determined to be a 5,568 km^2 (2,150 mi^2) storm in the upper basin centered over Philadelphia, PA with an orientation of 263 deg. The upper basin storm was found to produce the highest flood level at the site through modeling with HEC-HMS and HEC-RAS as described below. A 1-hour PMP of 24.1 cm (9.5 in.) for the upper basin storm was used by the applicant as a basis for estimating the PMF for each of the drainage areas affected.

The applicant then used HEC-HMS based on the results of the two PMP events to determine the runoff hydrograph for the Delaware River Basin. To calculate discharge hydrographs, the applicant used the NRCS curve number method in HEC-HMS to determine precipitation losses.

NRCS soil survey information and USGS land use codes determine the curve numbers. The applicant used Antecedent Moisture Curves (AMC) III curve numbers (Reference 2.4.3-7) to represent ground that is nearly saturated with more than 5 cm (2 in.) of rainfall prior to and within 5 days of the PMP events. Routing of the drainage reaches was conservatively assumed to have no attenuation or diffusion. Prior to the PMP event, tributaries were assumed to flow at an average monthly base flow value based on USGS gauge values. These values were multiplied by the USGS base flow index and used as initial base flows for the HEC-HMS simulations. The USGS base flow index is the ratio of base flow to total flow, expressed as a percentage.

The method resulting in the highest water elevation of 0.79 m (2.6 ft) and discharge (41,852 m^3 (1,478,000 ft^3) per second) adjacent to the PSEG Site was associated with the PMP above for the upper basin. Once this PMF was established, the applicant used ANSI/ANS-2.8-1992 to determine combinations of tide and storm surge to establish an overall flood level associated with the PMF event.

The Staff's Technical Evaluation

The applicant utilized the PMP event computed by deterministic methods (rather than statistical methods), and based on site-specific hydrometeorological characteristics. The PMP has been defined as the most severe reasonably possible rainfall event that could occur as a result of a combination of the most severe meteorological conditions occurring over a watershed. No recurrence interval is normally assigned to the PMP; however, the staff finds that such an event being equaled or exceeded during the plant lifetime is unlikely.

Prior to determining the runoff, the flooding analysis requires the determination of PMP amounts for the site location. The procedures for estimating PMP values for several different durations were reviewed by the staff, and it was concluded that the PMP amounts determined by the applicant are acceptable for the various subbasins in the watershed and are considered by the staff to provide an acceptable design basis. Using these reports, a 1-hour PMP of 24.1 cm (9.5 in.) was used by the applicant as a basis for estimating the PMF for each of the various small drainage areas affected. A typical PMP value is derived for periods of about 1 hour. If the time of concentration is less than 1 hour, it is necessary to extrapolate the data presented in the various hydrometeorological reports to shorter time periods. For example, the applicant used distributions recommended in HMR-52 and determined the 5-minute PMP to be about 15.5 cm (6.1 in.).

Infiltration losses were conservatively determined using the runoff methodology developed by the NCRS (Reference 2.4.3-4). In this AMC III method, a runoff curve number (CN) is estimated for the various subbasins to assume nearly saturated soil conditions prior to the design storm to maximize runoff. Curve numbers incorporate the drainage basin's soils, vegetation, and vegetation density, in addition to the assumed antecedent moisture conditions.

The time of concentration is the amount of time required for runoff to reach the outlet of a drainage basin from the most remote point in that basin. The peak runoff for a given drainage basin is inversely proportional to the time of concentration. The applicant used the lag method, which conservatively assumes no attenuation (lag) or diffusion for the hydrographs used for routing.

Based on a review of the applicant's assumptions and calculations, the staff concludes that the parameters incorporated into the basin discharge model are adequate.

Initially in SSAR, Revision 0, it was unclear to the staff if these rainfall distributions had been appropriately incorporated into the runoff models. Therefore, in RAI 25, Questions 02.4.02-1 and 02.04.02-2, the staff requested that the applicant provide additional information regarding USACE HEC-HMS (Reference 2.4.3-3) and HEC-RAS (Reference 2.4.3-5) input data, and basin-specific drainage details to address these information needs. In a June 23, 2011, response, the applicant submitted the requested information. The staff finds that the applicant appropriately and satisfactorily incorporated the rainfall distributions into the runoff model. Accordingly, the staff considers RAI 25, Questions 02.04.02-1 and 02.04.02-2 resolved.

2.4.3.4.3 Computation of Peak Water Levels

Information Submitted by the Applicant

The applicant used peak discharges computed by HEC-HMS (Reference 2.4.3-3) as input to the HEC-RAS (Reference 2.4.3-5) model, which was then used for simulating Delaware River hydraulic processes and flood water routing downstream through the basin system and for the computation of the peak water levels at the PSEG Site. In the HEC-RAS model, Manning's *n* value for the lower Delaware River was calibrated to range from 0.013 to 0.027 to tide and Trenton stage-discharge data. For the combinatory events including the PMF, the applicant used ANSI/ANS-2.8-1992 alternatives which included 10 percent exceedance high tide and, surge and seiche. Wind wave activity as prescribed in the USACE Coastal Engineering Manual (Reference 2.4.3-6) was included in the resulting peak water level estimate.

The combinatory events as prescribed by ANSI/ANS-2.8-1992 Alternative I produced a maximum peak water level at the plant of 6.40 m (21.0 ft) NAVD88 based on a one-half PMF contribution with 10 percent exceedance high tide (2.01 m (6.6 ft)), surge and seiche from Hurricane Hazel, the most severe regional hurricane on record (3.44 m (11.3 ft)), and coincident wave runup (0.94 m (3.1 ft)).

At the COL stage, the applicant will ensure that site grading adequately routes runoff to swales and pipes away from SSCs to the Delaware River (**COL Action Item 2.4-1**). Once a reactor technology is selected, the design of the intake structure will be determined and, protection from the design basis flood and associated effects will be evaluated at the COL stage (**COL Action Item 2.4-2**). Safety related site grade (11.25 m (36.9 ft) NAVD88) SSCs are at a sufficient elevation above the design basis flood (9.78 m (32.1 ft) NAVD88) to provide for flood protection based on the PPE.

The Staff's Technical Evaluation

The staff reviewed the methods and procedures above and determined them consistent with the suggested criteria in RG 1.59 and current best practices. However, the staff required further information including the applicant's HEC-HMS and HEC-RAS surface-water modeling files and drainage basin details to evaluate and confirm calculations of peak flood flows and surface water levels. In addition, the vertical datum used by the applicant for the surface water elevation data and the identification of gaging stations was unclear. Therefore, in RAI 25, Questions 02.04.02-1 and 02.04.02-2, the staff requested that the applicant provide additional information regarding the surface-water modeling files for staff review. In a June 23, 2011, response, the applicant provided information to clarify water surface datum and elevations, and gaging station locations. The staff reviewed the applicant's HEC-HMS and HEC-RAS input and output files and, associated information and finds the results and information presented in the SSAR adequate. In this response, the applicant provided SSAR revisions that were

subsequently incorporated into SSAR, Revision 1 (May 21, 2012). Accordingly, the staff considers RAI 25, Questions 02.04.02-1 and 02.04.02-2, resolved.

The staff reviewed the description of methods used to determine the PMF as described in the SSAR, and summarized above to ensure that the methods and procedures used were reasonable and adequate. Although the staff noted that some of the model versions used had been superseded by newer versions, these changes were not significant in the ESP analysis based on the staff's independent analysis using the current model versions. Additionally, although the overall resolution of the applicant's basin model was somewhat coarse (e.g., using daily mean discharges rather than 15-minute intervals, daily mean precipitation rather than 15-minute intervals, and large time step intervals compared to the lag times for a majority of the subbasins), the staff recognizes that these assumptions are needed given the large area the model encompasses and associated computational limitations. The staff finds that the modeling conducted by the applicant was adequate for obtaining an appropriately conservative representation of flows resulting from postulated events.

The staff agreed that PMF associated with combinatory events considered by the applicant is appropriate for the PSEG Site. Based on a review of the applicant's information in the SSAR, the staff finds that the applicant has appropriately considered flood-causing phenomena related to the PMF on streams and rivers for the PSEG Site.

2.4.3.4.4 Effects of Sedimentation Erosion and Deposition

Information Submitted by the Applicant

Based on extensive HCGS operating experience at the adjacent HCGS intake structure, and HEC-RAS simulations performed by the applicant, sediment deposition in the vicinity of the intake structure will not result in significant accumulations nor impact SSCs. The applicant will monitor and maintain the intake structure to mitigate any sedimentation effects.

The Staff's Technical Evaluation

The information provided by the applicant is considered to be sufficient to demonstrate that the effect of sediment deposition is negligible. Considering the extensive HGS operating experience and staff's review of the HEC-RAS modeling simulations, the staff considers the applicant's evaluation of sediment deposition adequate.

2.4.3.5 *Post Early Site Permit Activities*

There are no post ESP activities for this section.

2.4.3.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the plant parameter envelope, and that there is no outstanding information required to be addressed in the SSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description to allow the staff to evaluate, as documented in Section 2.4.3 herein, whether the applicant has

met the relevant requirements of 10 CFR 52.17(a)(1) and 10 CFR Part 100 with respect to determining the acceptability of the site. In conclusion, the applicant has provided sufficient information for satisfying the requirements of 10 CFR Part 52 and 10 CFR Part 100.

2.4.4 Potential Dam Failures

2.4.4.1 *Introduction*

SSAR Section 2.4.4 addresses potential dam failures to ensure that any potential hazard to safety-related structures due to failure of onsite, upstream, and downstream water control structures is considered in the plant design.

Section 2.4.4 herein presents a review of the specific areas related to dam failures. The specific areas of review are as follows: (1) flood waves resulting from severe dam breaching or failure, including those due to hydrologic failure, routed to the site and the resulting highest water surface elevation that may result in the flooding of SSCs important to safety; (2) failures of dams in the path to the plant site caused by the failure of upstream dams due to earthquakes and the effect of the highest water surface elevation at the site under the failure conditions; (3) dynamic effects of dam failure-induced flood waves on SSCs important to safety; (4) effects of sediment deposition or erosion during dam failure-induced flood waves that may result in blockage or loss of function of SSCs important to safety; and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.4.2 *Summary of Application*

In SSAR Section 2.4.4, the applicant addresses the site-specific information on potential dam failures. There are no dams downstream of the PSEG Site nor are there any dams on the main stem of the Delaware River. Therefore, downstream dam failures and cascading dam failures were not considered in the applicant's analyses. No safety-related water control structures will be constructed on the site; therefore, failure-induced flooding of onsite water control or storage structures is not considered. In summary, the areas for review consideration include flood waves from severe breaching of upstream dams, simultaneous dam failures, and effects of sediment deposition.

2.4.4.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the identification of floods, flood design considerations, and potential dam failures, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.4.

The applicable regulatory requirements for identifying the effects of dam failures are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirements to consider physical site characteristics in site evaluations are specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used the appropriate sections of the following regulatory guides for the acceptance criteria identified in NUREG-0800, Section 2.4.4:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed.

- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding.

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.

- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.4.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.4. The staff confirmed that the applicant addressed the relevant information related to the flood elevation site characteristics associated with the most severe plausible dam failure event. The staff's technical review of this application included an independent review of the applicant's information in the SSAR. The staff supplemented this information with other publicly available sources of data.

2.4.4.4.1 Dam Failure Permutations

Information Submitted by the Applicant

The applicant observed that there are no dams on the main stem of the Delaware River either up- or down- stream of the PSEG Site. Therefore, the site or water supplies will not be affected by Delaware River dam failures. The applicant identified 24 reservoirs used for water supply, flood control, flow augmentation, and hydropower on Delaware River tributaries (Reference 2.4.4-1). The USACE National Inventory of Dams (NID) database was used to obtain information on dam and reservoir characteristics. Selected groupings of these reservoirs based on storage volume and distance from the PSEG Site were used for the dam breach modeling combinations.

To calculate maximum water level, peak flows and velocities due to postulated dam failure, the applicant used dam breach flows calculated by HEC-HMS (Reference 2.4.4-2) coupled with NOAA bathymetry data and USGS topography information for the Delaware River, tributaries and associated flood plains as input to HEC-RAS to calculate water levels and flow velocities. The resulting high water level at the PSEG Site was the basis for calculating wave runup for the 2-year wind speed in the critical direction. The flow velocities were used in the calculation of sediment deposition to evaluate the potential effects on the safety-related intake structure. The

applicant applied the dam breach analysis based on the approach described in ANSI/ANS-2.8-1992 (Section 9.2.1.2 “Seismic Dam Failures,” Reference 2.4.4-3) and RG 1.59 for combined events criteria associated with dam failure modeling.

To incorporate tidal influences of the Delaware River at the PSEG Site, a 10 percent high tide exceedance was included in the analyses. In summary, the applicant’s analyses were developed considering the following:

- A downstream boundary condition of 10 percent high tide exceedance
- Multiple dam failure peak flows reaching the site simultaneously at high tide
- Full reservoirs at the time of dam breach
- Instantaneous dam failure due to a seismic event

Due to the large areal extent of the Delaware River basin and the spatially variable distributions of dams on its tributaries, the applicant developed permutations of dam failures to evaluate estimated flooding at the PSEG Site. The permutations were based on the largest volumes of water stored and the distance from the PSEG Site. The permutations included the four scenarios in Table 2.4.4-1 below.

Table 2.4.4-1 Summary of Tributary Dam Failure Output Data Excluding Tidal Effects

Name of Dam/ Reservoir	Failure Scenario	Maximum Discharge at Breach (per second)	Discharge at PSEG Site (per second)	Maximum Water Surface at PSEG Site (NAVD88)	Time from Failure to Peak Discharge (days:hours:min)
Pepacton Reservoir	1	7,590,000 ft^3 214,925 m^3	839,000 ft^3 23,758 m^3	0.80 ft 0.24 m	09:22:00
Cannonsville Reservoir		6,530,000 ft^3 184,909 m^3			
Lake Wallenpaupack	2	1,080,000 ft^3 30,582 m^3	721,000 ft^3 20,416 m^3	0.60 ft 0.18 m	09:21:00
Neversink Reservoir		1,790,000 ft^3 50,687 m^3			
F.E. Walter Reservoir	3	2,210,000 ft^3 62,580 m^3	686,000 ft^3 19,425 m^3	0.6 ft 0.18 m	09:22:00
Beltzville Reservoir		1,120,000 ft^3 31,715 m^3			
Nockamixon Reservoir		455,000 ft^3 12,884 m^3			
Blue Marsh Reservoir	4	1,070,000 ft^3 30,299 m^3	634,000 ft^3 17,953 m^3	0.50 ft 0.15 m	09:18:00
Marsh Creek Reservoir		214,000 ft^3 6,060 m^3			
Springton Reservoir (Geist Dam)		113,000 ft^3 3,200 m^3			
Edgar Hoopes Reservoir		51,600 ft^3 1,461 m^3			

Excluding tidal effects, the maximum change in water level of the dam failure scenarios results in an increase in surface-water levels of less than .31 m (1 ft) at the PSEG Site inclusive of the 500-year flood. When tidal effects and wind waves are added, the maximum flood elevation does not exceed 2.87 m (9.4 ft) NAVD88. This scenario includes 10 percent high tide exceedance (1.37 m (4.5 ft) NAVD88) coincident with the 500-year flood (0.61 m (2.0 ft)), the

combined dam failures of Pepacton and Cannonsville reservoirs (0.09 m (0.3 ft)), and the 2-year wind speed in the critical direction (0.79 m (2.6 ft)).

The applicant evaluated suspended sediment accumulation that could affect the operation of the intake structure for the new plant. For the evaluation, the closest reservoirs to the PSEG Site, (Hoopes and Marsh Creek Reservoirs at RM 37 and 53, respectively), were chosen representing 125 years of sediment build-up even though the reservoirs are less than 78 years old. Sediment characteristics were based on soil types with settling velocities determined by Stoke's Law (Reference 2.4.4-4). Based on the simulations, an average of 12.7 cm (5 in.) of sediment build-up results with less occurring at the area of the intake structure at the PSEG Site.

The Staff's Technical Evaluation

In SSAR Section 2.4.4 and as summarized above, the staff reviewed the description of methods used to determine the effects of dam breach to assure that the methods and procedures used were adequate and reflect current and acceptable methods.

In RAI 26, Question 02.04.04-1, the staff requested that the applicant provide more information on sediment deposition conclusions reached in the SSAR. In a June 9, 2011, response, the applicant detailed the description of the analysis, which the staff found adequate. The staff re-ran confirmatory modeling of the four dam breach scenarios using the input and output file information from the applicant's June 23, 2011, response to RAI 25, Question 02.04.01-1. The staff finds that the modeling conducted by the applicant was adequate for obtaining an appropriately reasonable characterization of the effects of the dam breach scenarios. Accordingly, the staff considers RAI 26, Question 02.04.04-1 resolved.

In RAI 26, Question 02.04.04-2, the staff requested that the applicant provide additional information concerning the timing of potential coincident flood waves arriving at the site and the corresponding conceptualization of the flood wave characterization. In a June 9, 2011, response, the applicant provided the requested information and committed to update SSAR Section 2.4.4.1 to note the conservatism used in the analysis. The staff finds the applicant's response adequate, and confirmed that the committed updates were included in SSAR Revision 1 (May 21, 2012). Accordingly, the staff considers RAI 26, Question 02.04.04-2 resolved.

2.4.4.5 *Post Early Site Permit Activities*

There are no post ESP activities related to this section.

2.4.4.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the PPE, and that there is no outstanding information required to be addressed in the SSAR related to this section.

As set forth above, the applicant has provided sufficient information pertaining to dam failures. Therefore, the staff concludes that the applicant has met the requirements 10 CFR 52.17(a), 10 CFR 100.20(c), and 10 CFR 100.23(c) relating to dam failures. Further, the applicant has considered the most severe natural phenomena that have been historically reported for the site and surrounding area with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4.5 Probable Maximum Surge and Seiche Flooding

2.4.5.1 *Introduction*

This section of the SSAR addresses the probable maximum surge and seiche flooding to ensure that any potential hazard to the safety-related SSCs at the proposed site has been considered in compliance with NRC regulations.

This section presents the evaluation of the following topics based on data provided by the applicant in the SSAR and information available from other sources: (1) probable maximum hurricane (PMH) that causes the probable maximum surge as it approaches the site along a critical path at an optimum rate of movement; (2) probable maximum wind storm (PMWS) from a hypothetical extratropical cyclone or a moving squall line that approaches the site along a critical path at an optimum rate of movement; (3) a seiche near the site and the potential for seiche wave oscillations at the natural periodicity of a water body that may affect the elevations of the floodwater surface near the site or cause a low water-surface elevation affecting safety-related water supplies; (4) wind-induced wave runup under PMH or PMWS winds; (5) effects of sediment erosion and deposition during a storm surge and seiche-induced waves that may result in blockage or loss of function of SSCs important to safety; (6) the potential effects of seismic and non-seismic information on the postulated design bases and how they relate to a surge and seiche in the vicinity of the site and the site region; and (7) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.5.2 *Summary of Application*

This section addresses the information related to probable maximum surge and seiche flooding in terms of impacts on structures and water supply.

2.4.5.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the effects of probable maximum storm surge (PMSS), and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.5.

The applicable regulatory requirements for identifying surge and seiche hazards, design considerations, and the associated acceptance criteria, are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d)(3), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used appropriate sections of the following regulatory guides for the acceptance criteria identified in NUREG-0800, Section 2.4.5:

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants,” as it relates to those SSCs intended to protect against the effects of flooding or those associated with the Makeup Water Intake Structure (MWIS).

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.

- RG 1.102, “Flood Protection for Nuclear Power Plants,” as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.5.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.5. The staff’s review confirmed that the information in the application addresses the probable maximum surge and seiche flooding. The staff’s technical review of this section includes an independent review of the applicant’s information in the SSAR and in the responses to the RAIs.

This section describes the staff’s evaluation of the technical information presented in the SSAR Section 2.4.5.

2.4.5.4.1 Methodology

Information Submitted by the Applicant

In SSAR Section 2.4.5, the applicant determined the PMH storm surge still water level by combining the effects of surge at the open Atlantic coast coincident with the 10 percent exceedance high tide. That surge plus tide is propagated through Delaware Bay to the new plant location; and the effects of wind setup resulting from wind stress over Delaware Bay are superimposed, by addition, on the propagated storm surge. The applicant’s overall approach uses the following methods and analysis.

- One-dimensional (1D) Bodine method to determine storm surge at the open coast using PMH parameters from NOAA’s Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields National Weather Service Technical Report NWS-23 (NWS-23) coincident with the 10 percent exceedance high tide.

- HEC-RAS analysis to propagate that surge through Delaware Bay to the site. The approach uses Bodine surge hydrograph as the stage boundary condition at RM 0 and discharge hydrographs generated by HEC-HMS for the Delaware River at Trenton and its major downstream tributaries. The approach includes effects of hurricane-associated precipitation.

- Kamphuis method to determine wind setup at the site caused by winds blowing over the Delaware Bay using PMH parameters from NWS-23.

- NOAA two-dimensional (2D) SLOSH (Sea, Lake, and Overland Surges from Hurricanes) Display Program (Version. 1.61g) data for comparison with Bodine model results at the open coast. The approach uses a Category 4 hurricane. The approach does not model effects from river flow, sea-level rise and 10 percent exceedance high tide included in the still water level (SWL) calculation.

- 2D ADCIRC+SWAN Models to determine final design basis flood PMSS.

The Staff's Technical Evaluation

The staff determined through independent confirmatory analysis that PSEG's application of PMH storm parameters as input in the SLOSH model produces water surface elevations that exceed the publicly available SLOSH Display Program (V. 1. 61 g) data for Category 4 storms in the PSEG Site area. In RAI 67, Question 02.04.05-12, the staff requested that the applicant provide an analysis of the PMH events using a conservative, current practice approach such as those predicted by a 2D storm surge model (e.g., ADCIRC, FVCOM, SLOSH, other) with input from appropriate PMH scenarios and with resolution that captures the nuances of the bathymetry and topography near the project site. In a November 27, 2013, response to RAI 67, Question 02.04.05-12, the applicant submitted a 2D probabilistic storm surge analysis (PSSA) using the ADCIRC (ADvanced CIRCulation) storm surge model driven by hurricanes determined by the Joint Probability Method with Optimal Sampling (JPM-OS). The staff conducted a public telephone conference with the applicant on January 8, 2014, to clarify an apparent inconsistency in the low water level description in SSAR Sections 2.4.5 and 2.4.11, which the applicant committed to address by modifying these two SSAR Sections. During this teleconference, the staff also asked a series of questions regarding the applicant's use of the PSSA for a revised storm surge analysis, which is the first application of the methodology for evaluating flood hazard at a U.S. nuclear power plant site. The staff asked questions regarding models and parameters as well as interpretation of the applicant's results in light of the use of both deterministic and probabilistic models, treatment of epistemic and aleatory uncertainty in the probabilistic models, and the basis and implication of the selected discretization scheme for the JPM-OS integration. Lastly, the staff discussed the need for a regulatory audit in order to gain an in-depth understanding of the overall approach used, modeling assumptions, and results of the storm surge analysis before making safety conclusions concerning the characteristics and assessment of storm surge flooding at the PSEG Site.

From February 4 to 6, 2014, the staff conducted a regulatory audit involving SSAR Section 2.4, "Hydrology," of the application. On March 5, 2014, the staff informed the applicant of significant issues involving the PSSA and corresponding documentation. On April 30, 2014, the applicant requested an exemption from completing the storm surge flood analysis until the COL stage. On June 10, 2014, the staff held a public meeting at the applicant's request, to discuss the bases and rational of the exemption request. During the meeting, the staff suggested that the applicant perform additional 2D deterministic calculations to compare with the original one dimensional (1D) storm analysis to reach a conclusion on the conservatism of the flooding height determination. On June 17, 2014, the staff issued a letter to the applicant denying the exemption request based on the staff's determination that its bases in support of describing special circumstances, as required by regulations, were insufficient to grant the exemption.

On July 10, 2014, the staff held a public meeting at the applicant's request to discuss its approach to a revised response to RAI 67, Questions 02.04.05-12 through 02.04.05-17. The applicant provided an overview of the 2D deterministic ADCIRC storm surge calculations and comparison with the original 1D Bodine calculations in order to establish that the latter produced a more conservative flooding height. During the meeting, the staff provided detailed feedback regarding potential or actual gaps in the applicant's approach. On July 17, 2014, the applicant submitted a response to the staff's June 17, 2014, exemption denial letter. In this letter, the applicant included a schedule for the revised response to RAI 67, Questions 02.04.05-12 through 02.04.05-17.

On August 14, 2014, the staff held a public meeting at the applicant's request to discuss its completed storm surge revised analysis results and the SSAR markups in conjunction with the application review. In an August 21, 2014, letter, the applicant provided a revised response to RAI 67, Questions 02.04.5-12 through 02.04.5-17, which are discussed in the following sections. The applicant's response to RAI 67, Questions 02.04.05-12 and 02.04.05-15 included a regulatory commitment which will result in revisions to the SSAR and Environmental Report (ER). The staff has identified these revisions to the SSAR as **Confirmatory Item 2.4-1**. The applicant provided the following revised methodology:

- The replacement of the November 2013 PSSA analysis with original storm surge analysis based on the PMH storm (NWS23) model with the 1D Bodine storm surge model, coupled with HEC-RAS and wind setup model of Kamphuis.

- The use of a deterministic 2D storm surge analysis using ADCIRC+SWAN (Simulating WAves Nearshore) to provide data for comparison with 1D Bodine model results.

On April 06, 2015, the staff held a public meeting to discuss the staff's evaluation of the applicant's August 21, 2014, revised response to RAI 67. During the meeting, the staff pointed out that the applicant's statement in the RAI response regarding flood protection to safety-related SSCs did not provide a sufficient level of detail for the staff to develop a permit condition to require such flood protection. The applicant stated that they used the PPE approach and until a reactor technology is selected, details on flood protection cannot be available. Instead, the applicant discussed their approach to the completed storm surge revised analysis as well as the results, in particular the 2D deterministic results, the associated SSAR markups submitted with the August 21, 2014, RAI response, and their plan to submit a supplement to the RAI response by April 15, 2015. The applicant stated that SSAR Section 2.4.5 will be revised to describe the use of a 2D model to define the design basis water surface elevation level (WSEL), and the narratives in Subsections 2.4.5.5 and 2.4.5.6 will be revised to emphasize the use of the already applied ADCIRC+SWAN model as a refined modeling approach. The applicant further stated that their PMH Simulation #2 with antecedent WSEL set to projected sea level rise of 0.41 m (1.35 ft) is used to produce the design basis total WSEL of 32.1 ft. NAVD88, and appropriate areas of SSAR Section 2.4 will be updated to show a design basis total WSEL of 9.78 m (32.1 ft) NAVD88. In addition, the applicant stated that the site grade elevation (11.25 m (36.9 ft)) NAVD88) will be established at a level providing for adequate clearance above the design basis flood based on the PPE. The applicant also stated that in the supplement, they will highlight that their Bodine/HEC-RAS/Kamphuis screening process based on NWS 23 is maintained, the description of the Bodine/HEC-RAS/Kamphuis/ CEM model is retained, and a high resolution ADCIRC+SWAN model has been used to perform a refined analysis of the selected PMH storm, establishing design basis flood level of 9.78 m (32.1 ft) NAVD88.

Subsequently, on April 15, 2015, the applicant supplemented their revised August 21, 2014, response to RAI 67. The applicant stated that the design basis Water Surface Elevation (WSEL), as provided in their August 21, 2014, revised response to RAI 67, was established using a 1D model, and that the results from the use of this model are considered unrealistically conservative. In addition, the applicant stated that their deterministic analysis using a 2D, high-resolution storm surge model, submitted on August 21, 2014, provides a conservative, yet more realistic, design basis WSEL. The applicant affirmed that the supplemental response revises the SSAR to credit the results from the high-resolution storm surge model to establish the design basis flood level for the PSEG Site. The applicant provided the following revised hierarchical hazard approach (HHA) methodology:

- 1D Bodine storm surge model, coupled with HEC-RAS and wind setup model of Kamphuis used as a sensitivity analysis and screening method to determine the PMH parameters for the development of the PMSS.

- 2D ADCIRC+SWAN model simulations of the screened PMH parameters in conjunction with Coastal Engineering Manual (CEM) wave runup equations used to determine PMSS design basis flood level.

In the supplemental response, the applicant also provided changes to the SSAR. The staff reviewed the applicant's supplemental information including changes to the SSAR. The staff's evaluation of the applicant's supplemental information involving the 1D and the 2D deterministic analyses, comparison of the results from these analyses, and the applicant's selection of one of their 2D storm surge values as the DBF, is described in the following sections, including the staff's conclusion in Section 2.4.5.6 of this report. The applicant included a regulatory commitment to incorporate the SSAR changes in the next revision of the ESP application. The staff has identified this commitment as **Confirmatory Item 2.4-2**.

2.4.5.4.2 Probable Maximum Winds and Associated Meteorological Parameters

Information Submitted by the Applicant

In SSAR Section 2.4.5.1, the applicant described the PMWS and associated meteorological parameters. The development process for the PMWS applies guidance and data from the NOAA NWS 23 report (1979). The applicant presents the development of the PMWS and associated meteorological parameters in SSAR Section 2.4.5.1.1. The development of the PMWS applies guidance and data from the Dover, DE, weather station. A summary of the applicant's PMH parameters is provided in the table below:

Table 2.4.5-1 ESP Applicant's Probable Maximum Hurricane Parameter Values

Parameter, units	Symbol	Range/Value
Peripheral Pressure, cm (in. of Hg) (mb)	P_w	76.50 (30.12) (1019.98)
Central Pressure, cm (in. of Hg) (mb)	P_o	67.69 (26.65) (902.47)
Radius of Maximum Winds, km (nautical miles, NM)	R	20.4 to 51.9 (11,20 and 28)
Forward Speed, km/hr (knots, kt)	T	48.1 to 77.8 (26, 34 and 42)
Hg = mercury; in. of Hg = one-thirtieth of atmospheric pressure (e.g., 0.49 psia).		

Pressure Drop, ΔP = 3.5 in. of Hg (118.5 mb)

SSAR Section 2.4.5.1 documents that the PMH, as defined by NOAA's, "Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields," National Weather Service Technical Report NWS 23, represents the PMWS at the PSEG Site. As defined by NOAA, the applicant states that the PMH may exhibit a range of meteorological characteristics, so preliminary screening level calculations are performed that identify the PMH bounding characteristics that produce the PMSS at the new plant location. A PMH with R = 51.9 km (28 NM (nautical miles)), T = 29.9 mph (26 kt (knots)), and track direction, from 138 degrees (moving northwest) is used by the applicant to specify the PMWS at the site location. The maximum sustained winds over the ocean are calculated by the applicant to be 145 mph (126 kt); while the maximum winds over Delaware Bay are 126 kt, and maximum winds at the new plant location are 133.5 mph (116 kt). Thus, the applicant's PMH is a relatively strong Category 4 hurricane by the Saffir-Simpson hurricane scale.

To verify that the PMH is the PMWS for the PSEG Site, the applicant averaged the winds at Dover to over 4 hours, a sufficient duration to cause wind setup of Delaware Bay, based on the observations summarized in SSAR Section 2.4.5.1.1. The applicant's analysis shows that 4-hour average winds parallel to the long axis of Delaware Bay did not exceed 35 mph (30 kt) at Dover. The applicant states that the overwater winds are expected to be 57.5 mph (50 kt) when overland winds are 34.5 mph (30 kt) (Reference 2.4.5-1). Therefore, winds of sufficient duration to cause wind setup or seiche did not exceed 57.5 mph (50 kt) over Delaware Bay during the period of 1978 through 2008 (e.g., climatological period use by NWS-23). By comparison, the wind speeds associated with the PMH are 145 mph (126 kt) over Delaware Bay. Therefore, the applicant states that the PMH represents the PMWS for the PSEG Site.

The applicant concluded SSAR Section 2.4.5.1 with a discussion of the appropriateness of using NWS 23. The applicant stated that the PMH parameters in NWS 23 are based on historical data for hurricanes making landfall on the U.S. coasts between 1851 and 1975, and that comparisons of hurricane climatology during the period evaluated in NWS 23 indicate that the NWS 23 parameters for the PMH are still applicable. NOAA published a technical memorandum (Reference 2.4.5-2) analyzing the number and strength of hurricane strikes by decade and location in the United States. The applicant stated that, according to this publication, on average, a Category 4 or stronger hurricane hits the United States once every 7 years. However, in the 35 years from 1970 to 2005, only three Category 4 or larger hurricanes have reached the U.S., which is less than the expected number of 5 in 35 years. Based on this information, the applicant stated that it is reasonable to conclude that the number and strength of hurricanes since NWS 23 was published are not greater than those of hurricanes prior to 1975. The NWS 23 climatological data set includes the relatively active period of 1945 through 1970. Therefore, the applicant concluded that the meteorological criteria for hurricanes affecting the gulf and east coasts of the United States, described in NWS 23, are conservative, even considering potential future climatic variability.

The Staff's Technical Evaluation

The staff evaluated the applicant's PMWS calculations as presented in SSAR Section 2.4.5.1. The applicant's development of the PMWS follows the relevant regulatory criteria. The staff verified the project location and meteorological parameters — central pressure, pressure drop, radius to maximum winds, forward speed, coefficient related to the density of air, and track direction — with the tables and figures provided in NOAA NWS 23. The staff confirmed the track orientation relative to the near shore bathymetric contours through a review of bathymetric contour figures presented during the site audit. However, the SSAR Section 2.4.5.1 did not contain enough information to ensure proper development and evaluation of wind speeds from the PMH storm parameters.

SSAR Section 2.4.5.1 contains statements about maximum storm surge resulting from selected PMH storm parameters; however, evaluation of the influence of different parameters is not possible without a table of results. Therefore, in RAI 39, Question 02.04.05-1, the staff requested that the applicant provide a table of wind speeds developed from the PMH meteorological parameters given in SSAR Section 2.4.5. In a November 22, 2011, response, the applicant provided an acceptable table of speeds based on the PMH meteorological parameters. Accordingly, the staff considers RAI 39, Question 02.04.05-1 resolved.

The staff reviewed the applicant's wind stress coefficient values specified in the Bodine (1971) model and report (Equation 9a). Based on the staff's review of values applied by other recent coastal storm surge studies for the Federal Emergency Management Agency (FEMA), the

applicant's value of 3.3 x 10^{-6} seems reasonable. However, SSAR Section 2.4.5.1 contains no sensitivity results to demonstrate how the applicant's capping the maximum wind drag coefficient influences the final surge values from the Bodine model. Therefore, in RAI 39, Question 02.04.05-3, the staff requested that the applicant provide results of sensitivity testing undertaken to evaluate the effect of modifying the default wind drag coefficient in the Bodine storm surge model. In a December 9, 2011, response, the applicant provided a sensitivity study of wind drag coefficients in the Bodine model, which the staff finds adequate. Accordingly, the staff considers RAI 39, Question 02.04.05-3 resolved.

The staff evaluated the applicant's analyses completed for the PMWS calculations as presented in SSAR Section 2.4.5.1.1. The staff verified the relevant proximity of Dover to the proposed project site. The staff questioned the appropriateness of applying a 31-year measured wind speed record to develop the PMWS condition. The 31-year record may prove sufficient to extrapolate wind speeds at higher return periods. However, with an undefined return period for the PMH, extrapolation of measured wind speeds and comparison to PMH values has challenges. The staff reviewed the applicant's procedure used to develop the overwater wind speed and agrees that the PMH represents the PMWS for the proposed project site.

2.4.5.4.3 Antecedent Water Level

Information Submitted by the Applicant

The applicant used the maximum monthly high tide values from 1987 through 2008 to analyze the NOAA tidal gauge stations upstream and downstream from the PSEG Site to determine the 10 percent exceedance high tide at the site (Reference 2.4.5-3). The applicant stated that this approach for estimating 10 percent exceedance high tide, based on recorded tides, intrinsically includes the effects of sea level anomaly (also known as initial rise). In addition, ANSI/ANS-2.8-1992, Section 7.3.1.1.2, concludes sea level anomaly need not be included when 10 percent exceedance high tide is based on recorded tides. Therefore, sea level anomaly is not included in the applicant's analysis because recorded tide data is used to calculate the 10 percent exceedance high tide.

The applicant's analysis calculates a 10 percent exceedance high tide of 1.3 m (4.3 ft) NAVD88 at the Lewes, DE, NOAA tidal gauge (8557380) at RM 0, and 1.4 m (4.6 ft) NAVD88 for the Reedy Point, DE, NOAA tidal gauge (8551910) at RM 59. Based on these values, the applicant calculated a 10 percent exceedance high tide at the new plant location at RM 52 is determined by linear interpolation to be 1.37 m (4.5 ft) NAVD88.

The applicant briefly discusses the methodology to determine sea level trends near the proposed project site in SSAR Section 2.4.5.4. The applicant used the trend of sea level at the tide gauge with the nearer location to the PSEG Site—the NOAA Reedy Point tidal gauge station. The applicant stated that 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. The applicant's analysis of the NOAA Reedy Point tidal gauge station determined that a monthly sea level trend based on monthly mean sea level data from 1956 through 2006 is 0.35 m (1.14 ft)/century, with an upper 95 percent confidence limit of 0.41 m (1.35 ft)/century (Reference 2.4.5-20). Thus, the maximum flood level determined by the applicant at the new plant location includes 0.41 m (1.35 ft) to conservatively account for sea level rise over the projected 60-year lifespan of the new plant.

The Staff's Technical Evaluation

The staff reviewed the applicant's work outlined in SSAR Section 2.4.5.2.2.1 that details the estimation of the 10 percent exceedance high tide required as part of the maximum storm surge evaluation. The 10 percent exceedance high tide value of 1.37 m (4.5 ft) NAVD88 at the proposed project site provides a reasonable value. The USACE Shore Protection Manual (SPM) (1984) contains an analysis of the tide record at Lewes, DE. The SPM analysis indicates that the SSAR Section 2.4.5.2.2.1 value for the 10 percent exceedance high tide of 1.37 m (4.5 ft) NAVD88 provides a conservative estimate. The staff also reviewed the Reedy Point tidal gauge station data and analysis developed by NOAA, and located at: http://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=8551910. The staff verified that the tide data and analysis match the values contained in SSAR Section 2.4.5.4.

The staff reviewed the Intergovernmental Panel on Climate Change (IPCC) (2007) sea level rise estimates. IPCC (2007) Synthesis Report Table 3.1 provides estimates for sea level change by 2100 for different scenarios with a minimum low-end value near 0.2 m (0.66 ft) and a maximum high-end value near 0.6 m (1.97 ft). Given that IPCC values apply for a 93-year period (2007 to 2100), a 60-year horizon and a linear increase in sea level rise would produce low-end and high-end changes of 0.13 m (0.43 ft) and 0.40 m (1.3 ft). Given the review of the NOAA tidal station data and the IPCC report plots, the sea level change value applied by the applicant (0.41 m (1.35 ft) over 60 years) provides a reasonable estimate.

2.4.5.4.4 Surge Water Levels at the Open Coast

Information Submitted by the Applicant

The applicant's review of historical surges near the PSEG Site determined that Hurricane Hazel and the Chesapeake-Potomac hurricane produced the maximum historical storm surges recorded in Delaware Bay. Of these, the Chesapeake-Potomac hurricane storm center passed closer to the PSEG Site, exhibiting a northwesterly track most similar to the hypothetical storm track of the PMH (References 2.4.5-1 and 2.4.5-4). Based on the storm track and adequate available data related to this storm, the applicant selected the Chesapeake-Potomac hurricane of August 1933 to validate the storm surge model used to determine the PMSS.

The storm surge water levels determined by the Bodine method are used by the applicant as a stage boundary (at the open coast) condition at the mouth of Delaware Bay for the HEC-RAS simulation within the Delaware River estuary. The applicant inputs the PMH identified in SSAR Section 2.4.5.1 into the Bodine calculations which results in a maximum surge elevation of 6.37 m (20.9 ft) NAVD88 at the mouth of Delaware Bay. The applicant value included a fluctuating tide at the mouth of the bay that generates the 10 percent exceedance high tide at the PSEG Site coincident with the peak storm surge.

The applicant validated the Bodine model methodology by reproducing the surge observed during the Chesapeake-Potomac hurricane of 1933. The applicant's pressure distribution and winds associated with this storm are specified as described by Bretschneider (Reference 2.4.5-5) and NOAA (Reference 2.4.5-1). The Bretschneider method reports maximum sustained winds over the ocean of 50 kt (58 mph), and maximum sustained winds over Delaware Bay of 43 kt (50 mph). The simulated storm exhibits maximum winds of 56 kt (64 mph) over the ocean, and 41 kt (47 mph) over Delaware Bay, similar to the wind speeds reported for the Chesapeake-Potomac hurricane.

The applicant's comparison of the Bodine model results with the actual response to the Chesapeake-Potomac hurricane is expressed as storm surge, the difference between actual water levels and the predicted astronomical tide level. The applicant stated that the storm surge calculated at the mouth of Delaware Bay, using the Bodine method, reproduces the observed surge as described by Bretschneider (Reference 2.4.5-5). For example, the applicant's peak storm surge result at Reedy Point, DE, is calculated to be 2.4 m (7.9 ft), while the observed surge at Reedy Point was 2.35 m (7.71 ft). The applicant stated that this margin of error is consistent with comparable models, such as NOAA's SLOSH model which has a stated margin of error of plus or minus 20 percent (Reference 2.4.5-6).

The Staff's Technical Evaluation

Historical Surges

The staff reviewed the applicant's information on the significant historical surge events near the proposed plant site. The Bretschneider (1959) report provides a thorough historical account of the Chesapeake-Potomac hurricane characteristics and surge levels. Without another significant storm to provide an extensive and accurate measured surge data set, the Chesapeake-Potomac hurricane provides the best available validation storm. Notably, the Chesapeake-Potomac hurricane surge at the project site equals approximately 30 percent of the PMH surge at the project site, specifically 2.44 m versus 8.14 m (8 ft versus 26.7 ft). The large difference between the applicant's validation storm and the PMH model surge values introduces some concern that processes that occur during a PMH-level surge event may not occur during lower surge events (such as the validation storm). Examples of different processes that may occur with very large surge levels include more flow over inundated inland areas and changes in the effects of bottom friction given the greater water depths of a PMH-level surge.

Bodine One-dimensional Surge Model

A staff evaluation of the influence of different applicant storm surge parameters is not possible without a table of storm surge parameters and surge level results. Therefore, in RAI 39, Question 02.04.05-2, the staff requested that the applicant provide a table of storm surge levels developed with the Bodine model for the different PMH meteorological parameter combinations given in SSAR Section 2.4.5. In addition, the staff requested that the applicant provide any analyses that demonstrate the influence of varying track direction on surge levels at the open coast and project site. In a November 22, 2011, response to RAI 39, Question 02.04.05-2, the applicant provided an acceptable table of storm surge levels developed on the Bodine model and justification that varying track direction from that used in the analysis would produce the maximum surge at the site. Therefore, the staff considers RAI 39, Question 02.04.05-2 resolved.

The applicant applied a method to satisfy the combined events criteria specified in ANSI/ANS-2.8-1992, Section 9.2.2. The method combines the surge derived from the PMH storm with wind wave activity. The method specifies that the surge coincide with the 10 percent exceedance high tide level. Given the models applied for the study and their range of application and assumptions, the method requires combining results from several models. Specifically, the method requires combining results from models that (1) estimate the surge at the open ocean, (2) propagate the surge through the bay, and (3) determine the wind setup that occurs within the bay. The timing of the model simulation ensures that the 10 percent exceedance high tide coincides with the time of maximum surge at the proposed project site.

The staff reviewed the Bodine (1971) model report and the applicant's model development and application sections in the SSAR. ANSI/ANS-2.8-1992 cites the Bodine model as an acceptable methodology to develop storm surge estimates at the open coast. However, SSAR Section 2.4.5 did not provide enough detail to completely understand and evaluate the model application. Therefore, in RAI 39, Question 02.04.05-4, the staff requested that the applicant provide the Bodine model input files and information on boundary conditions applied in the modeling. In a December 9, 2011, response to RAI 39, Question 02.04.05-4, the applicant provided the Bodine model input files and information on boundary conditions applied in the modeling. Therefore, the staff considers RAI 39, Question 02.04.05-4 resolved.

2.4.5.4.5 Propagation of Surge through Delaware Bay

Information Submitted by the Applicant

The propagation of surge through Delaware Bay is calculated by the applicant using the HEC-RAS computer program. The HEC-RAS model is developed by the applicant using channel geometry and floodplain elevations for the Delaware River between Trenton, NJ, and the head of Delaware Bay determined from the Triangular Irregular Network (TIN) terrain model developed from the USGS National Elevation Dataset (NED) (Reference 2.4.5-8) digital elevation model (DEM), and the NOAA Estuarine Bathymetry DEM (Reference 2.4.5-9). The applicant calibrated the HEC-RAS model using observed tidal data. The calibrated model is then used by the applicant to simulate the propagation of the open coast surge from the mouth of Delaware Bay to the PSEG Site.

The upstream boundary conditions used as inputs to the applicant's HEC-RAS model, consisting of discharge of the Delaware River at Trenton, and discharge of tributaries downstream of Trenton, are based on a 2006 event to account for hurricane-related precipitation. The water levels determined by HEC-RAS, and winds defined by NOAA (Reference 2.4.5-1) for the PMH, are then used by the applicant to determine wind setup at the PSEG Site. The applicant's combination of HEC-RAS surge, which includes the 10 percent exceedance high tide, and Kamphuis wind setup determines the PMH surge still water level at the PSEG Site.

The applicant stated that the effect of winds blowing over Delaware Bay, referred to as wind setup, is calculated using a standard method presented by Kamphuis (Reference 2.4.5-10), and is added to the HEC-RAS simulated water levels. Wind setup depends on wind speed and direction over the center of Delaware Bay; a coefficient accounting for wind and bottom stress; and water depth. The applicant's winds over the center of Delaware Bay at model time step 20.5 hours are 120 kt (138 mph) from the south-southeast, and are determined in accordance with NWS 23 (Reference 2.4.5-1). The applicant's stress coefficient is 3.3×10^{-6} (Reference 2.4.5-5). The applicant stated that the cross-section average depth of water varies with the radius of maximum winds (RM) and time, and is determined from the HEC-RAS water levels and channel geometry. The calculated wind setup at time 20.5 hours is 4.27 m (14.0 ft) at the PSEG Site. The wind setup is added by the applicant to the HEC-RAS water level to determine the still water level: 8.19 m (26.86 ft) at t = 20.5 hours.

The applicant stated that Bretschneider (Reference 2.4.5-5) determined that cross-wind effects on storm surge are virtually negligible (less than 3 percent) upstream of the head of Delaware Bay (upstream of RM 48), and reduces surge on the east side of the estuary at the new plant location (Reference 2.4.5-5); therefore, neglecting cross-wind effects is conservative at the new plant location. The applicant stated that the wind setup algorithm of Kamphuis is a steady-state analytical solution of the fundamental equations governing hydrodynamics, which can be found

in reference texts (References 2.4.5-11 and 2.4.5-12). Its primary assumption, that water levels exhibit a steady state response to varying winds, is considered conservative by the applicant because the bay does not respond to the winds instantaneously.

The Staff's Technical Evaluation

The staff reviewed the HEC-RAS model development and discussion sections in SSAR Section 2.4.5.2.2. HEC-RAS is a one-dimensional model and does not account for flow perpendicular to the primary longitudinal axis of Delaware Bay and estuary. The applicant stated that this limitation should not have a significant effect on the HEC-RAS model's ability to simulate either tide or storm surge at the proposed plant site.

HEC-RAS Model Upgrades

The applicant's November 22, 2011, response to RAI 39, Question 02.04.05-7 lists HEC-RAS model upgrades in Version 4.1 (V4.1) that are not found in Version 4.0 (V4.0) (applied in the SSAR analysis). The applicant's response also states that only two model corrections in V4.1 could affect results for analysis (related to bridge crossings). The staff's comparison of output for bridge crossing data developed with V4.1 and V4.0 of code indicate identical curves. However, the applicant did not conduct a model results comparison between more recent HEC-RAS versions and the model used for the SSAR. Instead, the applicant relied on the development of bridge curves with each model version and on documented differences between the versions. Therefore, in RAI 67, Question 02.04.05-17, the staff requested that the applicant provide a discussion of its V4.0 HEC-RAS model compared to the latest HEC-RAS model version to confirm that there is no effect to any of the HEC-RAS model results from recent software updates. In an August 21, 2014, response to RAI 67, Question 02.04.05-17, the applicant provided details of testing completed to understand differences in model results near the PSEG Site for HEC-RAS V4.0 and V4.1. The staff notes that the results presented in the RAI response demonstrate minimal differences in the river flow and WSEL at the PSEG Site when comparing HEC-RAS V4.0 and V4.1 results.

The staff review of the RAI response indicates the additional testing sufficiently answers the RAI request to confirm that there is no effect to any of the HEC-RAS model results from recent software updates. The comparison of HEC-RAS V4.0 and V4.1 demonstrates that recent software upgrades do not influence model results near the PSEG site. Therefore, the staff considers RAI 39, Question 02.04.05-17 resolved.

HEC-RAS Model Setup

The applicant's HEC-RAS modeling discussion did not provide sufficient detail to analyze the modeling approach and results. Therefore, in RAI 25, Question 02.04.01-1, the staff requested that the applicant provide the HEC-RAS model input files, model control files, calibration procedure, model version, and HEC-RAS modeling report. In a June 23, 2011, response to RAI 25, Question 02.04.01-1, the applicant provided the HEC-RAS model input files, model control files, calibration procedure, model version, and HEC-RAS modeling report. The staff finds that the information submitted by the applicant provided sufficient detail to analyze the modeling approach and results. Therefore, the staff considers RAI 39, Question 02.04.05-5 resolved. However, the staff had the following comments and observations.

- A staff review of the HEC-RAS geometry file and model setup (i.e., MASTER.g01) showed the roadways leading to the two bridges are not included in the model. The

surge flow could have been partially blocked by the roadways. The applicant possibly used an ineffective flow scheme (HEC-RAS model ineffective flow area method).

- During a staff review of the unsteady HEC-RAS model (Delaware River Hydraulic Model) surge calibration run (Plan: Surge calibration 1933), the applicant's animation of the longitudinal water surface profile appeared to indicate model numerical instability during the simulation. The numerical instability occurred to a degree that could affect calibration and model prediction values.

- During a staff review of the unsteady HEC-RAS model (Delaware River Hydraulic Model) PMH surge run (Plan: PMH_R28_T26_25YR_FLD_DYNAMIC), the applicant's animation of the longitudinal water surface profile appears to indicate model numerical instability during the simulation. The instability, which occurs approximately 51.5 km (32 mi) from the PSEG Site, appears very pronounced just before the passing of the peak surge wave for the PMH surge run.

In RAI 67, Question 02.04.05-16, the staff requested that the applicant provide a discussion and justification for the applied HEC-RAS model setup. The staff also requested that the applicant describe any steps taken to minimize the model instabilities, and if steps were taken to reduce model instabilities, to describe how these steps affected the model calibration. In an August 21, 2014, response to RAI 67, Question 02.04.05-16, the applicant provided information related to questions concerning HEC-RAS model bridge approach embankments and model instabilities.

The response related to HEC-RAS model bridge approach embankments provided details of additional model simulations designed to evaluate different approaches to handle the bridge approach embankments within the model. The applicant modified the HEC-RAS model geometry file, which included the approximate bridge roadway approaches using estimates based on available information. The applicant then re-executed the PMH simulation in HEC-RAS and computed the resultant WSELs at the PSEG Site. The results indicated minimal changes in the flow rate and WSEL at three locations near the PSEG Site.

The response related to model instabilities provided a general discussion of the features within the model results and details of additional simulations performed to better understand the model features. The response stated that stability issues are not uncommon in unsteady flow analyses. The oscillating water level pointed out in the RAI is described as a perceived numerical instability. To better understand the significance of the water level oscillations, the applicant completed four additional model tests with each designed to evaluate the sensitivity of the model results to various parameters or model settings changed for the specific test.

The applicant made the following four sensitivity runs:

1. Adjust computational time step from 30 seconds to 5 seconds.

2. Set Theta coefficient = 1.0, (changed from 0.6 in PMH run). The Theta coefficient is a weighting factor applied in the model finite difference calculations.

3. Adjust Options and Tolerances, including keeping Theta = 1.0, increasing maximum number of warm up time steps from 20 to 40, and adjusting Theta during the warm up period from 0.6 to 1.0.

4. Add interpolated cross sections every 610 m (2,000 ft) from RM 56.8 to the most upstream cross section.

The model results indicate, as shown in Table RAI 67-16-2 of the August 21, 2014, response to RAI 67, Question 02.04.05-16, that the current PMH run in the unchanged HEC-RAS model generally produces the most conservative estimate, with the exception that the sensitivity run using a 5-second time step, which has the effect of increasing oscillations rather than decreasing, is 0.01 foot higher at RMs 50.36 and 52 (near the PSEG Site). Given the results, the applicant stated that the results show the current HEC-RAS model is appropriate for the PMH storm surge event and no changes to the HEC-RAS model or resultant water surface elevations are required.

Concerning the bridge geometry portion in RAI 67, Question 02.04.05 16, the staff requested that the applicant provide a discussion and justification for the model setup applied. The August 21, 2014, RAI response provided additional detail to justify the method applied and demonstrated how model results change, given an alternative model setup.

Concerning the model instability portion in RAI 67, Question 02.04.05-16, the staff requested that the applicant describe any steps taken to minimize the model instabilities and, if steps were taken to reduce model instabilities, describe how these steps affected the model results. The August 21, 2014, RAI response detailed four different analyses conducted to understand the instabilities, which occur approximately 51.5 km (32 mi) from the PSEG Site, and the influence of the instabilities on the water levels at the PSEG Site. The RAI response described the rationale and procedure of implementing the four different tests. The results indicated that the different parameter values in the four test cases induce only minimal changes in water level at the PSEG Site. The staff notes that this shows a lack of sensitivity of the WSEL at the PSEG Site to the parameter settings tested. The model results do not significantly alter the model instability signal that led to the RAI. Therefore, it remains unknown to the staff if, and by how much, the model instability affects the downstream water levels near the PSEG Site. However, given the explanations provided in the RAI response and the model results within the RAI response that indicate minimal WSEL changes at the PSEG Site for the four parameter cases tested, the staff finds that the RAI response has sufficiently addressed the concern. Therefore, the staff considers RAI 67, Question 02.04.05-16 resolved.

HEC-RAS Water Levels

The staff reviewed the Bretschneider (1959) report and confirmed the report's statement that crosswind effects result in a water level change equal to about 3 percent of the total water level change caused by longitudinal (long-axis) effects. Notably, the counter-clockwise rotation of hurricane winds near the project site would cause a lowering of the water level on the east side of Delaware Bay and estuary (proposed plant location) as the PMH storm approaches from the sea. After the storm passes, easterly-directed winds could cause an increase in water levels at the proposed project site; however, the surge and wind vector timing must be analyzed to determine if maximum water level conditions would occur.

The staff reviewed the applicant's comparison of model results with observed water levels and surge for the Chesapeake-Potomac (1933) hurricane. SSAR Figure 2.4.5-3, "Comparison of Bodine Method and Observed WSEL in Delaware Bay for the Chesapeake-Potomac Hurricane," presents water level comparisons for the modeled and observed conditions at the mouth of Delaware Bay. SSAR Figure 2.4.5-4, "Comparison of Calculated and Observed Surge at Reedy Point, DE," presents water level comparisons for the modeled and observed conditions at Reedy Point, DE, and includes the tidal record.

In the discussion, the applicant stated that SSAR Figure 2.4.5-3 presents surge (i.e., excluding astronomical tide) comparisons; however, the figure labels designate water level comparisons.

The applicant's discussion for the Reedy Point data presents surge level magnitudes of 2.41 m (7.9 ft) modeled, versus 2.34 m (7.7 ft) observed; however, the figure presents water level records and determination of the surge values that are unclear to the staff. The measured water level record at Reedy Point, DE indicates a higher water level than the simulated value for almost the entire storm. The applicant's statement that the simulated storm surge provides a conservative estimate is valid for only a short time period. The model is not conservative when applied to the total water level at the project site, as the observed peak water levels exceed those modeled (2.5 m (8.2 ft) versus 2.44 m (8.0 ft) NAVD88 at time 27 hours in SSAR Figure 2.4.5-4) at peak conditions. Therefore, in RAI 39, Question 02.04.05-6, the staff requested that the applicant provide additional discussion and verification of the development of water level records, including datum conversions, from the Bretschneider (1959) report. In addition, the staff requested that the applicant clarify the calculation of the storm surge from the observed water levels and tidal record at the Reedy Point, Delaware Station, in SSAR Figure 2.5.4-4, and to ensure that its model predictions are conservative. In a December 9, 2011, response to RAI 39, Question 02.04.05-6, the applicant provided the additional discussion and verification of the development of water level records, including datum conversions, from the Bretschneider (1959) report and clarified the calculation of the storm surge from the observed water levels and tidal record at the Reedy Point, DE Station in SSAR Figure 2.5.4-4. Therefore, the staff considers RAI 39, Question 02.04.05-6 resolved.

The staff evaluated the SSAR Section 2.4.5.2.2.3 discussion of the method to propagate the surge from the mouth of Delaware Bay to the proposed project site. The discussion explains that the applicant used observed tidal data to calibrate the HEC-RAS model. In RAI 39, Question 02.04.05-7, the staff requested that the applicant provide additional information on the testing done to confirm that execution of more recent HEC-RAS model versions (V4.1 released in early 2010) than applied in the SSAR did not result in significant changes to the HEC-RAS model results. In a November 22, 2011, response to RAI 39, Question 02.04.05-7, the applicant provided the additional information on the testing done to confirm that execution of more recent HEC-RAS model versions (V4.1 released in early 2010) did not result in significant changes to the HEC-RAS model results. Therefore, the staff considers RAI 39, Question 02.04.05-7 resolved.

The staff discussed the rainfall event the applicant applied during the HEC-RAS modeling of the PMH surge. The applicant applied a historical rainfall event (June 2006) that produced a basin average rainfall of 15.24 cm (6.0 in.) in the Delaware River Basin. During the site audit, the applicant provided additional discussion of the selection criteria for the rainfall event and stated that the selected rainfall event exceeded the National Hurricane Center guidance for inland flooding related to hurricanes in the project area (approximately 8.38 cm (3.3 in.)). During the site audit, the applicant stated that the rainfall event applied in HEC-RAS represents a 25-yr rainfall event for the study area (http://www.srh.noaa.gov/lub/?n=climate-pcpn-freq-atlas). The staff concludes that the rainfall condition applied during the HEC-RAS simulation of the PMH surge provides a reasonably conservative estimate of precipitation during the PMH event.

The staff reviewed SSAR Section 2.4.5.2.2.3 describing the applicant's inclusion of wind setup in the total surge estimates with a standard method detailed in Kamphuis (2000). The applicant's discussion provided limited details on the development and application of the wind setup model within Delaware Bay. Discussions with the applicant during the site audit provided some additional details, but questions remained concerning the methods applied. For example, it was unclear if the method applied by the applicant considered the shape of the bay in the wind setup analysis.

To better understand the applicant's wind setup method and develop a staff confirmatory analysis that considers the shape of Delaware Bay and estuary, in RAI 39, Question 02.04.05-8, the staff requested that the applicant provide the model setup and input conditions applied to develop the wind-induced water level changes from the mouth of Delaware Bay to the project site. The staff also requested that the applicant provide information related to any additional analysis completed to understand how the shape of Delaware Bay would influence wind-induced water level changes in the Delaware Bay. Therefore, in RAI 67 (follow-up to RAI 39, Question 02.04.05-8), Question 02.04.05-13, the staff requested that the applicant provide the following:

(1) a discussion of depth values applied by the wind setup method. The PSEG response to RAI 39, Question 02.04.05-8 stated that bathymetry along the fetch line is applied in the wind setup model, but the bathymetry values necessary to calculate the total water depth were not clearly provided in the RAI 39 response. The wind setup calculation depends on the total water depth and the bathymetric location applied in the wind setup calculation is important (but not clearly demonstrated). The bathymetry across Delaware Bay varies significantly so the depth value can vary widely depending on where the value is chosen.

(2) a discussion of what wind speed averaging was applied to develop the wind speeds applied in the wind setup calculations. The PSEG response to RAI 39, Question 02.04.05-8 did not clearly describe the wind field averaging method applied in the application of the wind speeds within the wind setup calculation.

In RAI 67, Question 02.04.05-13, the staff also noted that using a conservative, current practice approach, such as those predicted by an execution of a 2D storm surge model (e.g., SLOSH) with input from appropriate PMH scenarios, the applicant will account for the shape of the bay when developing wind-induced water level changes from the mouth of Delaware Bay to the project site approximately 80 km (50 mi) inland. This methodology will negate the need for combining multiple models and methods.

In a December 9, 2011, response to RAI 39, Question 02.04.05-8, the applicant provided the additional information on the model setup and input conditions applied to develop the wind-induced water level changes from the mouth of Delaware Bay to the project site as well as how the shape of Delaware Bay would influence wind-induced water level changes in the bay. In an August 21, 2014, response to RAI 67, Question 02.04.05-12, the applicant provided results from a deterministic storm surge analysis using the ADCIRC storm surge model. In response to RAI 67, Question 02.04.05-13, the applicant provided details of the depth values applied in the wind setup methodology — NOAA's National Ocean Service Estuarine Bathymetry Data Set. A bathymetric profile along the wind setup fetch is generated from the NOAA data to provide the elevation of the bottom of the bay at 10-m (32.8-ft) intervals along the 85.5 kilometer (km) (53.1 mi) fetch. Wind setup is calculated in a step-wise manner at every 10 m (32.8 ft) along the fetch, starting at the midpoint of the fetch and stepping toward the PSEG Site.

The December 9, 2011, response to RAI 39, Question 02.04.05-8 provided additional details of the depth values applied in the wind setup methodology. The details indicate that the depth values come from a reasonable data source with values applied at 10-m (32.8-ft) intervals along the fetch.

The December 9, 2011, response to RAI 39, Question 02.04.05-8 also provided additional details of the wind speed averaging within the wind setup calculation and states that no temporal or spatial wind speed averaging was applied. The method applies the winds from

NWS 23. NWS 23 applies 10-min averaged winds and this averaging period is appropriate for these calculations.

In a December 9, 2011, response to RAI 39, Question 02.04.05-8, the applicant stated that the wind setup is calculated in a step-wise manner at every 10 m along the fetch, starting at the midpoint of the fetch and stepping toward the site. The staff required additional information to understand the rationale for starting the wind setup calculation at the midpoint of the fetch (listed as 85.5 km (53.1 mi)) for winds blowing from the south-southeast in the response to RAI 39, Question 2.04.05-8). Enclosure 3 to the December 9, 2011, response to RAI 39, Question 2.04.05-8 provided a portable document format (PDF) document with wind setup model files. However, the tables in the PDF document do not clearly demonstrate the fetch value applied in the wind setup calculations. Given the size of the opening of Delaware Bay to the Atlantic Ocean, the fetch value should equal entire fetch length and not half of the fetch length (the procedure for calculating wind setup in a lake or enclosed water body). Allowing wind setup to only occur over half of the fetch length will cause an underestimate of the wind setup in a bay. Independent calculations suggest that including the complete fetch (85.5 km (53.1 mi)), could approximately double the wind setup value near the PSEG Site (depending on the wind speeds and water depths applied) and potentially increase the surge by 1.2 m (3.9 ft). However, the 2D ADCIRC results presented in the applicant's response to RAI 67, Question 02.04.05-12 demonstrates the conservatism of the current 1D model results (12.92 m (42.4 ft)). Accordingly, the staff considers RAI 39, Question 02.04.05-8 and RAI 67, Question 02.04.05-13 resolved.

In RAI 67, Question 02.04.05-14, the staff requested that the applicant provide the following:

(1) clarification on the time of maximum still water level provided in the response to RAI 39, Question 02.04.05-9. In the PSEG response to RAI 39, Question 02.04.05-9, the simulation time of maximum still water level (21.0 hours) does not match the maximum still water level in SSAR, Revision 1, Table 2.4.5.1 and "Table RAI 39-9-2" (20.5 hours). The applicant also stated the design flooding condition occurs at simulation time 21.5 hours when SSAR Table 2.4.5-1 and Table RAI 39-9-2 indicate 21.0 hours.

(2) the relationship between the two wind speeds listed in the applicant's response to RAI 39, Question 02.04.05-9, "Table 39-9-1" (Column 2 and Column 4).

In a November 27, 2013, response to RAI 67, Question 02.04.05-14, the applicant provided additional information on the simulation timing of the maximum still water level and the simulation timing of maximum flooding. The response stated that due to the reanalysis of the design basis storm surge using different methodology as discussed in the response to RAI 67, Question 02.04.05-14, this question is no longer applicable to the PSEG Site Early Site Permit Application (ESPA). In an August 21, 2014, revised response to RAI 67, Question 02.04.05-14, the applicant stated that a typographical error was identified in the response text of RAI 39, Question 02.04.05-9 regarding the reported simulation time for maximum SWL and the time of the design flooding condition when SWL plus wave runup reaches its maximum elevation. The correct simulation time of maximum SWL is 20.5 hours. Similarly, the correct simulation time of the design flooding condition when SWL plus wave runup reaches its maximum elevation is 21.0 hours. These values are consistent with SSAR Table 2.4.5-1 and Table RAI 39-9-2. The staff's review of the August 21, 2014, revised RAI 67, Question 02.04.05-14 response confirmed that the additional detail and clarification removes the uncertainty concerning the timing of the maximum still water level.

Concerning the wind speed averaging procedure, Table RAI 67-14-1 provides additional detail on the applicant's application of wind speed within the wave runup calculations. It remained unclear to the staff why different wind averaging time periods — shown in column 7 and ranging from 1 to 5 hours — are applied for analysis periods that feature constant 30 minute increments. Review of the resultant wind speeds in column 10 revealed only modest reduction — approximately 10 percent — in the wind speeds from the base wind speed in column 2. Given the magnitude of the reduced wind speeds in column 10, the wave runup estimates should provide reasonable values at the PSEG Site. In an August 21, 2014, revised response to RAI 67, Question 02.04.05-14, the applicant stated that the wind speed in Column 4 of Table 39-9-1 in applicant's response to RAI 39, Question 02.0.4.05-9 (Reference 3), is the average of the wind speeds in Column 2 for the averaging period shown in Column 6, Table RAI 67-14-1, which includes additional columns with notes describing the procedure used to average the wind speed values and directions is provided to clarify the relationship between the two wind speeds in Table RAI 39-9-1. Therefore, the staff considers RAI 67, Question 02.04.05-14 and RAI 39, Question 02.04.05-9 resolved.

2.4.5.4.6 Coincident Wave Runup

Information Submitted by the Applicant

The applicant presents the methodology to determine wave runup coincident with the PMH surge in SSAR Section 2.4.5.3 and subsections. The section provides estimates of wave runup at the proposed project site.

The applicant determined that the maximum wave runup at the PSEG Site does not occur simultaneously with maximum still water level (SSAR Table 2.4.5-1). The analysis demonstrates that when the still water level reaches its maximum at 8.20 m (26.9 ft) NAVD88 wave runup is 2.1 m (6.9 ft) NAVD88, which combines to an elevation of 10.3 m (33.8 ft) NAVD88. Thirty minutes later, the still water level drops to 8.1 m (26.6 ft) NAVD88 and wave runup increases to 2.4 m (7.9 ft) NAVD88, which combine to 10.5 m (34.6 ft) NAVD88, 0.2 m (0.7 ft) higher than the previous time step.

The Staff's Technical Evaluation

The staff reviewed the wave runup calculation methodology presented in the CEM and applied by the applicant. The wind field surrounding the proposed plant site near the time of maximum surge proves critical to understanding the wave runup. However, SSAR Section 2.4.5.3 did not present enough information for the staff to completely understand and evaluate the methods applied by the applicant to calculate the wave runup at the proposed project site. During the PSEG Site audit, the applicant provided preliminary design drawings to demonstrate the preliminary design of the riprap protection (slope 3H:1V or flatter).

SSAR Section 2.4.5.3 did not adequately describe the wind field surrounding the project site near the time of maximum PMH surge. Therefore, in RAI 39, Question 02.04.05-10, the staff requested that the applicant provide plots that illustrate the wind vector directions and magnitudes at the time of, and at several times before and after, maximum PMH surge.
In addition, the staff requested that the applicant provide wave runup estimates at the proposed project site for these times. In a December 9, 2011, response to RAI 39, Question 02.04.05-10, the applicant provided the wind vector plots and associated wave runup estimates for a riprap embankment at the proposed site. Therefore, the staff considers RAI 39, Question 02.04.05-10 resolved.

In RAI 67, Question 02.04.05-15, the staff requested that the applicant provide additional justification for the equation applied to develop the runup (i.e., justification for the use of a roughness coefficient with the CEM section II-4-4-a(1) equation). In addition, the staff requested that the applicant provide a discussion on the exceedance level of the runup estimate developed and the appropriateness of that exceedance level. In an August 21, 2014, response to RAI 67, Question 02.04.05-15, the applicant provided an explanation of the wave runup methodology described in SSAR Section 2.4.5.3 that has been revised to use the methodology presented in USACE Coastal Engineering Manual (CEM), Chapter VI-5. Enclosure 2 to the response provided a revised SSAR Section 2.4.5.3 that details the methodology used, input parameters at critical time steps, and resulting wave runup values. The revised methodology increases the WSEL due to the PMH event to 12.92 m (42.4 ft) NAVD88.

The staff's review of the wave runup methodology indicated that the analysis applies the method of USACE CEM, Chapter VI-5. Accordingly, the staff considers RAI 67, Question 02.04.05-15 resolved.

After reviewing SSAR Section 2.4.5.3.2, "Wave Runup at the New Plant Location," the staff determined that additional information was needed relative to the details of the analysis to estimate the wind-induced wave runup at the project site. Therefore, in RAI 39, Question 02.04.05-10, the staff requested that the applicant provide details of the equations and parameters applied to estimate the wind-induced wave runup at the project site. Specifically, the staff requested that the applicant provide information on the equations applied, the wind speed averaging calculations, and the breaking ratio applied. In addition, the staff requested that the applicant clearly define the wave heights (maximum versus significant) applied in the equations. In a December 9, 2011, response to RAI 39, Question 02.04.05-10, the applicant provided the details of the equations and parameters applied to estimate the wind-induced wave runup at the project site. Based on its review of the response to RAI 39, Question 02.04.05-10, the staff concluded that applicant demonstrated an adequate understanding of the application of the CEM equations to develop the wave runup at the project site. Accordingly, the staff considers RAI 39, Question 02.04.05-10 resolved.

2.4.5.4.7 Maximum Water Level Associated with the PMH

Information Submitted by the Applicant

The applicant discusses the methodology to determine the maximum water level associated with the PMH at the proposed project site in SSAR Section 2.4.5.5.

The PMH, defined in SSAR Section 2.4.5.1, is determined by the applicant to produce the PMH surge, as defined in RG 1.59. The storm used by the applicant to determine maximum water elevation is the PMH that causes the PMSS as it approaches the PSEG Site along a critical path at an optimum rate of movement. The applicant determined that the maximum water elevation occurs at the time when water levels, including wave runup, peak. At the time of the maximum water level, the still water level at the new plant location is calculated to be 8.1 m (26.7 ft) NAVD88.

The applicant's addition of wave runup, 2.4 m (7.9 ft), creates a water surface elevation of 10.5 m (34.6 ft) NAVD88. A future sea level rise of 0.41 m (1.35 ft) per century is added to the effects of storm surge and wave runup for a PMSS during the projected life of the new plant of 10.9 m (35.9 ft) NAVD88 at the PSEG Site. In an August 21, 2014, response to RAI 67, Question 02.04.05-15, Enclosure 2 to the response provided a revised SSAR Section 2.4.5.3, which details the USACE CEM, methodology used, input parameters at critical time steps, and

resulting wave runup values. The revised wave runup methodology increases the WSEL due to the PMH event to 12.92 m (42.4 ft) NAVD88.

The Staff's Technical Evaluation

The staff evaluated the data and discussion presented in SSAR Section 2.4.5.5. The staff agrees that the timing of the surge components proves critical to the development of the maximum water level at the project site. The applicant's response to the information requested in RAI 67, Question 02.04.05-12, allowed the staff to more completely evaluate the time and magnitude of the wind fields and water levels near the project site before and after the storm passes the proposed project site. In addition, the staff's evaluation of the 2D ADCIRC+SWAN coupled model results shows the timing and magnitude of the storm surge — including wave effects — and wave heights near the PSEG Site. The staff's analysis of the water levels indicates the applicant's ADCIRC+SWAN model simulations produce lower total water levels than the combined Bodine/HEC-RAS/Kamphuis model.

2.4.5.4.8 Sediment Erosion and Deposition Associated with the PMH Surge

Information Submitted by the Applicant

The applicant discussed the evaluation of sediment erosion and deposition patterns associated with the PMH surge at the proposed project site in SSAR Section 2.4.5.6, "Sediment Erosion and Deposition Associated with the PMH Surge."

The applicant stated that the tidal current velocities normally range from 0.61 to 0.91 m/s (2 to 3 ft/sec). The applicant's analysis of velocities determined by the HEC-RAS model's simulation of the PMH surge show that velocities throughout Delaware Bay exceed 1.49 m/s (4.9 ft/sec), while velocities in the river channel near the new plant exceed 2.44 m/s (8 ft/sec). Therefore, the applicant concludes that these calculated current velocities are sufficient to cause re-suspension of natural sediments and cause erosion (Reference 2.4.5-13).

The applicant determined gross deposition by conservatively assuming that all total suspended solids in the water column are deposited within a few days after passage of the hurricane. The applicant stated that observations of total suspended solids (TSS) concentrations in other bays and estuaries shortly after passage of hurricanes indicate that TSS increase approximately tenfold more than normal pre-storm levels (References 2.4.5-14, 2.4.5-15, and 2.4.5-16). TSS levels near the bottom of the Delaware Bay normally range between 450 and 525 milligrams/liter (mg/L) (0.033 lb/ft^3) during the flood and ebb periods in the tidal cycle (Reference 2.4.5-13). Therefore, the applicant concludes that TSS levels immediately after the storm could reach 5,000 mg/L, which is 10 times greater than the normal level of approximately 500 mg/L. The applicant stated that since current velocities are higher in the river channel near the new plant than would generally occur throughout Delaware Bay, net erosion is more likely to occur than net deposition.

The applicant stated that an intake structure would be protected from erosion because net deposition would occur immediately around it. The applicant calculations based on the assumption that 5,000 mg/L of total suspended solids deposit shortly after the passage of a hurricane indicate that deposition is not expected to exceed 5.1 cm (2 in.) of sediment. Thus, the applicant concludes that the effect of the PMSS on sediment deposition and erosion is not expected to adversely affect operation of safety-related SSCs.

The Staff's Technical Evaluation

The information presented in SSAR Section 2.4.5.6 did not adequately explain the possible sediment dynamics near the proposed site during the PMH surge. To understand and evaluate local areas of sediment erosion and deposition requires estimation of the 2D current velocity field (application of a 2D hydrodynamic model). The SSAR analysis assumes uniform sediment deposition in Delaware Bay and estuary. Known 2D flow effects do not support the assumption of uniform deposition and erosion. Details of local sediment erosion and deposition patterns may prove unnecessary should safety-related SSCs not depend on erosion and deposition near the proposed project site. In RAI 39, Question 02.04.05-11, the staff requested that the applicant provide additional information concerning the sediment dynamics near the proposed project site under hurricane-induced current velocities. Analysis of the 2D (horizontal) distribution of sediment erosion and deposition requires estimation of the 2D current velocity field (application of a 2D hydrodynamic model). The applicant provided study results from Celebioglu (2006) that demonstrate relatively minor deposition and erosion depths for a 100-yr flood event based on a coupled hydrodynamic and sediment transport model. While the PMH storm forcing would produce greater current and wave forcing, the amount of erosion and deposition should not greatly exceed the estimate of 5.1 cm (2 in.) provided by the applicant. In addition, the SSAR documentation states that safety-related SSCs will be protected against sedimentation (erosion or deposition) that could affect the integrity of those facilities. This protection provides additional assurance that sedimentation won't affect critical infrastructure.

2.4.5.4.9 Comparative Storm Surge Analyses and Design Basis Flood Level

Information Submitted by the Applicant

Table 2.4.5-2 provides a comparison of the staff's and the applicant's storm surge analyses. The applicant used two other methodologies available from NOAA and NRC to determine storm surge at the open coast: NOAA's SLOSH program and RG 1.59, Appendix C. In an August 21, 2014, response to RAI 67, Question 02.04.05-12, the applicant developed a deterministic 2D storm surge analysis using ADCIRC+SWAN (Simulating WAves Nearshore) to provide data for comparison with 1D Bodine model results. The 2D ADCIRC+SWAN model results produced lower water levels for the storm simulation that matches the NWS-23 PMH forcing. In an April 15, 2015, supplement to the revised response to RAI 67, the applicant described their basis and justification for their selection of 2D ADCIRC+SWAN maximum WSEL of 9.78 m (32.1 ft) as the design basis flood for the new plant location.

RG 1.59 for Storm Surge Analysis

The applicant stated that RG 1.59 is applicable to determine PMH surge levels on open coast sites on the Atlantic Ocean and Gulf of Mexico. Therefore, the applicant concluded that it is appropriate to use this methodology for estimating storm surge up to the mouth of Delaware Bay, but it was not appropriate to use it beyond the area where a hurricane makes initial landfall. As such, the applicant stated that it is not an acceptable method for estimating surge at the PSEG Site. The applicant's RG 1.59, Appendix C, results for the mouth of Delaware Bay were based on interpolating results from Atlantic City, NJ, and Ocean City, MD, and then adjusting to NAVD88. Including the 10 percent exceedance high tide, RG 1.59 estimated a maximum storm surge of 6.61 m (21.7 ft), NAVD88 at the mouth of the Delaware Bay.

2D SLOSH Display Program V. 1.61g

The applicant's SLOSH results were accessed using the SLOSH Display Program v. 1.61g (Reference 2.4.5-7) and adjusted to account for the 10 percent exceedance high tide and NAVD88 datum. The applicant stated that storms presented in the Display Program include a Category 4 storm on the Saffir-Simpson scale, but the Delaware Basin v3 SLOSH dataset does not include a storm with the same parameters as the applicant's PMH determined for the PSEG Site. Using the SLOSH Display Program, the applicant shows the highest surge elevation at the mouth of Delaware Bay is 5.36 m (17.6 ft) NAVD88. Accounting for the 10 percent exceedance high tide indicates a Category 4 storm elevation of 6.04 m (19.8 ft) NAVD88.

The Staff's Technical Evaluation

The Applicant's 2D SLOSH Display Program V. 1.61g

The staff reviewed the applicant's comparison of the Bodine model results at the open coast with the SLOSH Display Program V. 1.61g data. The applicant discussion indicated that the SLOSH data represent a Category 4 (Saffir-Simpson scale) storm, but the discussion did not provide sufficient detail to compare the storm characteristics simulated by the Bodine and SLOSH models. During the site audit, the applicant stated that it was not able to obtain the SLOSH source code from NOAA. Having the source code could have allowed the applicant to execute SLOSH model simulations with the PMH parameters. In RAI 39, Question 02.04.05-7, the staff requested that the applicant provide additional information on the storm parameters for the SLOSH model that developed the SLOSH Display Program V. 1.61g data applied in the study.

The data above allows a more direct comparison of the storm parameters applied to develop the SLOSH (visualization program) and the Bodine model storm surge estimates at the mouth of Delaware Bay and at the proposed project site. SSAR Sections 2.4.5.2.2.2 and 2.4.5.2.2.3 discuss and compare the model results; however, the storm characteristics for each method were not completely explained. In RAI 67, Question 02.04.05-12, the staff requested that the applicant provide an analysis of the PMH events using a conservative, current practice approach such as those predicted by a 2D storm surge model (e.g., ADCIRC, FVCOM, SLOSH, other) with input from appropriate PMH scenarios and with resolution that captures the nuances of the bathymetry and topography near the project site.

Discussions with the applicant during the site audit suggested that the applicant may obtain the SLOSH executable files and conduct SLOSH model simulations. The staff requested results from any SLOSH simulations conducted by the applicant for storms with the PMH parameters. In a November 22, 2011, response to RAI 39, Question 02.04.05-7, the applicant provided the additional information on the storm parameters for the SLOSH model that developed the SLOSH Display Program. This information allowed the staff a more direct comparison of the SLOSH storm parameters and Bodine model storm surge estimates at the mouth of the Delaware Bay and site. In an August 21, 2014, response to RAI 67, Question 02.04.05-12, the applicant provided results from a deterministic storm surge analysis using the ADCIRC storm surge model. Accordingly, the staff considers RAI 39, Question 02.04.05-7 and RAI 67, Question 02.04.05-12 resolved.

Staff SLOSH Analysis

The applicant applied the SLOSH model with publically available storm results from the SLOSH Display Program (V. 1.61g) with intensities comparable to Saffir-Simpson scale Category 4

forcing. However, the applicant did not provide SLOSH model results for storm forcing created to match the PMH storm parameters as provided in NWS 23. Through independent confirmatory analysis, the staff determined that application of PMH storm parameters as input in the SLOSH model produces water surface elevations that exceed the publically available SLOSH Display Program (V. 1.61g) data for Category 4 storms in the PSEG project area. The staff applied the Delaware Basin V3 (DE3) SLOSH grid with storm files developed to simulate various combinations of PMH storm parameters. The staff's SLOSH analysis added 10 percent exceedance tide levels to the final results for comparison to the applicant values. Note that the SLOSH results do not account for wave-induced water level effects (wave setup).

Staff ADCIRC+SWAN

As compared to the Bodine/HEC-RAS/Kamphuis model results, the applicant demonstrated that the ADCIRC+SWAN model results show the coupled 2D modeling system produces lower water levels for the PMH storm forcing (developed from the NWS 23 guidance). The applicant's analysis applied the ADCIRC+SWAN model mesh developed during the FEMA Region III coastal storm surge study with enhanced resolution near the project site. Revised SSAR Figure 2.4.5-10 compares the original and modified resolution near the project site. The increased resolution inserted for the applicant's analysis seems reasonable to the staff given the topography and bathymetry features near the project site and the need to study water levels and waves in the immediate vicinity of the PSEG Site.

As stated previously, the ADCIRC+SWAN model results show the coupled 2D modeling system produces lower water levels for the PMH storm forcing developed from the NWS 23 guidance (as compared to the Bodine/HEC-RAS/Kamphuis model). However, the PMH storm forcing applied represents a single event determined to result in the highest WSEL at the PSEG Site based on results from the Bodine/HEC-RAS/Kamphuis modeling approach. The Bodine/HEC-RAS/Kamphuis modeling approach has limitations developing water levels up a complex inland bay/estuary, so application of this approach as a screening tool could miss storm forcing that produces the highest WSEL at the PSEG Site (located well up the bay). Therefore, application of only the characteristics from the single PMH storm that produced the highest water levels in the Bodine/HEC-RAS/Kamphuis modeling approach required review. To confirm if other NWS 23-derived storm parameter sets produce higher water levels at the project site, the staff conducted independent ADCIRC+SWAN simulations.

The independent ADCIRC+SWAN simulations applied the study model mesh provided by PSEG (originally developed for the FEMA Region III coastal storm surge study with increased resolution near the project site). As a first step, the staff reviewed the model mesh resolution and features and found reasonable resolution to resolve important surge-altering features near the project site and within Delaware Bay.

As a second step in the independent analysis, the staff confirmed the ability to reproduce the PSEG study model results near the project site for similar model settings and storm forcing. The staff executed the PSEG study Hurricane Isabel validation simulation and the PMH storm simulation. The results from the independent Hurricane Isabel and PMH storm simulations showed nearly identical values near the project site with differences in maximum water levels on the order of 0.01 m (0.03 ft). The independent simulation with the PMH forcing applied a slightly modified mesh with a small channel near Cape May, NJ, removed. The initial independent PMH simulation developed water level instability in the small channel located over 64.4 km (40.0 mi) from the project site. Execution of the PMH simulation with the slightly modified mesh showed successful model completion with results almost identical to the PSEG PMH simulation. Given

the size of the channel, the feature should cause a very localized influence on surge and no influence on surge at the PSEG Site. The near-identical model results in the completed independent PMH simulation with the channel removed in the modified mesh demonstrate the lack of influence near the project site.

With confidence in the ability to reproduce the ADCIRC+SWAN results near the project site, the staff next developed and executed simulations for storms with variations in the PMH forcing. The PSEG PMH storm forcing was developed based on the maximum WSEL at the project site from the Bodine/HEC-RAS/Kamphuis modeling results. However, limitations in the Bodine/HEC-RAS/Kamphuis modeling approach may have led to the selection of PMH storm parameter values that do not truly reflect maximum water levels at the project site given possible NWS 23 storm parameter ranges. As listed in the SSAR Revision 3 (March 31, 2014), the NWS 23 meteorological parameters are the following:

- Central pressure, p0 = 26.65 in. of mercury (Hg) (902.5 millibars (mb))
- Pressure drop, Δp = 3.5 in. of Hg (118.52 mb)
- Radius of maximum winds, R = from 11 to 28 NM (20.4 to 51.9 km)
- Forward speed, T = from 26 to 42 kt (48.2 to 77.8 km/hr)
- Coefficient related to density of air, K = 68 (when parameters are in units of in. of Hg and kt)
- Track direction, from 138 degrees (moving northwest)

The PSEG PMH storm applies NWS 23 value for p0, Δp, K, and track direction along with the largest R value (28 NM (51.9 km)), slowest forward speed (26 kt (48.2 km/hr)) — also shown in SSAR Table 2.4.5-4. The staff notes that the selection of these values, as defined in NWS 23, seems reasonable. Importantly, the PSEG PMH simulations applied a landfall location offset 28 NM (51.9 km) southwest from the center of the Delaware Bay mouth (SSAR Figure 2.4.5-1). Given the complexity of the bay shape, selection of the landfall location could significantly influence the storm surge values at the PSEG Site. The Bodine/HEC-RAS/Kamphuis modeling approach does not adequately resolve the bay features or some of the physical processes necessary to accurately develop the storm surge near the PSEG Site. The ADCIRC+SWAN model contains a detailed representation of the bay features and the important physical processes necessary to simulate the influence of the landfall location on the storm surge levels at the PSEG Site. In addition, given the bay geometry, various forward velocities for the storm could induce site-specific changes to the timing and magnitude of the maximum WSEL at the PSEG Site. Given the bay geometry and the NWS 23 parameter ranges, the staff investigated the sensitivity of the landfall location and forward speed on the WSEL near the PSEG Site.

To investigate the sensitivity of the storm surge results to the landfall location, staff executed several additional simulations with the PMH track offset from the original value. The first set of additional simulations features the following storm tracks:

1. PMH storm track shifted 14 NM (25.9 km) to the southwest (SW_14_NM)
2. PMH storm track shifted 14 NM (25.9 km) to the northeast (NE_14_NM)

Near the PSEG Site, the SW_14_NM simulation showed increased maximum water levels as compared to the PSEG PMH simulation with differences near 0.75 ft (0.23 m). The NE_14_NM simulation showed decreased maximum water levels near the PSEG site as compared to the PSEG PMH simulation with differences near 1.52 m (5 ft). Based on this information, the staff executed additional shifted track simulations with the storm track shifted 7 NM to the southwest (SW_7_NM) and 21 NM to the southwest (SW_21_NM). The SW_7_NM and SW_21_NM simulations produced maximum WSEL values between 0 meters and .31 meters (0 and 1 ft) higher than the PSEG PMH simulation; however, the maximum WSEL increase was less than that of the SW_14_NM simulation. These simulations show that modifying the track landfall location can produce higher WSEL at the PSEG Site, but the increase in maximum WSEL is less than 0.31 m (1 ft).

To investigate the sensitivity of the storm surge results to the storm forward velocity, the staff executed two additional simulations with the PMH storm velocity increased to 55.6 km/hr (30 kt) and 63.0 km/hr (34 kt). The staff developed the modified forward velocity storms by altering the wind forcing time step applied in the ADCIRC model control file. Given the goal to evaluate the sensitivity of the WSEL to storm forward velocity, this approach allowed the staff to leverage the existing 2D wind and pressure fields developed for the PMH (with a 48.2 km/hr (26 kt) forward velocity). As compared to the PMH storm forcing results, the model results for the 55.6 km/hr (30 kt) and 63.0 km/hr (34 kt) forward velocities indicate reduced maximum WSEL values near the PSEG site. The maximum WSEL values are reduced by about 0.61 m (2.0 ft) for the 55.6 km/hr (30 kt) simulation and by about 1.22 m (4 ft) for the 63.0 km/hr (34 kt) simulation. These results indicate the 48.2 km/hr (26 kt) forward velocity — the slowest forward velocity in the range provided by NWS 23 — produces the largest WSEL at the PSEG Site.

Detailed review of the ADCIRC+SWAN model PMH simulation results in the immediate vicinity of the PSEG Site revealed some notable maximum water level features that the staff considered needed further investigation. The features presented as undulations in the maximum WSEL with the undulation magnitude on the order of a few feet. Review of the model mesh input file revealed a line of 92 land boundary nodes shaped in an arc that surrounded the north, east, and south side of the PSEG Site and extended into Delaware Bay. The staff did not find documentation for the rationale of including this feature in the model mesh. To evaluate the sensitivity of the maximum WSEL results near the PSEG Site to the node string, the staff removed the node string and executed an ADCIRC+SWAN simulation with the PMH storm forcing. The ADCIRC+SWAN model results for the simulation with the land boundary nodes removed shows similar water level features as compared to the original PMH simulation. Detailed review of the WSEL in contour plots shows no WSEL undulations in the vicinity of where the land boundary nodes were located in the original simulation. The differences in maximum WSEL near the PSEG Site range from approximately +/- 0.03 m (0.1 ft) with the land boundary versus without land boundary simulations. At times other than at maximum WSEL, differences can exceed 0.91 m (3.0 ft). These results indicate that the land boundary nodes, while not having a documented purpose, cause only a minor effect on the water level values near the project.

The staff also compared wave height results for the PSEG PMH simulation and the sensitivity ADCIRC+SWAN simulations. Comparison of significant wave height (Hs) time series at locations near the PSEG Site show similar wave heights and mean periods for the PSEG PMH and staff PMH simulations for most comparisons. The staff's PMH simulation with unexplained land boundary nodes removed produces slightly larger wave heights at some locations near the PSEG Site (locations adjacent the land boundary). The Hs results for the simulation with the land boundary nodes removed reach approximately 0.15 m (0.5 ft) to 0.31 m (1.0 ft) higher than

the PSEG PMH simulation results. For locations closer, the PSEG Site in areas that feature depth limited waves, the difference in Hs is negligible. Since the larger differences in Hs do not exceed 0.31 m (1.0 ft) and locations nearer the PSEG Site show negligible difference, the effect of the land boundary should not cause significant effects on water levels or wave runup.

The staff also executed additional simulations designed to understand the influence of changing the maximum number of SWAN iterations (MXITNS = 2) on wave height within the spectral wave model solution. Recent coastal surge studies have applied different values for the MXITNS parameter, with a value of two representing the low end of the range. The staff executed ADCIRC+SWAN simulations with MXITNS = 8 and MXITNS = 12 to evaluate the sensitivity of the ADCIRC+SWAN model result to the parameter selection. The results of the MXITNS = 8 and MXITNS = 12 simulations show similar wave height and period values near the PSEG Site with values that exceed those of the MXITNS = 2 simulation (original PMH simulation). At the west side of the PSEG Site (location with largest SWAN waves), the higher MXITNS simulations have maximum significant wave heights equal to 2.53 m (8.3 ft) versus 2.04 m (6.7 ft) for the MXITNS = 2 simulation. For locations further from the site, but still in close proximity — labeled "perimeter" locations in the PSEG input files — the higher MXITNS simulations have maximum significant wave heights from 0.31 m (1.0 ft) to 1.37 m (4.5 ft) larger than for similar locations in the PSEG MXITNS = 2 simulation. Review of mean wave periods near the site shows the higher iteration threshold generally reduces the simulation mean wave periods (Tmo1) on the order of 1 to 2 seconds.

Table 2.4.5-2 Storm Parameters and Maximum Total Water Surface Elevation

Parameter	**PSEG ESP Bodine/ HEC-RAS**	**PSEG ESP ADCIRC** Run #1	**PSEG ESP ADCIRC** Run #2 (DBF)	**PSEG ESP ADCIRC** Run #3	**Staff ADCIRC** Confirmatory Run #1 [3]	**Staff ADCIRC** Confirmatory Run #2 [4]
Peripheral Pressure (in. of Hg (mb))	30.12 (1020)	30.15 (1021)	30.15 (1021)	30.15 (1021)	30.15 (1021)	30.15 (1021)
Central Pressure (in. of Hg (mb))	26.64 (902)	26.64 (902)	26.64 (902)	26.64 (902)	26.64 (902)	26.64 (902)
Radius of Maximum Winds (NM (km))	28 (51.86)	28 (51.86)	28 (51.86)	28 (51.86)	28 (51.86)	28 (51.86)
Forward Speed (kt (km/hr))	26 (48.15)	26 (48.15)	26 (48.15)	26 (48.15)	26 (48.15)	26 (48.15)
Max Wind Speed (kt (km/hr))	116 (214.83)	116 (214.83)	116 (214.83)	116 (214.83)	116 (214.83)	116 (214.83)
10% Astronomical High Tide (ft (m))	4.5 (1.37)	4.5 (1.37) [1]	4.5 (1.37) [1]	4.5 (1.37) [2]	4.5 (1.37) [1]	4.5 (1.37) [1]
100-yr Sea Level Rise (ft (m))	1.35 (0.41)	1.35 (0.41) [1]	1.35 (0.41) [2]	1.35 (0.41) [2]	1.35 (0.41) [1]	1.35 (0.41) [1]
Maximum Still Water Level (ft-NAVD88 (m-NAVD88))	26.7 (8.14)	18.63 (5.68)	20.16 (6.14)	25.27 (7.70)	18.63 (5.68)	19.54 (5.96)
Wave Runup (ft (m))	14.3 (4.36) [1]	7.51 (2.29) [1]	7.43 (2.26) [1]	7.74 (2.36) [1]	10.97 (3.34) [1]	9.99 (3.04) [1]
Maximum Total Water Surface Elevation (ft. NAVD88 (m-NAVD88))	**42.4 (12.92)**	**31.99 (9.75)**	**32.09 (9.78)**	**33.01 (10.06)**	**35.46 (10.81)**	**35.38 (10.78)**

1) Added after model simulation to maximum still water level at site

2) Added prior to model simulation for initial sea level

3) Apply PSEG PMH parameters applied in ADCIRC+SWAN model

4) Shift PMH storm track 14 nmi to the southwest in SWAN+ADCIRC model

2.4.5.4.10 Seiche and Resonance

Information Submitted by the Applicant

The applicant discussed the evaluation of seiche and resonance effects at the proposed project site in SSAR Section 2.4.5.7, "Seiche and Resonance."

The applicant stated that the seiche motion in an estuary like Delaware Bay causes the largest water level fluctuations at the head of tide (near Trenton, NJ), while water levels are relatively constant at the mouth of the bay. This type of seiche is called the fundamental mode (Reference 2.4.5-11). The free oscillation period of the fundamental mode seiche propagating along the length of the Delaware Estuary from its mouth at RM 0 to the head of tide at Trenton (RM 134) is 31 hrs.

The applicant stated that shorter length seiche waves (with shorter oscillation periods) are possible. This situation may occur when the effect of winds blowing along the axis of Delaware Bay (northwest-southeast) may excite a seiche within Delaware Bay, but with little effect on the upper estuary, due to the change in orientation of the river in the upper estuary (more nearly northeast-southwest) and less surface area for the wind to act on. Therefore, the applicant concludes that winds from the northwest tend to excite a shorter length wave with greater effect in Delaware Bay and less effect in the upper estuary. Fluctuations in the strength of northwest winds could generate seiche waves of the second mode, which have a period of 10 hrs (Reference 2.4.5-11).

The applicant stated that there are observed water level fluctuations in Delaware Bay that have lower frequency than tides (subtidal), which are semidiurnal (indicating 12-hr periods). The magnitude of these subtidal oscillations at the PSEG Site is less than 0.6 m (2.0 ft). The applicant also stated that these observed water level fluctuations are associated with wind forces of two types. The first type is direct wind stress on the surface of Delaware Bay, while the second is an indirect forcing associated with wind stress fluctuations over the Atlantic Ocean. The applicant's analysis indicated that the fluctuations in wind stress are associated with fluctuations in water levels in the Delaware Bay at periods of more than 3 days. Together, these direct and indirect wind stress fluctuations are associated with nearly all subtidal fluctuations of water surface elevations observed at Reedy Point, DE, 7 mi (11.3 km) from the new plant location (References 2.4.5-17 and 2.4.5-18).

The applicant's analysis of reported observations show that the atmospheric forcing, associated with seiche motion in Delaware Bay, occurs with longer periods (more than 3 days) than the natural period of oscillation of the Delaware Estuary (30 hours or less). Therefore, the applicant concludes that Delaware Bay does not resonate with the meteorologically-induced wave periods.

The applicant stated that Delaware Bay would not resonate with seismic activity. The applicant's analysis showed that seismic waves have a period of 1 hr or less (Reference 2.4.5-19). SSAR Section 2.4.6 documents the effect of tsunami-induced seiche motion in Delaware Bay, showing that the magnitudes of water level fluctuations are too small to affect safety-related SSCs. Therefore, the applicant concludes that due to the lack of resonance with identified forcing functions, as well as observational evidence of the relatively small magnitude of seiche motions, potential seiche waves produce much smaller flood levels than the PMSS.

The Staff's Technical Evaluation

The staff applied the seiche equations presented in the CEM and confirmed the primary and secondary mode periods with representative length and depth values for the Delaware Bay system. Application of an open basin with a length of 215 km (134 mi) and an average depth of 6 m (20 ft) results in a primary seiche mode equal to 31.1 hrs. With the same bay configuration, the first fundamental seiche mode (first harmonic) equals 10.4 hrs. These seiche periods confirm the values stated by the applicant in SSAR Section 2.4.5.7.

The staff reviewed the two studies of subtidal (lower frequency than the tide) water level fluctuations in Delaware Bay (Wong and Hall, 1998; and Wong and Garvine, 1984) referenced in SSAR Section 2.4.5.7. The staff review of the articles confirms the applicant's statements in SSAR Section 2.4.5.7 related to wind effects on subtidal water level fluctuations and the periods of the fluctuations.

The information provided by the applicant and the review conducted by the staff indicate that seiche motion in Delaware Bay should produce water level changes much lower than the PMSS.

2.4.5.5 *Post Early Site Permit Activities*

There are no post ESP activities related to this section.

2.4.5.6 *Conclusion*

The staff accepted the final 1D Bodine and 2D ADCIRC+SWAN methodologies used by the applicant to determine the severity of the surge and seiche phenomena reflected in this analysis, as documented in this section of the report. In the context of the above discussion, the staff finds the applicant's analysis acceptable for use in establishing the design bases for SSCs important to safety. Accordingly, the staff concludes that the use of these methodologies results in an analysis containing a sufficient margin for the limited accuracy, quantity, and period of time in which the data were accumulated.

In order to verify that the applicant's screening 1D storm surge model results of a PMSS with wave runup of 12.9 m (42.4 ft) NAVD88 was very conservative, the applicant conducted several separate, industry-standard 2D analyses of storm surge, resulting in DBF values between 9.75 to 10.06 m (31.99 to 33.01 ft) NAVD88 which is well below the one-dimensional analysis as well as the proposed site grade, and in agreement with the staff's confirmatory analysis. As the 2D ADCIRC+SWAN modeling system represents the current state-of-the-art practice in storm surge hazard assessment, the applicant's PMH maximum WSEL of 9.78 m (32.1 ft) is the DBF. The staff accepted the applicant's PMSS of 9.78 m (32.1 ft) as the DBF noting that it was a very conservative analysis and most realistic of the simulations with the post-addition of the 10 percent exceedance high tide. For example, the highest storm surge of record (8.85 m (29.0 ft) NAVD88) in the U.S. was a result of Hurricane Katrina in New Orleans in 2005. Further, during 2012, when Hurricane Sandy made landfall approximately 120.7 km (75 mi) northwest of the PSEG Site, it resulted in a maximum storm surge of 2.1 m (7.0 ft) NAVD88 near the Oyster Creek Nuclear Generating Station (an operating nuclear facility on the New Jersey coast). Finally, the staff notes that the applicant has established the site grade 1.47 m (4.8 ft) above the maximum flood elevation.

Subject to the resolution of **Confirmatory Items 2.4-1 and 2.4-2** identified in Section 2.4.5.4.1 of this report, the staff concludes that the applicant's identification and consideration of the surge and seiche hazards set forth above is acceptable and meets the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.23(d).

2.4.6 Probable Maximum Tsunami Hazards

2.4.6.1 *Introduction*

This section of the SSAR addresses the hydrological design basis developed to ensure that any potential tsunami hazards to the SSCs important to safety are considered in plant design.

This section presents the staff's review of the flood levels caused by postulated tsunami wave-forming scenarios. The specific areas of the review include the description of the PMT, historical tsunami records, source generator characteristics, tsunami analyses, tsunami water levels, hydrograph and harbor or breakwater influences of a tsunami-like wave, and its effects on safety-related facilities.

2.4.6.2 *Summary of Application*

In SSAR Section 2.4.6, the applicant provides site-specific information about potential tsunami effects on the site.

2.4.6.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the consideration of probable maximum tsunami hazards, design considerations, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.6.

The applicable regulatory requirements for identifying PMT hazards are as follows:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

    - 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The related acceptance criteria are as follows:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed.

- RG 1.29, “Seismic Design Classification,” as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS.
- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.
- RG 1.102, “Flood Protection for Nuclear Power Plants,” as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.6.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.6. The staff confirmed that the information in the application addresses the relevant information related to the PMT. The staff’s technical review of this section includes an independent review of the applicant’s information in the SSAR and the responses to the RAIs.

This section describes the staff’s evaluation of the technical information in SSAR Section 2.4.6.

2.4.6.4.1 Probable Maximum Tsunami

<u>*Information Submitted by the Applicant*</u>

The applicant evaluated potential tsunamigenic sources that could affect the PSEG Site in southern New Jersey (Reference 2.4.6-1). The applicant indicates that the Method of Splitting Tsunami (MOST) model is used to propagate the tsunamis from their sources to the PSEG Site.

In SSAR Section 2.4.6.2, the applicant indicates that tsunami events that could affect the PSEG Site could be generated by a range of near- and far-field geoseismic sources. The near-field sources include submarine mass-failure events associated with slope failures on the continental shelf margin, or large sediment movements in the form of turbidity currents. The applicant suggested that because Delaware Bay is a low-lying coastal-plain estuary bounded by nearly flat terrain, the occurrence of locally generated waves due to subaerial or submarine landslides is unlikely. The far-field sources include coseismic activity in Caribbean subduction zones, including the Hispaniola and Puerto Rico Trenches, and faulting zones in the regions west and south of Portugal that the applicant interprets to be inactive. The applicant indicated that large-scale submarine mass-failure events have also been identified along the Mid-Atlantic Ridge and British Isles, and that catastrophic failure of volcanic cones associated with the island of La Palma in the Canary Islands could generate a tsunami of concern.

The applicant stated that based on previous studies and historical tsunami records, three potential tsunamigenic sources were chosen for further study. These include: a submarine landslide off the coast of North Carolina or Virginia, a volcanic flank failure on La Palma, and submarine fault displacement from an earthquake along the Hispaniola Trench. Analysis of the geology along the Mid-Atlantic continental margin of the United States suggests the presence of historical landslide deposits, and indicates that larger events are commonly associated with low sea levels. The applicant indicated that large submarine landslides along the North Carolina and Virginia coasts could result in large tsunami amplitudes along the United States east coast, where the Currituck landslide is of particular interest among previous events.

The applicant indicated that a volcanic flank failure could result in large tsunami waves along the western Atlantic Ocean boundary (Reference 2.4.6-2), but that more recent studies have suggested smaller amplitudes of 3 m (10 ft) along the United States east coast (References 2.4.6-3 and 2.4.6-4).

The applicant indicated that although the Puerto Rico Trench is commonly suggested as a possible source of tsunamigenic activity, the Hispaniola Trench has a greater tsunamigenic potential. For example, a series of events with M_w between 6.8 and 7.6 occurred between 1946 and 1953 in the Hispaniola Trench. The applicant noted that a set of sources along the Hispaniola Trench that combine to produce a 9.0 M_w event is used here.

The applicant indicated that the amplitudes of the PMT positive runup and negative drawdown at the PSEG Site are computed for each source using the MOST model and that none of the simulations predict tsunami-induced water elevations that result in the design-basis flood at the site.

The Staff's Technical Evaluation

The staff conducted an independent confirmatory analysis to determine the PMT at the PSEG Site; it is described in the sections that follow. The staff considered both far-field seismogenic (Puerto Rico subduction zone (see Figure 2.4.6-1 of this report)) and far-field (Canary Islands (see Figure 2.4.6-2 of this report)) and near-field (Currituck (see Figure 2.4.6-3 of this report)) landslide sources as potential generators for the PMT. Initial analysis indicates that the near-field submarine landslide is the likely source that determines the PMT maximum water level. The PMT minimum water level is determined by a far-field earthquake source along the Puerto Rico subduction zone.

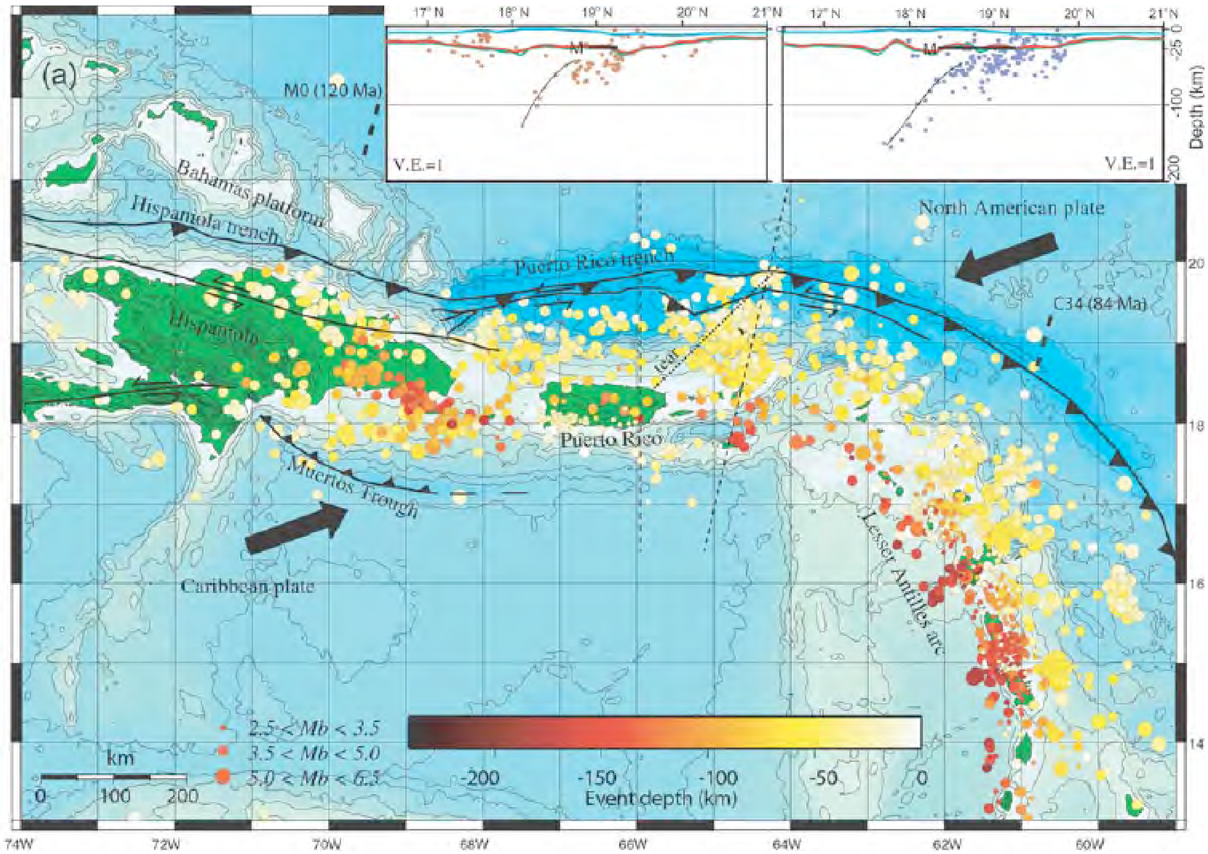

Figure 2.4.6-1 Major faults in the Greater Antilles region

Subduction zone fault represented by line with barbed pattern. Insets show the subduction of the North American plate beneath the Caribbean plate along two different transects. Large arrows show the direction of relative convergence between the two plates. North latitudes are shown (Reference 2.4.6-5).

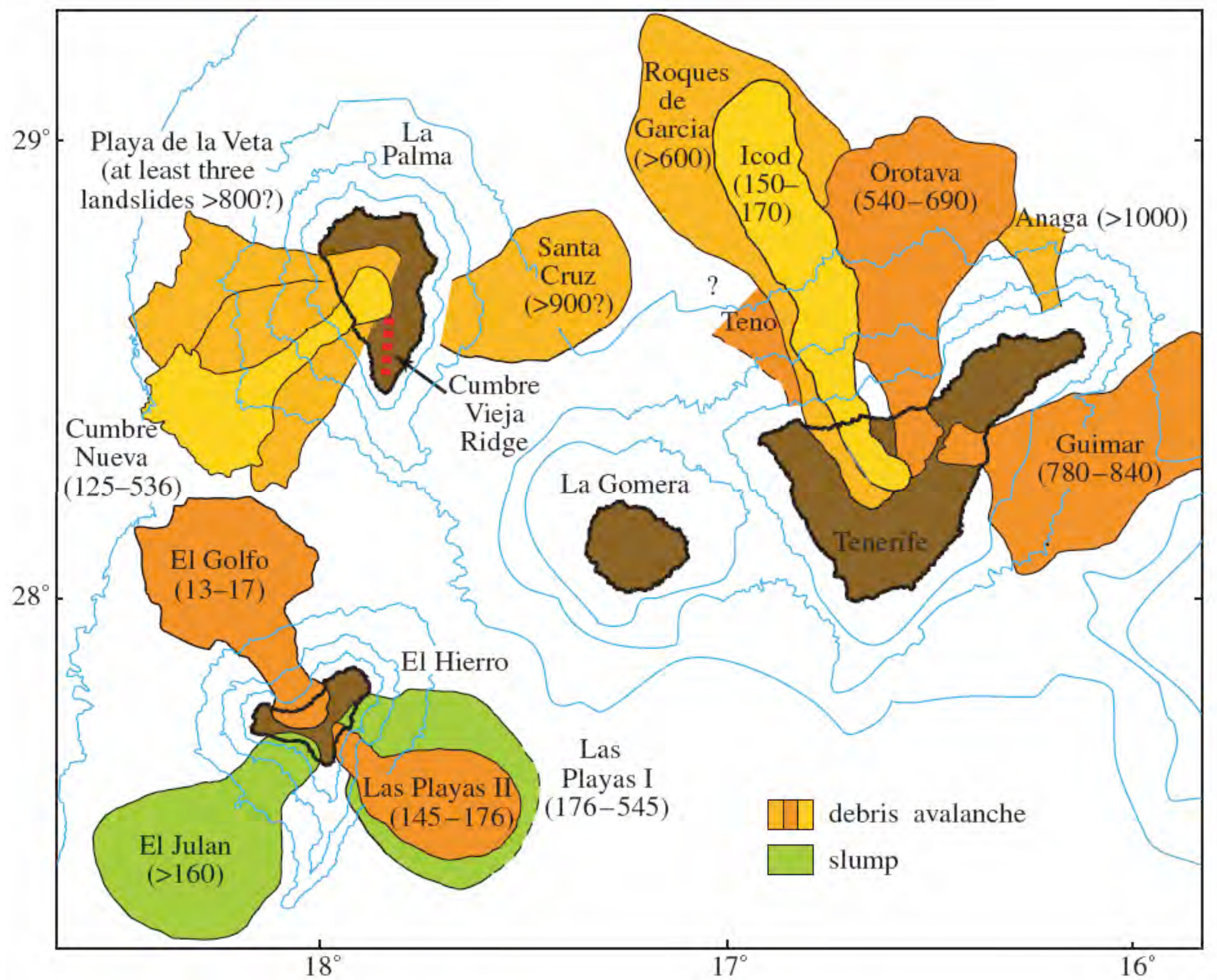

Figure 2.4.6-2 Location and ages (in thousands of years before present) of landslides in the Canary Islands (Reference 2.4.6-6). North latitudes and west longitudes are shown. Bathymetric contour interval is 1 km.

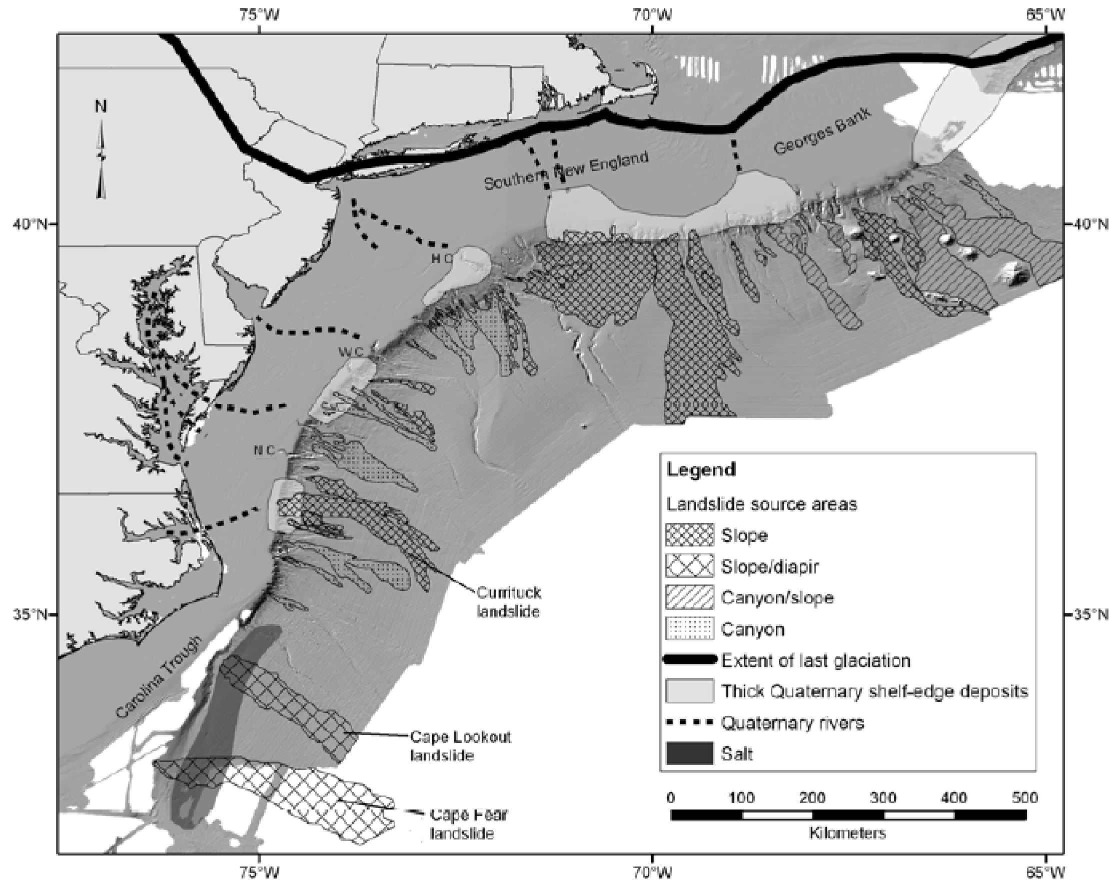

Figure 2.4.6-3 Observed landslides offshore NE Atlantic coast (Reference 2.4.6-7)

In RAI 20, Question 02.04.06-1, the staff requested that the applicant provide additional information, an evaluation, and a discussion in the SSAR of the following items:

1. <u>1918 Puerto Rico Tsunami</u> (SSAR 2.4.6.3). PSEG stated that the 1918 earthquake occurred within the Puerto Rico Trench and that it was responsible for the tsunami. It is believed that the earthquake actually occurred in the Mona Passage or just north of it and that the landslide likely contributed to the tsunami. Provide a clarification of the 1918 earthquake source location.

2. <u>Paleotsunami deposits</u> (Missing from SSAR). Related information is presented in SSAR Section 2.5.1. PSEG stated that for the site no references to paleotsunamis have been found in existing literature, and no evidence of tsunami has been found in site borings.

In the May 11, 2011, response to RAI 20, Question 02.04.06-1, the applicant provided the following:

1. <u>1918 Puerto Rico Tsunami</u>

Current research into the 1918 Puerto Rico Tsunami indicates that the October 11, 1918, Mona Passage earthquake triggered a tsunami that affected the western coast of Puerto Rico. The cause of the tsunami was previously suggested to be seafloor displacement by a normal fault on the eastern wall of the Mona Rift. Using newly available multibeam bathymetry and multichannel seismic reflection profiles, research has identified a submarine landslide with steep

headwall and sidewall scarps 15 km (9 mi) off the northwestern coast of Puerto Rico. Based on this new data it has been postulated that the landslide, which was induced by the earthquake, was responsible for the generation of the tsunami.

The staff verified that in Revision 1 of the ESP application (May 12, 2012), the applicant revised SSAR Section 2.4.6.1.3 to reflect this change in source location.

2. Paleotsunami Deposits

A tsunami deposit is usually identified by sedimentary context such as larger grain size than surrounding sediments, spatial distribution of the deposit, and by ruling out other higher-energy depositional modes (Reference 2.4.6-8).

Samples obtained from site borings were consistent with the fluvial and marine depositional conditions described in published literature and discussed in SSAR Section 2.5.1.2.3.2. The geologic strata at the PSEG Site consist of Lower Cretaceous, Upper Cretaceous, Lower Tertiary (Paleocene), Upper Tertiary (Neogene), and Quaternary formations above the basement rock. The dominant depositional processes for these strata were marine and fluvial over a series of regressive and transgressive events. The Cretaceous/Tertiary boundary was penetrated by the 16 borings performed for the PSEG Site exploration. Review of samples from the borings indicated strata or features that are consistent with the depositional environments described in SSAR Section 2.5.1.2.3.2, and the site samples were not interpreted to represent a paleotsunami occurrence.

Representatives of the New Jersey Geological Survey (NJGS) were contacted to determine if they have any knowledge of geologic evidence for paleotsunamis in the New Jersey area. As a result of the conversations with NJGS, Reference 2.4.6-9 was identified as reporting evidence of tsunami deposits in the New Jersey area. Review of Reference 2.4.6-9 determined that boreholes drilled at Bass River, NJ (approximately 95 km (59 mi) east of the PSEG Site) and at Ancora, NJ (approximately 64 km (40 mi) northeast of the PSEG Site), as part of the Ocean Drilling Program, found a thin (less than 10 cm (4 in.) thick) clast unit above the Cretaceous/Tertiary boundary that appears to be related to a tsunami. The tsunami was not considered related to earthquakes, but is attributed, possibly, to a massive slumping on the Atlantic slope related to the bolide impact near Chicxulub, Mexico, that marked the end of the Cretaceous (Reference 2.4.6-9).

The applicant committed to revise SSAR Sections 2.4.6, 2.5.1 and 2.5.4 to expand on the discussion of paleotsunamis. The staff confirmed that Revision 1 of the ESP application (May 21, 2012) reflects the revised SSAR text and figure. Accordingly, the staff considers RAI 20, Question 02.04.06-1 resolved.

In RAI 20, Question 02.04.06-2, the staff requested that the applicant provide an updated figure showing a maximum slope angle of 0.3 degrees and an updated figure/reference to related work in SSAR Section 2.5.5 in a revision of the SSAR. In a May 11, 2011, response to RAI 20, Question 02.04.06-2, the applicant stated that SSAR Figure 2.4.6-1 has a scale of slope (i.e., dimensionless rise over run) ranging from 0 to 0.002. At the maximum scale value, the angle of the slope would equal 0.115 degrees. To minimize any further confusion over this figure, the applicant revised the figure to have a dimensioned scale in angular degrees. The staff verified that Revision 1 of the ESP application (May 21, 2012) contains the applicant's committed change.

In SSAR Section 2.5.5, the applicant stated that the analysis of slopes will be conducted at the COL stage. SSAR Section 2.5.5.1 discusses the general site slope characteristics and states that analyses will consider potential failure surfaces extending into the Delaware River. The applicant's text also states that portions of the site outside the new plant power block are relatively flat, and that there are no existing slopes on the site, either natural or manmade, that could affect the stability of the site. The applicant committed to revise SSAR Section 2.4.6.2 to reflect the following:

> Figure 2.4.6-1 shows the naturally occurring angular topography slopes on a grid in the vicinity of the PSEG Site, and shows a maximum slope value of 0.3° occurring inland of the site. Stability analysis will be conducted during the COLA phase of the project, and will include consideration of failure surfaces that extend into the Delaware River adjacent to the site as discussed in SSAR Subsection 2.5.5.1.

SSAR Figure 2.4.6-1, provided in Enclosure 3 of the applicant's May 11, 2011, response to RAI 20, Question 02.04.06-2, was revised by the applicant and submitted in a May 31, 2011, supplement to the May 11, 2011, response. The applicant committed to revise the SSAR to provide a scale in angular degrees. The staff confirmed that Revision 1 of the ESP application (May 21, 2012) contains the applicant's committed changes. Accordingly, the staff considers RAI 20, Question 02.04.06-2 resolved.

2.4.6.4.2 Historical Tsunami Record

Information Submitted by the Applicant

SSAR Section 2.4.6.1 provides a list of 10 historical tsunamis that have affected the eastern United States and Canada since 1755. From these, the applicant identified four potential tsunamigenic sources that could affect the PSEG Site: a submarine landslide on the continental shelf along the U.S. east coast; seismic or volcanic sources along the Atlantic Ocean's eastern boundary; coseismic activity in the subduction zones of several Caribbean trenches; and earthquake zones in the North Atlantic Ocean. The applicant indicated that historical records suggest the largest tsunami in the mid-Atlantic region of the U.S. east coast would originate from the first three of these sources.

The applicant suggested a large submarine mass-failure event, known as the Currituck landslide, occurred off the coast of North Carolina in the late Pleistocene era. Simulations of the event (Reference 2.4.6-1) suggest that coastlines immediately facing the slide experienced tsunami amplitudes of about 6 m (20 ft), but that the upcoast and downcoast effects were on the order of 2.01 m (6.6 ft).

The applicant indicated that a significant Atlantic Ocean tsunami generated off the coast of Portugal in 1755 affected the U.S. east coast. However, although runup in Portugal may have been more than 30 m (98 ft), numerical simulations indicate the maximum tsunami amplitudes along the U.S. east coast reached 3 m (9.8 ft) (Reference 2.4.6-10).

The applicant indicated that a tsunamigenic M_w = 7.3 earthquake occurred within the Puerto Rico Trench in 1918. The resulting tsunami caused runup in Puerto Rico of almost 6 m (20 ft), but only 0.06 m (0.2 ft) at a tide gauge in Atlantic City, NJ, 64.37 km (40 mi) northeast of the mouth of the Delaware Bay.

The 1929 M_w = 7.2 earthquake and associated landslide in the Grand Banks caused the largest recorded tsunami in the northern part of the North American east coast. The applicant indicates

that the runup height at the Burin Peninsula (Newfoundland) was 27 m (89 ft), but that the effects were mostly confined to the Newfoundland coast. The applicant noted that the water-level records at Atlantic City suggest a maximum tsunami amplitude of 0.68 m (2.2 ft).

The Staff's Technical Evaluation

The applicant summarized the essential historical record of tsunamis in the region. The staff performed an independent review of the tsunami historical record with respect to the source characteristics needed to determine the PMT. These characteristics include detailed geo-seismic descriptions of the controlling local tsunami generators, including location, source dimensions, and maximum displacement. Based on this review, the staff concluded that the applicant needed to provide additional information regarding the historical record to assist in the characterization of potential tsunami sources that might impact the site.

(1) Other Regional Landslide Sources (Missing from SSAR). Provide description, parameters, and tsunami estimates of other mapped landslide sources that might impact the site, as well as a discussion of how the Currituck was chosen as the primary landslide tsunami source on the continental shelf.

(2) Activity of Offshore Portugal Seismic Zone (SSAR 2.4.6.2 2nd Paragraph). Discuss what the applicant means by "inactive" as applied to the seismic zone offshore Portugal. This is an important consideration with regard to the historical tsunami record and tsunami generating potential from that region.

In a May 11, 2011, response to RAI 20, Question 02.04.06-3, the applicant provided the following:

(1) Other Regional Landslide Sources

The applicant stated that the Currituck slide is one of several apparent Paleolithic slide events occurring on the outer slope of the U.S. East Coast continental shelf. Landslide-generated tsunamis typically cause the greatest levels of inundation on shorelines immediately landward of the slide event. Therefore, the applicant stated that it is most relevant to consider additional historical or potential slides in the Mid-Atlantic Bight region, spanning from the Hudson Canyon to Cape Hatteras. The applicant's review of morphological studies (Reference 2.4.6-11) of slide deposits in this region concluded that the most prominent slides are fluvial in origin, being linked to river delta deposits formed during the late Quaternary low stand of sea level, when the major rivers of the regions reached across the present shelf. In particular, the Currituck slide is associated with the delta of the Susquehanna River. Additional deltas of the Delaware and Hudson Rivers also have associated slide deposits. Information on the distribution of slide volumes (Reference 2.4.6-12) showed that the Currituck slide is the largest slide occurring in the region, making it the most logical candidate for study. SSAR Section 2.4.6 was revised to include this discussion.

(2) Activity of Offshore Portugal Seismic Zone

That applicant stated that the word "inactive" was not intended to minimize the tsunami generating potential from the offshore Portugal region. The applicant revised SSAR Section 2.4.6 to delete the term.

The applicant committed to add References 2.4.6-30 and 2.4.6-31 to SSAR Section 2.4.6.

The staff confirmed that Revision 1of the ESP application (May 21, 2012), reflects the revised SSAR text and references. The staff considers RAI 20, Question 02.04.06-3 resolved.

2.4.6.4.3 Source Generator Characteristics

Information Submitted by the Applicant

The applicant stated that the values used in this study for the source generator characteristics are from available literature sources. For the Currituck landslide, the applicant followed Reference 2.4.6-1 in using a total slide volume of 165 cubic kilometers (216 billion cubic yards), and a vertical slide displacement of 1,750 m (5,742 ft). The applicant indicated that although the source location was initially taken as the location of the actual landslide, three additional locations to the north were also tested.

The applicant also considered the collapse of the flank of the Cumbre Vieja volcano on the island of La Palma in the Canary Islands. The applicant indicated that this hypothetical event has been extensively studied and that the main source input is based on the scientific literature. The applicant indicated that a previous study (Reference 2.4.6-13) using a Boussinesq-type model, predicted that the maximum runup in the Canary Islands was 188 m (617 ft) based on a landslide depth of 1,635 m (5,363 ft). The applicant noted that model predictions of Reference 2.4.6-13 are smaller than those of Reference 2.4.6-2, but larger than those of Reference 2.4.6-3.

For a Hispaniola Trench earthquake, the applicant assumed that the subduction zone slip event occurs along the full length of the trench. The applicant indicated that this event is modeled by dividing the trench into seven segments and that the vertical displacement of each segment is determined using the half-plane solution of Okada (1985) to obtain a $M_w = 9.0$ earthquake.

The Staff's Technical Evaluation

Potential tsunami sources that are likely to determine the PMT at the PSEG Site include subaerial and submarine landslides, near-field intra-plate earthquakes, volcanic eruption and sector collapse, and inter-plate earthquakes. Based on the analysis of currently available data, the staff concludes that the causative tsunami generator for the PMT at the PSEG Site is local submarine landslides. Details are provided below.

Subaerial Landslides

With regard to subaerial landslides, there are no significant coastal cliffs near the PSEG Site that would produce tsunami-like waves that exceed the amplitude of those generated by other sources. The lower Delaware Estuary-Bay region is characterized by gently sloping topography inland, transitioning to a relatively flat coastal plain along the coast, dominated by salt marshes, sandy beaches and dunes, and coastal forests. Coastal elevations do not exceed 3 m (10 ft), except in the Wilmington, DE area where gently sloping hills reach the Delaware Bay, resulting in elevations of approximately 7 m (23 ft) at the coastline (USGS Marcus Hook Quadrangle PA-DE-NJ).

Volcanogenic Sources

According to the Global Volcanism Program of the Smithsonian Institution (http://www.volcano.si.edu/), there are two general regions of volcanic activity that have the potential to generate localized wave activity along the east coast of the United States:

(1) Lesser Antilles; and (2) Canary Islands/Azores/Cape Verdes Islands. Subaerial and submarine eruptive and debris avalanche processes on the volcanic islands of the Lesser Antilles have generated a number of tsunamis over the last 150 years (References 2.4.6-4, 2.4.6-14, and 2.4.6-15). While observations and modeling indicate significant local effects, wave heights attenuate rapidly before reaching other islands within the Lesser Antilles chain (References 2.4.6-15 and 2.4.6-16). Due to the rapid attenuation of wave heights and complicated propagation path created by the islands of the Lesser Antilles themselves, tsunami amplitudes from these volcanoes are unlikely to be significant along the east coast of the United States (Reference 2.4.6-17).

Canary Islands Region: The maximum credible landslide event is a catastrophic volcanic flank failure along the SW flank of La Palma Island. The maximum estimated landslide volume is 500 km^3 (120 mi^3) (Reference 2.4.6-2), though Reference 2.4.6-6 notes that this volume is 2 to 3 times bigger than a typical Canary island landslide and that such landslides often fail as separate (in terms of tsunami generation) sub-events. The geologic age of these landslides range from 13,000-17,000 ybp (Years Before Present (geology)) for the El Golfo landslide on El Hierro Island to over a 1 million ybp (References 2.4.6-6 and 2.4.6-18). From these studies, the age of the Cumbre Nueva landslide for which the maximum credible landslide event is based is 125,000-536,000 ybp. The initial research on the La Palma flank failure (Reference 2.4.6-2) predicts wave heights of 10-25 m (33 ft to 82 ft) on the eastern shore of North America from the 500 km^3 (120 mi^3) landslide volume. The hydrodynamic model used by Ward and Day (Reference 2.4.6-2), however, does not include the effects of non-linear advection or wave breaking. More recent research that incorporates these effects suggests wave heights along the eastern U.S. coast from this failure would be less than 3 m (9.8 ft) (Reference 2.4.6-3) or less than 1 m (3 ft) (Reference 2.4.6-19).

Based on existing evidence, volcanoes along the Lesser Antilles or in the eastern Atlantic Ocean are too far away, unfavorably situated, and/or have modeling to show reduced wave heights along the U.S. east coast.

Intra-Plate Earthquakes

The primary sources of intra-plate earthquakes suitably located to generate tsunamis are the mid-Atlantic Ridge and associated transform faults. Mid-ocean ridge faults are unlikely to generate transoceanic tsunamis because of a low corner magnitude (M_{cm} = 5.82±0.07) (Reference 2.4.6-20). Oceanic transform faults have a higher corner magnitude because there is little vertical displacement associated with strike-slip earthquakes; only small tsunamis can be expected from these fault zones.

Inter-Plate Earthquakes

The Azores-Gibraltar Oceanic Convergence Boundary: The offshore boundary between the African and Eurasian tectonic plates is classified as an oceanic convergence boundary (Reference 2.4.6-21). An M=8.4-8.7 earthquake along this plate boundary offshore of Lisbon in 1755, generated a transoceanic tsunami that was observed in the Caribbean and Canada. The specific faults that make up this plate boundary in the Azores Gibraltar region are highly complex.

Using the statistical analysis of Reference 2.4.6-22, we can estimate the magnitude distribution of earthquakes along the world's oceanic convergence zones. Due to a much smaller sample size in comparison to subduction zones, however, there is much greater uncertainty in the

distribution curves for the earthquakes (Reference 2.4.6-23). The maximum tsunami amplitude offshore of the Delaware Bay entrance from a M=8.4-8.7 Azores-Gibraltar earthquake is approximately 0.5 m (Reference 2.4.6-24). The annual probability for this size earthquake is $1.0 \times 10^{-6} – 2.5 \times 10^{-4}$ (high degree of uncertainty).

The Greater Antilles Subduction Zone: This fault represents the boundary between the North American and Caribbean tectonic plates, in which the North American plate is being subducted (pulled beneath) the Caribbean plate. The types of earthquakes that are generated along subduction zones involve thrust motion with large amounts of vertical seafloor motion and are relatively efficient at generating tsunamis. In comparison, transform plate boundaries involve strike-slip motion and are much less efficient at generating tsunamis. Since the relative convergence direction between the two plates at the Greater Antilles subduction zone is highly oblique to the orientation of the fault, it is possible that there may be a mixed mode of thrust and strike-slip motion for earthquakes at this subduction zone.

Due to the large surface area of these faults, the world's largest earthquakes occur on subduction zone thrusts. As explained in Reference 2.4.6-23, there are several methods to determine the maximum magnitude that can occur on subduction zones. The most conservative method is a statistical fit to the frequency-magnitude distribution of earthquakes (known as the Gutenberg-Richter distribution) that occur on all of the world's subduction zones (Reference 2.4.6-22). Since the length of the Greater Antilles subduction zone may limit the maximum earthquake magnitude possible, parametric and empirical methods are also considered.

The maximum tsunami amplitude offshore of the Delaware Bay entrance from a M=9.1 Greater Antilles subduction earthquake is approximately 1-3 m (3 ft to 10 ft) (Reference 2.4.6-25).

Far-Field Submarine Landslides

Puerto Rico trench: Numerous landslide scarps of various sizes are present along the southern margin of the Puerto Rico trench, primarily within the Arecibo and Loiza amphitheaters, but also elsewhere along the edge of the Puerto Rico-Virgin Island (PR-VI) carbonate platform and within Mona Canyon. While the Aricebo and Loiza amphitheaters were initially considered to each be the result of large, potentially catastrophic slope failures (volume estimates of up to 1,500 km^3; (360 mi^3) References 2.4.6-26 and 2.4.6-27), recent analysis of high-resolution geophysical data and sediment cores suggests that the amphitheaters were created by numerous, smaller failure events (Figure -11; Reference 2.4.6-28). The largest of the landslides identified in Reference 2.4.6-28 has a volume of 22 km^3 (5.3 mi^3). Reference 2.4.6-29 identified a submarine landslide at the head of Mona Canyon northwest of Puerto Rico (volume of 10 km^3 (2.4 mi^3) that may have been initiated by the 1918 Mona Passage earthquake and been the principle source of the tsunami that impacted Puerto Rico and nearby coasts.

East Atlantic Ocean Margins: Numerous submarine landslide scars and mass transport deposits have been identified along the European and African coasts of the Atlantic Ocean (Reference 2.4.6-30). The Storegga (Norway) and Sahara (Africa) landslides are two of the largest and most well studied from these margins. Modeling of the tsunami generated by the Storegga landslide shows significant local wave heights that diminish with distance that correspond to coastal inundations identified by onshore tsunami deposits in Norway, Scotland, etc. The U.S. east coast will likely experience limited or no effect from this tsunami with wave heights lower than that from a local submarine landslide source. No numerical modeling has yet been performed on the 60,000-year-old Sahara Slide, but given its similarity to submarine

landslides along the west North Atlantic margin, it is expected that any transoceanic tsunami will not exceed the effects of a local submarine landslide source.

For the remainder of this section, the staff focuses on submarine landslide sources as the principal generator for the PMT at the PSEG Site.

Local Submarine Landslides

Much, if not all, of the continental slope offshore of the U.S. mid-Atlantic coast has been shaped by geologically recent (Late Pleistocene-Holocene) submarine mass failures (Reference 2.4.6-7). The most recent mapping of this region highlights the prevalence of composite landslides/debris flows, rather than discrete failures, across this region, complicating the determination of tsunami source characteristics. Since it is the best expressed and most well studied of the submarine landslides in the mid-Atlantic region, the maximum credible landslide event in this region is based on the past occurrence of the Currituck landslide (approximately 60 km (37 mi) south of Norfolk Canyon), one of the four largest submarine landslides (in volume) identified along the U.S. east coast.

The Currituck landslide occurred as two subevents that appear to have occurred contemporaneously (Reference 2.4.6-31). The total volume of the landslide is estimated to be 128 km^3 (30 mi^3) in Reference 2.4.6-31 and 165 km^3 (40 mi^3) in Reference 2.4.6-32. As the latter estimate is most conservative, it is used as the maximum credible volume.

Quaternary shelf edge delta deposits derived from the ancestral Delaware, Susquehanna, and Roanoke Rivers likely make up the bulk of the failed material along the mid-Atlantic continental shelf and slope, but some Pliocene strata may have been removed as well (References 2.4.6-32 and 2.4.6-33). Approximately 4-9 m (13 ft to 30 ft) of sediment has accumulated since the Currituck landslide (Reference 2.4.6-32) leading to an estimated age of the failure of between 25,000-50,000 ybp, based on average sedimentation rates of 5 cm/year (2 in./year) for sediment burying the scar and deposits (References 2.4.6-32 and 2.4.6-34).

Seismic Seiches

Seismic seiches are fundamentally a different type of wave than tsunamis. Rather than being impulsively generated by displacement of the sea floor, seismic seiches occur from resonance of seismic surface waves (continental Rayleigh and Love waves) within enclosed or semi-enclosed bodies of water. The harmonic periods of the oscillation are dependent on the dimensions and geometry of the body of water. Seismic seiches have not been recorded along the U.S. east coast.

Evaluation of PSEG Site Geotechnical Boring Logs

An independent analysis of the geotechnical and observation well boring logs collected on behalf of the applicant by MACTEC Engineering and Consulting, Inc. (SSAR Appendix 2AA) primarily within the footprints of the proposed new power block and east of the existing operating station, was conducted to identify any intervals with characteristics commonly associated with tsunami deposits. Logs from 26 borings were reviewed. It should be noted that the borings are not continuously sampled and are primarily a geotechnical tool and, therefore, do not contain detailed stratigraphic, lithologic, or textural descriptions. The PSEG Site sits on an artificial island over what once was a peripheral margin of Delaware Bay/River. Filling of the island began in the early 1900s and was essentially complete by the early 1940s. Sedimentary deposits ('alluvium') below the artificial fill is consistent with an environment that has switched

between estuarine/salt-marsh and higher energy fluvial settings. Due to the limited geologic information and the complicated estuarine/fluvial and artificial fill architecture of the PSEG Site, the evaluation of the boring logs in a paleotsunami deposit sense is inconclusive.

Stratigraphy Encountered in Logs

The PSEG Site lies within the mid-Atlantic coastal plain, which consists of Mesozoic to Recent eastward thickening wedges of unconsolidated fluvio-deltaic and marine sediments that progress seaward across the continental shelf (Reference 2.4.6-35). In the Salem, NJ area, the coastal plain deposits consist of (from oldest to youngest): the Cretaceous Potomac Group; Upper Cretaceous Magothy; Merchantville; Englishtown; Marshalltown; Mt. Laurel; Navesink; and New Egypt formations, Paleocene Hornerstown and Vincentown formations, and the Miocene Kirkwood Formation (Reference 2.4.6-35). Based on the geotechnical logs (SSAR Appendix 2AA), the deepest of the borings (EB-3) encountered is the full Cretaceous to Miocene sequence as well as the overlying alluvium and fill deposits, while the remainder of the borings bottomed in the Mt. Laurel or Marshalltown formations.

2.4.6.4.4 Tsunami Analysis

Information Submitted by the Applicant

In SSAR Section 2.4.6.4, the applicant indicated that the Method of Splitting Tsunami (MOST) model (Reference 2.4.6-36) is used to simulate the three case studies. The MOST model has also been extensively validated and that model operation was verified by comparing numerical results with results from the operational version of the code at the University of Southern California. In addition, the MOST model provides a hierarchical environment that can describe tsunami generation, propagation, and inundation using a system of three nested grids. The grids used in the tsunami hazard analysis include a large-scale grid A, an intermediate-resolution grid B, and a high-resolution grid C that includes the PSEG Site.

The applicant stated that the MOST model is based on the nonlinear shallow-water equations and incorporates bottom friction by using Manning's formula. All three case studies are simulated using a Manning's coefficient (n = 0.01), which is assumed to represent smooth bed conditions and correspond to a conservative, worst-case PMT. Two sets of simulations are performed for each scenario. The first set uses a still water level corresponding to the 10 percent exceedance high tide to determine the maximum runup. The second set uses a still water level corresponding to the 90 percent exceedance low tide to determine the maximum drawdown.

The applicant noted that a water level in Delaware Bay corresponding to the 10 percent exceedance high tide at the PSEG Site represents a static water elevation of 1.4 m (4.5 ft) NAVD88. The applicant indicated that the topographic and bathymetric data used to construct the model domains were obtained from the NOAA National Geophysical Data Center (NGDC) Coastal Relief Model (CRM), the NOAA National Ocean Service (NOS) Arc Global Relief Model (ETOPO 1), and the New Jersey and Delaware Digital Elevation Grids.

The applicant indicated that the large-scale grids differ for each case study, and that these grids were generated based on ETOPO 1 for the La Palma and Hispaniola tsunamis, and include the continental shelf and offshore areas in the Atlantic Ocean for the Currituck landslide. The intermediate-resolution grids are based on CRM, and the same grid is used for the La Palma and Hispaniola case studies. A different grid is used for the Currituck landslide. The high-resolution grids are the same for all three case studies and are based on the CRM and NJ

and DE digital elevation grids. To account for the different datums of the different datasets, the applicant indicated that NOAA's vertical datum transformation tool was used.

The Staff's Technical Evaluation

Numerical Grid Development

The bathymetry/topography grid required by the hydrodynamic model is created via two main sources: (1) the GEBCO 1-minute global elevation database, and (2) 3-arcsec (approximately 90-m) resolution elevation data taken from the NOAA Coastal Relief Model for Delaware Bay. The bathymetry and topography are shifted vertically to account for high tide and sea level rise. Mean high water in the area of the site is 1.63 m (5.35 ft) above mean low water and 0.77 m (2.53 ft) above the NAVD88 datum (data taken from the Reedy Point, DE tidal station). To account for sea level rise, 0.75 m (2.46 ft) is added to the still water level; this value is at the upper limit of the expected 100-yr sea level rise as given in the IPCC 2007 report. Thus, the total vertical shift leads to a still water level of +1.52 m (4.99 ft) NAVD88.

In the Atlantic Ocean Basin, there are known significant potential tsunami source locations. Following the source discussion given in the previous section, most of these can be eliminated as being clearly less energetic than others. For example, for distance earthquake sources, a very large event along the Puerto Rico Subduction Zone will produce a larger wave at the site due to proximity and directionality. Distant landslide generated waves will be controlled by the Canary Island source, which will utilize information from the largest (in volume) published hypothetical event, even if this large volume is debatably implausible. The nearfield landslide source to be examined is the Currituck landslide, which occurred just offshore of the site.

Numerical Simulations – Physical Limits

The purpose of these simulations is to provide an absolute upper limit on the tsunami wave height that could be generated by the three potential sources. Note that these limiting simulations use physical assumptions that are implausible for landslide sources; the results of these simulations will be used to filter out tsunami sources that are incapable of adversely impacting the PSEG Site under even the most conservative assumptions. Specifically, these assumptions are as follows:

1. Time scale of the seafloor motion is very small compared to the period of the generated water wave (tsunami).

2. Bottom roughness, and the associated energy dissipation, is negligible in locations that are initially wet (i.e., locations with negative bottom elevation, offshore).

Assumption 1 simplifies the numerical analysis considerably. With this assumption, the sea surface response matches the change in the seafloor profile exactly. This type of approximation is used commonly for subduction-earthquake-generated tsunamis, but is known to be very conservative for landslide tsunamis (Reference 2.4.6-37). The incorporation of this modeling simplification is driven by the desire to remove specification of the landslide time history, and its large associated imprecision and uncertainty. The initial pre-landslide bathymetry profile, as estimated by examination of neighboring depth contours, is subtracted by the post (existing) landslide bathymetry profile. This "difference surface" is smoothed and then used directly as a "hot-start" initial free surface condition in the hydrodynamic model.

Assumption 2 does not simplify the analysis significantly; however, it does prevent the use of an overly high bottom roughness coefficient, which could artificially reduce the tsunami energy reaching the shoreline. Note that while the offshore regions are assumed to be without bottom friction, such an assumption is too physically unrealistic to accept for the inland regions where the roughness height may be the same order as the flow depth. For tsunami inundation, particularly for regions such as this project location where the wave might inundate long reaches of densely vegetated land, inclusion of some measure of bottom roughness is necessary.

Currituck Landslide Source

As provided in the landslide characterization section, the excavation depth of this slide is approximately 300 m (984 ft). This length provides the trough elevation (i.e., -300 m (984 ft)) of the hot-start initial water surface condition. The horizontal dimensions of the slide source region are ~20 km (12 mi) in width and 50 km (31 mi) in length.

For this tsunami hazard investigation, the simulation domain was divided into two separate, but coupled, components – an offshore domain and a nearshore domain. First, a simulation was performed to look at the waves near the offshore source and their evolution in shallow water approaching the Delaware Bay. These simulations provided a time series of water surface elevation and fluid velocity near the Delaware Bay entrance. These time series were then used to force the nearshore domain, which encompasses the entire Delaware Bay. The two domains, offshore and nearshore, were both too large in memory and computational requirements to be run simultaneously.

The Currituck landslide is the largest estimated submarine landslide in the region, thus the staff performed one-horizontal-dimension (1D) and two-horizontal-dimension (2D) simulations to examine the offshore source. The 1D simulations do not include the radial spreading and refraction effects. Physically, a 1D simulation is approximating a simultaneous slope failure of the entire continental shelf along the eastern seaboard.

First, results from the 1D offshore domain are discussed. The depth transect is taken from the source location directly to the Delaware Bay entrance. A constant spatial grid size of 25 m (82 ft) is used across the transect for the 1D cases. The simulation is based on the fully nonlinear Boussinesq equations, with wave breaking included. Note that the entire bottom profile is submerged, and thus there is no bottom friction dissipation in any form in this simulation. Although the generated wave is initially characterized as a leading depression wave, this depression is quickly overrun by the following and faster-moving positive elevation wave. The wide shallow shelf leads to a depth-limiting effect on the wave height. This height decreases from approximately 200 m (656 ft) at the shelf break to approximately 40 m (131 ft) near the Delaware Bay entrance. By this time, the incident wave has transformed into a long period pulse of positive elevation energy.

While there is little in the literature to evaluate these results in any context, these records can be compared with the numerical simulations presented in Reference 2.4.6-38. In Reference 2.4.6-38, attempts were made to simulate the waves directly from an assumed landslide motion (i.e., to generate the waves physically from the bottom boundary condition rather than use an initial hot start condition). In addition, the wave on the shelf was simulated in 1HD, similar to this NRC study. In the Reference 2.4.6-38, the tsunami elevation near the shoreline was approximately 6 m (20 ft), while at the shelf break it was approximately 15 m (49 ft). The difference in reduction factors, 6/15=0.4 from Reference 2.4.6-38 and 40/200=0.20 from this NRC study, is attributed to the depth-limiting effect. With long lengths of shallow depth

propagation, large amplitude waves will be dissipated – here meaning reduced in amplitude -- much faster than relatively smaller waves.

Next, with a time series from the 1HD offshore simulation taken near the Delaware Bay entrance, the nearshore domain simulation can proceed. The nearshore domain uses a constant spatial grid size of 100 m (328 ft). The simulation is based on the fully nonlinear Boussinesq equations, with wave breaking included. On initially dry land, bottom friction due to a roughness characteristic of a smooth, even surface (Manning's n=0.02) is employed; elsewhere again there is no friction. Note that the elevations given in these figures are relative to the simulation datum of 1.52 m (+4.99 ft) NAVD88; 1.52 m (4.99 ft) should be added to the presented values in order to convert them to a NAVD88 elevation. Of immediate note is the rapid attenuation of wave height through the entrance of the Delaware Bay. The tsunami elevation immediately offshore of the Delaware Bay is greater than 20 m (65.6 ft), yet 20 km (12.5 mi) up channel, the maximum elevation is close to 12 m (39.4 ft). The wave height continues to diminish as the wave propagates further up channel due to directional interference. Near the PSEG Site, the maximum 1HD water elevation reaches 8.6 m (+28.2 ft) NAVD88.

The maximum 1HD values of water surface elevation show a rapid decrease in wave height near the entrance. Similarly, the largest recorded fluid speed values are isolated to the area near the entrance, and quickly reduce inside the Delaware Bay. Note, however, that fluid speeds near the entrance are extreme, with a large area experiencing speeds greater than 10 m/s (32.8 ft/s). As expected, the channel just offshore of the PSEG Site shows a local maximum in speeds. Here the water velocity reaches 5.9 m/s (19.4 ft/s). This large velocity is largely isolated to the Delaware Bay channel, and maximum speeds at the PSEG site are 3.3 m/s (10.8 ft/s).

For the 2D investigation, two simulations, each using a different bottom friction coefficient, show the range of possible tsunami elevations near the site. Each 2D simulation setup is identical, except for bottom friction coefficient. In one simulation, the bottom friction is set to zero at all initially submerged grid points. The other simulation imposes a Manning's *n* value of 0.025, corresponding to a smooth, natural bed, at all initially submerged grid points. This friction coefficient is a realistic, if not conservative, estimate for the continental shelf seafloor. Inside the Delaware River estuary, a Manning's *n* of 0.025 would be considered conservative, as published studies have found values of 0.03-0.04 more realistic (e.g., Ambrose and Roesch, 1982). For both 2D simulations, all initially dry locations use a Manning's *n* value of 0.025.

The 2D simulations predict a maximum tsunami elevation of 6.0 m (19.7 ft) with the no-friction simulation and 1.0 m (3.3 ft) from the with-friction simulation. The PMT is taken from the with-friction simulation, which still employs a conservative friction coefficient.

Canary Islands Source

The Canary Islands landslide source has initiated significant debate within the tsunami research community. The initial tsunami assessment by Ward and Day (Reference 2.4.6-2), due to a coherent failure of an entire island into the ocean, led to runup predictions of 10 to 25 m (32 ft to 82 ft) along nearly the entire east coast of the United States. Subsequent studies (Reference 2.4.6-3) have attempted to downplay the hazard, with reductions in runup by a factor of 10 for the most extreme case. In this study, the staff applies the most conservative published source values. Similar to the previous examinations, if this conservative setup has a damaging effect on the PSEG Site, the source parameters will be given additional scrutiny and unreasonable conservatism will be relaxed under the Hierarchical Hazards Approach (HHA) methodology.

The simulation approach for the Canary Island scenario utilizes three different simulation domains. The first will be the Atlantic Ocean domain (ocean domain), which is used to simulate the tsunami from its source to the continental shelf of the eastern United States. The output from the ocean domain is used to force a domain focused on the effects of the continental shelf break and the shallow shelf waters (shelf domain). The reason for this separation of offshore domains is due to the fact that important physical spatial scales in the open ocean are 1-10 km (0.62 – 6.2 mi), while on the shelf, where front steeping and breaking play a role, the relevant length scales are 10-100 m (32.8 – 328 ft). To accommodate this variability across two orders of magnitude, it is computationally most reasonable to tackle the problem with separated domains, executed independently. The third domain used for this tsunami scenario is the same nearshore domain as used with the Currituck scenario, which is forced with output from the shelf domain.

Following Reference 2.4.6-2, a coherent La Palma collapse will generate an initial wave with amplitude approaching 1,000 m (3,281 ft). For the simulations here, a hot start condition is placed just offshore of La Palma, with a crest elevation of 1,000 m (+3,281 ft) and a trough elevation of -1,000 m (-3,281 ft). The disturbance has a length of 50 km (31.1 mi) and a width of 25 km (15.5 mi), again taken approximately from the information in Reference 2.4.6-2. The wave propagation is modeled in the entire northern Atlantic Ocean in the ocean domain, using a grid length of 2 km (1.2 mi). The simulation is based on the fully nonlinear Boussinesq equations, with wave breaking included. Snapshots of the wave field 30 minutes after generation show the wave field spreading radially, almost as a point source, with the wave spreading rapidly both through radial spreading and frequency dispersion. In time, the tsunami has transformed into a long train with the longest frequencies at the lead; note that the largest crest does not in fact occur with the first wave. When reaching the continental shelf break along the eastern United States, the maximum crest elevation is less than 10 m (32.8 ft). The leading wave has a period of approximately 750 seconds which decreases to approximately 350 seconds near the back end of the train. The largest wave heights are located within this period range.

The 2-km (1.2-mi) grid used in the Atlantic Ocean simulation described above is not fine enough to resolve the shoaling and dissipation processes on the shallow continental shelf. Thus, to estimate the wave height at the entrance of the Delaware Bay from the Canary Islands tsunami, a second offshore simulation must be run, described above as the shelf domain. The wave disturbance as it approaches the shelf has little along-coast variability, and it is deemed that a 1HD, cross-sectional simulation will very reasonably capture the transformation of this wave train over the shelf break and across the shallow shelf. Snapshots of this 1HD offshore simulation at the shelf break show the largest of the waves shoaling to a great height, with crest elevations close 40 m (131.2 ft), and break immediately thereafter. These waves then form individual bore fronts which quickly travel across the shallow water shelf, decreasing in crest elevation as they approach the shoreline. The resulting disturbance has the form of a large number of 5-10 m (~16 – 33 ft) high bore fronts, one after the next, spaced 2-8 minutes apart. These bore fronts can become stacked on top of one another. This process, driven by amplitude dispersion, can lead to an amplified bore front if a trailing large bore overtakes and combines with a leading smaller, and slower traveling, bore.

The offshore forcing for the nearshore domain uses the identical numerical setup described in the Currituck scenario section. Due to the relatively short period of the individual pulses, compared to the Currituck wave, as well as the smaller incident crest elevations, less wave energy is able to travel far up the Delaware Bay. Similar to Currituck, the scattering of the wave at the entrance is the primary wave height reducer. The maximum recorded water surface

elevation and fluid speed at the PSEG Site for the Canary Islands tsunami is also smaller than that due to the Currituck event. For the Canary Islands tsunami, the maximum sea surface elevation is 6.1 m (+20 ft) NAVD88 (including high tide and sea level rise) and the maximum fluid speed is 2.3 m/s (7.5 ft/s) at the PSEG Site. Thus, despite the tremendous wave heights at the source region, by the time the wave has spread radially in the Atlantic, spread energy through frequency dispersion, dissipated due to breaking along the continental shelf, and traversed the geometrically irregular Bay, the tsunami elevation is reduced by orders of magnitude.

Puerto Rico Subduction Zone Source

The last source to be investigated for the PSEG Site is the subduction zone that borders much of the northeastern and eastern extent of the Caribbean Islands. Here, the staff assumes that the entire fault zone ruptures during a single earthquake event. Seafloor displacements are taken as the expected maximum values that this fault might generate. The initial sea surface condition is a direct mapping of the vertical seafloor displacement to the ocean surface. It is clear to the staff that the total rupture is composed of five individual regions; a simplification used to reasonably characterize the entire length. It is also evident to the staff that the largest waves will be directed toward the northeast Atlantic basin.

With a subduction zone earthquake, the generated waves are long in wavelength. The staff notes that this implies that the physics of the waves are simpler, relative to the dispersive waves created by the two landslide sources examined previously. To numerically model this source, the open-source tsunami model COMCOT (Cornell Multi-grid Coupled Tsunami Model) is used. A grid covering the entire western Atlantic is generated with a spatial grid size of 1 minute (1/60 of a degree latitude or longitude). A single grid layer is used; there is no nesting of domains for refinement. The time step used by the model is 1 second. The linear version of the model is used, and there is no bottom friction applied anywhere in the domain. The linear version of the model is deemed acceptable because, as will be shown, the wave height to water depth ratio is less than 0.1 at all areas of interest, and usually no greater than 0.01.

Once the wave exits the source area, the crest elevation of the main wave is about 2 m (6.6 ft) in the open ocean; Bermuda would experience an extreme and damaging wave. It is clear that the east coast of the United States, while certainty feeling effects from this source, would see relatively minor wave impact. By the time the wave has reached the continental shelf offshore of the Delaware Bay, the maximum crest elevation of the wave is approximately 1 m (3.3 ft). When the wave hits the shallow shelf, the wavelength shortens quickly, and the wave height increases.

Due to the small offshore height of the wave, compared to the two previously examined sources, it would not be expected that this wave would break and steepen into bore fronts near the shelf break. In this location, at the shelf break, the water depth is roughly 50 m (164 ft), while the wave height is approximately 3 m (9.8 ft), and the transformation processes will still be largely governed by linear shallow water physics. As the wave approaches the Delaware Bay entrance, shoaling amplification and refractive spreading approximately cancel, and the wave crest elevation entering the Delaware Bay is 1.5 m (4.9 ft). Compared to the near-Bay maximum crest elevation of 40 m (131.2 ft) for the Currituck source and 10 m (33 ft) for the Canary Islands source, the Puerto Rico subduction zone source is not likely to produce larger impacts at the PSEG Site. Thus, the water surface elevation at the PSEG Site is quite low, well below 0.25 m (0.82 ft).

2.4.6.4.5 Tsunami Water Levels

Information Submitted by the Applicant

In SSAR Sections 2.4.6.4.5 – 2.4.6.4.8, the applicant summarized the water-level predictions for each case study. For the Currituck landslide, the numerical predictions suggest the wave heights in Delaware Bay are not sensitive to the landslide location or width among the cases tested as long as the total landslide volume is the same. The applicant suggested that the former is owing to the fact that the offshore shelf bathymetry, rather than the source location, controls the wave height distribution and focusing patterns. The applicant indicated that the remaining numerical simulations use the historical landslide location.

The applicant indicated that the model predictions suggest that Delaware Bay filters out the high frequency components of the tsunami and that there is a region of high waves in the Delaware Bay entrance, but that this high wave energy does not extend into the bay itself. The applicant indicated that including bottom friction in the model reduces the magnitude of the predicted runup and drawdown. The water levels (Currituck landslide) associated with maximum runup and drawdown at the site are 1.72 m (+5.64 ft) NAVD88 and -1.88 m (-6.17 ft) NAVD88, respectively.

The applicant indicated that the La Palma event is simulated using an initial N-wave source input as a static initial condition. The applicant indicated that the wave has a dominant wave period of approximately 25 minutes and that the wave is filtered by the lower Delaware Bay. The applicant indicated that the water levels (La Palma event) associated with maximum runup and drawdown at the site are 1.45 m (+4.76 ft) NAVD88 and -1.62 m (-5.32 ft) NAVD88, respectively.

The applicant indicated that for the Hispaniola Trench subduction zone coseismic event, the tsunami source is based on a composite source consisting of seven fault segments with a total M_w of 9.0 and that the vertical displacement of each segment is calculated following Okada (1985). The applicant indicated that, similar to the other two case studies, the model predicts that refraction directs waves away from the Delaware Bay entrance, and that the bay effectively filters the high-frequency components of the tsunami. The applicant indicated that the water levels (Hispaniola Trench subduction zone coseismic event) associated with maximum runup and drawdown at the site are 1.59 m (+5.22 ft) and -1.69 m (-5.55 ft) NAVD88, respectively.

The applicant indicated that the PMT at the PSEG Site is generated by the Currituck landslide.

The Staff's Technical Evaluation

In RAI 20, Question 02.04.06-4, the staff requested that the applicant provide additional information, evaluation, and a discussion in the SSAR of the following items:

- Appropriateness of Shallow-Water Wave Models (SSAR Section 2.4.6.4.1). Reference to NUREG/CR-6966 and physics-based discussion on possible limitations of the MOST model for this application.

- Water Levels for Bottom Friction Experiment (SSAR Sections 2.4.6.4.1 and 2.4.6.4.5). Resolve the discrepancy between water levels shown in SSAR Figure 2.4.6-2 with the water levels stated in the last paragraph of SSAR Section 2.4.6.4.5. Reference to section presenting 10 percent exceedance tidal levels, and repeat tidal values when presenting runup/rundown in SSAR Section 2.4.6.4.5.

- <u>Input Parameters and Results for All Water Level Models (SSAR Section 2.4.6.2)</u>. Provide images of initial conditions and snapshots of the wave field in time in a revised version of the SSAR.

- <u>Determination of Simulation Time (SSAR Section 2.4.6.4.4)</u>. Provide information in the updated SSAR that shows that the results of a long-time Currituck landslide simulation, out to 40 hours of real elapsed time, show no evidence of a seiche.

- <u>Sensitivity Experiments for Atlantic Margin Landslides (SSAR Section 2.4.6.4.5, 2nd Paragraph)</u>. Provide information regarding whether the other locations of the landslides used in the sensitivity experiments are in a geologically similar environment compared to the actual Currituck landslide.

- <u>Landslide Initial Conditions (SSAR 2.4.6.4.5 and 2.4.6.4.6)</u>. Provide a discussion of conservativeness of the TOPICS method of determining initial conditions for the Currituck landslide and the N-wave for the Canary Islands. Provide all input parameters.

- <u>Effective Filtering of Delaware Bay (SSAR Section 2.4.6.4.5, 3rd Paragraph, SSAR Section 2.4.6.4.6, 1st Paragraph, and SSAR Section 2.4.6.4.7, 3rd Paragraph)</u>. Provide additional simulation results for a case or cases with a finer resolution, to test the numerical effect of high frequency filtering and to ensure that the model is not unrealistically damping these components.

- <u>Hispaniola Earthquake Source Parameters (SSAR Section 2.4.6.4.7, 2nd Paragraph)</u>. Provide a discussion on how the source parameters are derived.

In a May 11, 2011, response to RAI 20, Question 02.04.06-4, the applicant provided the requested information and committed to provide the following revisions:

- SSAR Section 2.4.6.4.8: line 6, first paragraph will be revised to add the negative sign in front of -5.08 ft NAVD**.**

- Time series figures will be added for each of the model runs. These figures will be referenced in SSAR Sections 2.4.6.4.5, 2.4.6.4.6, and 2.4.6.4.7.

- A sentence will be added to end of SSAR Section 2.4.6.4.4 and a figure showing seiche effects (Figure 2.4.6-7) will be added.

- SSAR Section 2.4.6.4.5, second paragraph will be revised and a third paragraph added to better describe sensitivity experiments for Atlantic Margin Slides.

- SSAR Section 2.4.6.3.1 will be revised to describe landslide initial conditions for Currituck and SSAR Section 2.4.6.3.2 will be revised to describe N-wave source for Canary.

The staff verified that Revision 1 of the ESP application (May 21, 2012) contains the applicant's committed changes. Therefore, the staff considers RAI 20, Question 02.4.06-4 resolved.

The staff performed numerical modeling of three different tsunami sources to determine their impact on the PSEG Site. The three sources are a near field landslide source along the continental shelf break (the Currituck source), a far field landslide source with extremely large local waves (the Canary Islands source), and a far field earthquake source (the Puerto Rico

Subduction Zone source). For all conditions, the most conservative source parameters were employed, even when arguably unphysical, to provide an absolute upper limit on the possible tsunami effects at the PSEG Site. The local (Currituck) landslide source proved to have the largest impact at the PSEG Site, with maximum 1HD water surface elevations due to the tsunami of 8.6 m (+28.2 ft) NAVD88 and maximum fluid speeds of 3.3 m/s (10.8 ft/s). Note that these elevations assume that the tsunami occurs at high tide (1.68 m (5.51 ft)) above Mean Low Water), with an additional depth of 0.75 m (2.46 ft) added for sea level rise. The Canary Islands source, despite generating sea surface elevation of 1 km (.62 mi) at the source, leads to a 1HD tsunami crest elevation of 4.8 m (+15.8 ft) NAVD88 near the PSEG Site. The earthquake source has by far the smallest effect on the site, with maximum 1HD water surface elevations less than 0.25 m (0.82 ft). Thus the local Currituck-like landslide source is the PMT. However, the effects of the PMT are below that of the DBF of 9.78 m (32.1 ft).

2.4.6.4.6 Hydrography and Harbor or Breakwater Influences on Tsunami

Information Submitted by the Applicant

The Delaware River in the vicinity of the Site does not contain any harbors or breakwaters. Information on bathymetry and topography in the Site vicinity is provided in SSAR Section 2.4.6.4.3.

The Staff's Technical Evaluation

Based on the staff evaluation of the applicant's numerical simulations provided in SSAR Section 2.4.6.4.5, the staff concurs that the bathymetry of the Delaware Bay is adequately included in the tsunami propagation computations (See Section 2.4.6.4.4 for details).

2.4.6.4.7 Effects on Safety-Related Facilities

Information Submitted by the Applicant

In SSAR Section 2.4.6.5, the applicant indicated that the new plant grade will be established at an elevation of 11.25 m (36.9 ft) NAVD88, and that none of the maximum predicted runup elevations obtained in this study overtop this elevation. The applicant indicated that the PMT will not constitute a limiting design basis for the new plant.

The Staff's Technical Evaluation

The staff concurs that since the maximum tsunami water level associated with the PMT is below grade elevations at the site, there will be no onsite tsunami waves affecting safety-related facilities. Minimum low water levels associated with the PMT do not define the design basis for the safety-related ultimate heat sink (UHS) water intake structure.

2.4.6.4.8 Hydrostatic and Hydrodynamic Forces

Information Submitted by the Applicant

In SSAR Section 2.4.6.5, the applicant stated that hydrodynamic and hydrostatic forces will not impact any safety-related structures.

For the safety-related SSCs, the applicant indicated that the hydrostatic and hydrodynamic design bases are controlled by the PMSS and not by the PMT.

The Staff's Technical Evaluation

The staff concurs that the PMT does not define the hydrostatic and hydrodynamic design basis.

2.4.6.4.9 Debris and Water-Borne Projectiles

Information Submitted by the Applicant

In SSAR Section 2.4.6.6, the applicant indicated that as the grade elevation of the plant will not be flooded by the PMT, debris and waterborne projectiles will not come into contact with any safety-related structures. The applicant further indicated that the intake structure at the new plant will be designed to protect it from impacts of waves and waterborne projectiles.

The Staff's Technical Evaluation

The staff concurs that the grade elevation of the plant will not be flooded by the PMT. The intake design and details on impacts of waves and waterborne projectiles will be provided in the COL phase.

2.4.6.4.10 Effect of Sediment Erosion and Deposition

Information Submitted by the Applicant

In SSAR Section 2.4.6.7, the applicant acknowledged that strong water currents associated with tsunamis can cause erosion and deposition. However, the applicant indicates that the current speeds predicted near the site fall within the range of normal tidal current activity in the Delaware Bay and, therefore, a rapid morphologic response to tsunami activity at the site is not expected.

The Staff's Technical Evaluation

Using the staff 1HD tsunami analysis for the Currituck landslide, the results show that the channel just offshore of the EPS site has a local maximum in current velocity of 5.9 m/s (19.4 ft/s). However, this large current velocity is largely isolated to the Bay channel, and maximum speeds at the PSEG site reduce to 3.3 m/s (10.8 ft/s). Although the staff did not calculate current velocity for the 2D analysis, the 2D analysis shows approximately a 45 percent reduction in tsunami amplitude. A corresponding reduction in current velocity would result in a current velocity of 1.8 m/s (5.9 ft/s). The nominal tidal current in Delaware Bay can reach or exceed velocities around 1.0 m/s (3.3 ft/s). Thus, the staff agrees with the applicant that tsunami current velocities would create sediment and erosion within the range of normal tidal activity.

2.4.6.4.11 Consideration of Other Site-Related Evaluation Criteria

Information Submitted by the Applicant

In SSAR Section 2.4.6.8, the applicant indicated that of the three tsunami sources examined, two (the Currituck and La Palma landslides) are not necessarily tied to strong seismic activity. The applicant indicated that only the Hispaniola Trench source is associated with seismic activity, but is located 2,494 km (1,550 mi) away. For these reasons, the applicant stated that a combined tsunami and seismic event was not considered in designing safety-related SSCs for the plant.

The Staff's Technical Evaluation

The staff concurs that the PMT sources will not be combined with the design-basis earthquake when evaluating the design of safety-related SSCs.

2.4.6.5 *Post Early Site Permit Activities*

There are no post ESP activities related to this section.

2.4.6.6 *Conclusions*

The staff reviewed the applicant's submittals in SSAR Section 2.4.6 and in response to the RAIs. As set forth above, the applicant presented and substantiated sufficient information pertaining to estimates of the effects from probable maximum tsunami hazards at the proposed PSEG Site, and no outstanding information is required to be addressed in the SSAR for this section. Furthermore, the staff finds that the applicant considered the most severe natural phenomena that have been historically reported for the site and surrounding area while describing the probable maximum tsunami hazards, with a sufficient margin for the limited accuracy, quantity, and period of time in which the historical data were accumulated.

The staff accepted the methodologies used by the applicant to determine the severity of the tsunami phenomena reflected in this analysis, as documented in this section of the report. In the context of the above discussion, the staff finds the applicant's analysis acceptable for use in establishing the design bases for SSCs important to safety. Accordingly, the staff concludes that the use of these methodologies results in an analysis containing a sufficient margin for the limited accuracy, quantity, and period of time in which the data were accumulated. Additionally, the 1HD PMT flood level 8.6 m (+28.2 ft) NAVD88 and 2D PMT flood level 1.0 m (+3.3 ft) estimated by the staff are below the bounding 2D PMSS water level of 9.78 m (32.1 ft) NAVD88 as well as the plant grade of 11.25 m (36.9 ft) NAVD88. The applicant provided a more conservative 2D PMT flood level of 1.72 m (+5.65 ft) NAVD88 which is also below the 2D PMSS and PSEG Site grade. Therefore, the staff concludes that the postulated PMT would not affect the proposed PSEG Site. Therefore, the staff finds the identification and consideration of the PMT hazards set forth above acceptable and meet the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR 100.20(c), and 10 CFR 100.23(d).

2.4.7 Ice Effects

2.4.7.1 *Introduction*

SSAR Section 2.4.7 addresses ice effects to ensure that safety-related facilities and water supply are not affected by ice-induced hazards.

The ice effects are addressed to ensure that safety-related facilities and water supply are not affected by ice-induced hazards. The specific areas of review are as follows: (1) regional history and types of historical ice accumulations (e.g., ice jams, wind-driven ice ridges, floes, frazil ice formation); (2) potential effects of ice-induced, high- or low-flow levels on safety-related facilities and water supplies; (3) potential effects of a surface ice sheet to reduce the volume of available liquid water in safety-related water reservoirs; (4) potential effects of ice to produce forces on, or cause blockage of, safety-related facilities; (5) potential effects of seismic and nonseismic data on the postulated worst-case icing scenario for the proposed plant site; (6) any

additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.7.2 *Summary of Application*

In this section, potential ice effects at the proposed plant location are evaluated, including the review of ice formations or ice jams; modeling combined events to ensure protection of the safety-related facilities from ice-affected floods, and mitigation to protect safety-related structures from ice. Analysis of ice effects at the proposed plant includes review of historic winter conditions and the simulation of flooding due to an upstream ice jam break.

2.4.7.3 *Regulatory Basis*

The relevant requirements of the NRC regulations for identifying ice effects and the associated acceptance criteria, are in NUREG-0800, Section 2.4.7.

The applicable regulatory requirements for identifying ice effects are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirements to consider physical site characteristics in site evaluations are specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used the appropriate sections of the following regulatory guides for the acceptance criteria identified in NUREG-0800, Section 2.4.3:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed.
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding.
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.7.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.7. The staff's review confirmed that the information in the application addresses the relevant information related to the site ice effects.

The staff's technical review of this section includes an independent review of the applicant's information in the SSAR, Revision 2.

The applicant modeled flooding caused by an upstream ice jam utilizing the historical record of surface water elevations, instantaneous failure of a historic upstream ice jam, 10 percent exceedance high tide, averaged spring base flows and wave runup resulting from the maximum 2-year wind in the critical direction to obtain peak surface water level elevations at the site. Additionally, low water levels were considered as a result of upstream river blockage from an ice jam. The staff independently assessed the potential for formation of ice at the PSEG Site using available data. This section of the report provides the staff's evaluation of the technical information presented in SSAR Section 2.4.7.

2.4.7.4.1 Historical Ice Accumulation

Information Submitted by the Applicant

Temperature records from 1894 to 2009 (Reference 2.4.7-1) were reviewed to determine the minimum temperature for the analysis. The applicant used the lowest temperature (-26 °Celsius (C) (-15 °Fahrenheit (F))) on record for Wilmington, DE for the analysis rather than the value ((-21 °C (-6 °F)) available for the 32 year record at the site. Historically, surface ice has been observed at the PSEG Site during January and February, and conditions (i.e., air temperatures at or below -6 °C (21 °F), super cooled water below freezing, open water and clear nights), amenable for potential frazil ice formation may occur at the site.

The applicant reviewed the USACE CRREL Ice Jam Database and found no recorded ice jams on the Delaware River downstream of Trenton (RM 134) for the period of record (1780 through 2009). In combination with rapid snow melt, an ice jam on the Delaware River closest to the PSEG Site (RM 52) at Trenton (RM 134) during 1904 caused the highest recorded ice jam induced flooding on the river producing a maximum gauge height of 9.02 m (29.6 ft) NAVD88 at the Trenton, NJ, USGS gauge 01463500.

The Staff's Technical Evaluation

The staff reviewed historical temperature records and found the applicant's characterization to be reasonable and adequate for representation of potential surface and frazil ice formation. Although ice jam flooding has occurred approximately 132 km (82 mi) upstream of the PSEG Site, the staff reviewed the CRREL Ice Jam Database and confirmed that no record of ice jam flooding has occurred downstream of the site. Although the CRREL database lists an 1857 ice jam flooding event at Trenton that may have had a stage equal to or exceeding the 1904 event cited by the applicant, the record for the 1857 event lacks a comparable datum to the USGS gauge (the USGS was established in 1879). The staff finds the applicant's review and characterization of the historical record adequate.

2.4.7.4.2 High Water Levels

Information Submitted by the Applicant

To estimate high water levels the applicant used the HEC-RAS (Reference 2.4.7-2) model to simulate an instantaneous breach of the 1904 ice jam event at Trenton, NJ. As an estimate of worst case conditions with the event, the applicant combined a 10 percent exceedance high tide, mean spring monthly discharge for the period of record (1913 through 2008) as a base flow, and 2-year wind effects of wave runup. Terrain models were based on the USGS digital

elevation models and NOAA Estuarine Bathymetry Data. Manning's *n* coefficient for the HEC-RAS modeling was set to 0.025 for non-tidal portions of the Delaware River and 0.05 for the flood plain areas. Discharge from the individual drainage areas developed in HEC-HMS (Reference 2.4.7-3) defined the inputs to the HEC-RAS modeling.

The 10 percent exceedance high tide (1.37 m (4.5 ft)) was based on linear interpolation to the PSEG Site of the tides between Lewes NOAA gauge (RM 0) and Reedy Point (RM 59). The highest monthly mean discharge for the period of record (1913 to 2008) was applied to the 10 percent exceedance high tide. The application of 10 percent exceedance high tide and the highest monthly mean discharge resulted in a maximum water surface elevation of 1.58 m (5.2 ft) NAVD88.

The ice jam at Trenton was assumed to instantaneously breach with a 2-year wind speed applied on the resulting water level in the critical direction to determine coincident wave runup. The ice jam flooding resulted in a 0.03 m (0.1 ft) increase in surface water level at the site. A 2-year annual extreme wind speed of 50 mph determined to be consistent with ANSI/ANS-2.8-1992 (Reference 2.4.7-4) was adjusted for fetch and duration limits in accordance with the Coastal Engineering Manual (Reference 2.4.7-5). Based on the analysis, a maximum wave height of 1.7 m (5.6 ft) was determined with a runup of 0.85 m (2.8 ft) based on the Coastal Engineering Manual methods.

The applicant determined that the resulting water level at the site would be 2.47 m (8.1 ft) NAVD88 based on the sum of 10 percent exceedance high tide (1.37 m (4.5 ft)), spring base flows (0.21 m (0.7 ft)), Trenton ice jam (0.03 m (0.1 ft)), and the coincident wave runup from a 2-year wind speed in the critical direction (0.85 m (2.8 ft)).

The Staff's Technical Evaluation

Using discharge for the individual drainage areas generated by HEC-HMS (Reference 2.4.7-3), the applicant applied a HEC-RAS (Reference 2.4.7-2) surface water model to simulate the largest ice jam determined from the historical record in combination with a wind event in the critical direction similar to that applied to other types of flooding in the SSAR, Revision 2. Given the dominating tidal influence on the Delaware River adjacent to the site and the wide and open connection of the Delaware River to the Atlantic Ocean, ice jams upstream of the site are extremely unlikely to impact safety-related SSCs at the PSEG Site. The applicant's simulation of a major historic ice jam event is adequate and results in a flood level of 2.47 m (8.1 ft) NAVD88, which is below the design basis flood (9.78 m (32.1 ft) NAVD88).

Based on the staff's review of the physiography of the site location, the staff's review of the CRREL ice jam database, and the applicant's reasonable application of a conservative upstream ice jam analysis, the staff concludes that ice jams would have no high water safety-related impacts to the water supply intake or the water supply for the PSEG Site. The staff finds the applicant's analysis adequate.

2.4.7.4.3 Low Water Levels

Information Submitted by the Applicant

The ice jam low water condition and resulting effects are evaluated in SSAR Section 2.4.11.

The Staff's Technical Evaluation

Given the dominating tidal influence on the Delaware River adjacent to the site and the wide and open connection of the Delaware River to the Atlantic Ocean, ice jams upstream of the site are extremely unlikely to impact safety-related SSCs at the PSEG Site. The staff evaluated the applicant's assessment of low water levels in SSAR Section 2.4.11.

2.4.7.4.4 Ice Sheet Formation

Information Submitted by the Applicant

The applicant reviewed and summarized the historical record from the National Ice Center (Reference 2.4.7-6) and found that sheet ice that has formed in the mid and upper portions of the Delaware Bay was not concentrated enough to be considered fast ice or ice that is anchored to the shore. The applicant summarized the thickness and concentration of the ice reported in the Delaware River transition zone to the Delaware Bay that is adjacent to the PSEG Site. The thickest portion of ice adjacent to the PSEG Site was estimated to be 12 to 28 in. for mature areas of ice and 0 to 4 in. for newly formed areas of ice.

The applicant stated that protective measures for the intake structure will be in accordance with ANSI/ANS-2.8-1992, Section 8.3, "Surface Ice," to mitigate potential effects from frazil ice, surface ice, and other dynamic forces associated with ice effects.

The Staff's Technical Evaluation

The staff reviewed the surface ice of record formed at the PSEG Site and agreed that the surface ice is neither continuous nor does it reduce access to available water for safety-related cooling. Since tidal effects dominate the flow adjacent to the PSEG Site, the water volume forming the surface ice is negligible in comparison to the tidally induced flows and the volume of the Delaware Bay. Therefore, there is no potential for surface ice to reduce the volume of water available for safety-related cooling. The applicant stated that protective measures in accordance with ANSI/ANS-2.8-1992 will be implemented to mitigate the potential effects of frazil ice, surface ice and other dynamic forces associated with ice on the intake structure. Since there is no potential to reduce safety-related cooling water from surface ice, the staff finds the applicant's approach to implement protective measures, as called out in ANSI/ANS-2.8-1992, adequate.

2.4.7.4.5 Potential Ice-Induced Forces and Blockages

Information Submitted by the Applicant

The applicant reviewed the tri-agency (U.S. Navy/NOAA/U.S. Coast Guard), National Ice Center data and noted that ice formed in the mid and upper portions of the Delaware River Bay was concentrated enough to form a solid sheet but not considered to be anchored to the shoreline. No ice blockages have occurred downstream of the PSEG Site based on the historical record. The potential formation of frazil ice was determined by the applicant using USACE CRREL design procedures (Reference 2.4.7-7). The applicant noted that the proposed plant is located in a tidal transition zone of the Delaware River and that the icing events depicted in this section represent worst case scenarios adjacent to the PSEG Site. The applicant stated that the intake structure at the new plant will be designed with protective measures in accordance with ANSI/ANS-2.8-1992 to mitigate the potential effects of frazil ice, surface ice and other dynamic forces associated with ice.

The Staff's Technical Evaluation

The staff reviewed the applicant's evaluation of ice effects including the National Ice Center data and the applicant's analyses of frazil ice formation to determine the depth of frazil ice formation. This review included verification of historical reports of ice dams along with river stage data and discharge data downstream as available. At the PSEG Site location, the Delaware River is tidally influenced, 4.0 km (2.5 mi) wide, and progressively widens to 16.1 km (10 mi) at the entrance to the Delaware Bay with a lack of constricting terrain making the formation of a surface ice blockage extremely unlikely consistent with the lack of recorded ice jams downstream of the site. Additionally, after a reactor technology is selected, protective measures in accordance with ANSI/ANS-2.8-1992, Section 8.3, "Surface Ice," for an intake structure are acceptable to the staff for mitigation of frazil ice formation. Therefore, the staff finds the information and evaluation provided in the application adequate.

2.4.7.5 *Post Early Site Permit Activities*

There are no post ESP activities related to this section.

2.4.7.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the plant parameter envelope, and that there is no outstanding information required to be addressed in the SSAR related to this section.

As set forth above, the applicant has provided sufficient information pertaining to ice effects. Therefore, the staff concludes that the applicant has met the requirements concerning ice effects with respect to 10 CFR 52.17(a) and 10 CFR Part 100. Further, the applicant has considered the most severe natural phenomena that have been historically reported for the site and surrounding area with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated, in establishing site characteristics pertaining to ice effects that are acceptable for design purposes.

2.4.8 Cooling Water Canals and Reservoirs

2.4.8.1 *Introduction*

The cooling water canals and reservoirs used to transport and impound water supplied to the SSCs important to safety are reviewed to verify their hydraulic design basis. The specific areas of review are as follows: (1) design bases postulated and used by the applicant to protect structures such as riprap, inasmuch as they apply to safety-related water supply; (2) design bases of canals pertaining to capacity, protection against wind waves, erosion, sedimentation, and freeboard and the ability to withstand a PMF (surges, etc.), inasmuch as they apply to a safety-related water supply; (3) design bases of reservoirs pertaining to capacity, PMF design basis, wind wave and run-up protection, discharge facilities (e.g., low-level outlet, spillways), outlet protection, freeboard, and erosion and sedimentation processes inasmuch as they apply to a safety-related water supply; (4) potential effects of seismic and nonseismic information on the postulated hydraulic design bases of canals and reservoirs for the proposed plant site.

2.4.8.2 *Summary of Application*

This section of the SSAR addresses the cooling-water canals and reservoirs used to transport and impound water supplied to the safety-related SSCs. This section of the report presents an evaluation of the design basis for the capacity and operating plan for safety-related cooling-water canals and reservoirs, and any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.8.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the cooling-water canals and reservoirs, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.8.

The applicable regulatory requirements for describing cooling-water canals and reservoirs are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

    - 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used the appropriate sections of the following regulatory guides for the acceptance criteria identified in NUREG-0800, Section 2.4.8:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed.

- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding.

- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.

- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.8.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.8. The staff confirmed that the information in the application addresses the relevant information related to the site cooling water canals and reservoirs. The staff's technical review of this section included an independent

review of the applicant's information in the SSAR. The staff supplemented this information with other publicly available sources of data. The staff's technical review of this section described below includes an independent review of the applicant's information provided in the SSAR.

Information Submitted by the Applicant

The proposed PPE does not include any safety-related canals or reservoirs used to transport or impound plant cooling water. Makeup to the safety-related UHS system and the non-safety-related cooling water system for the new plant is provided by an intake structure located on the east bank of the Delaware River, north of the existing HCGS service water intake structure. As the reactor technology for the new plant has not been chosen, the specific design of the intake structure is not finalized. The intake structure will be set at an elevation low enough that it can provide an uninterrupted supply of water to the proposed plant, even under extreme low water conditions.

The Staff's Technical Evaluation

The staff reviewed SSAR Section 2.4.8. The staff confirmed that the information in the application addresses the relevant information related to this section and is sufficient and appropriate. The staff concludes that because there are no safety-related reservoirs or canals proposed for the PPE design, Section 2.4.8 is not applicable to the PSEG Site.

2.4.8.5 *Post Early Site Permit Activities*

There are no post ESP activities related to this section.

2.4.8.6 *Conclusion*

The staff reviewed the application and confirmed that there are no safety-related reservoirs or canals proposed for the new plant in the plan parameter envelope. There is no outstanding information required to be addressed in the SSAR related to this section.

2.4.9 Channel Diversions

2.4.9.1 *Introduction*

This section of the SSAR addresses channel diversions. It evaluates plant and essential water supplies used to transport and impound water supplies to ensure that they will not be adversely affected by stream or channel diversions. The evaluation includes stream channel diversions away from the site (which may lead to a loss of safety-related water) and stream channel diversions toward the site (which may lead to flooding). In addition, in such an event, it must be ensured that alternate water supplies are available to safety-related equipment.

This section of the report presents an evaluation of the following specific areas: (1) historical channel migration phenomena including cutoffs, subsidence, and uplift; (2) regional topographic evidence that suggests a future channel diversion may or may not occur (used in conjunction with evidence of historical diversions); (3) thermal causes of channel diversion, such as ice jams, which may result from downstream ice blockages that may lead to flooding from backwater or upstream ice blockages that can divert the flow of water away from the intake; (4) potential for forces on safety-related facilities or the blockage of water supplies resulting from channel migration-induced flooding (flooding not addressed by hydrometeorologically induced

flooding scenarios in other sections); (5) potential of channel diversion from human-induced causes (i.e., land-use changes, diking, channelization, armoring, or failure of structures); (6) alternate water sources and operating procedures; (7) potential effects of seismic and nonseismic information on the postulated worst-case channel diversion scenario for the proposed plant site; (8) any additional information requirement prescribed in the “Contents of Application” sections of the applicable subparts of 10 CFR Part 52.

2.4.9.2 *Summary of Application*

In SSAR Section 2.4.9, the applicant described site-specific information related to the channel diversions.

2.4.9.3 *Regulatory Basis*

The relevant requirements of NRC regulations for channel diversions, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.9, and “Channel Diversions.”

The applicable regulatory requirements for identifying and evaluating channel diversions are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used the appropriate sections of the following regulatory guides for the acceptance criteria identified in NUREG-0800, Section 2.4.9.

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants,” as it relates to providing high assurance that the water sources relied on for the sink will be available where needed.

- RG 1.29, “Seismic Design Classification,” as it relates to those SSCs intended to protect against the effects of flooding.

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.

- RG 1.102, “Flood Protection for Nuclear Power Plants,” as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.9.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.9. The staff's review confirmed that the information in the application addresses the relevant information related to the channel diversions. The staff's technical review of this section includes an independent review of the applicant's information in the SSAR. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented by the applicant in SSAR Section 2.4.9.

2.4.9.4.1 Historical Channel Diversions

Information Submitted by the Applicant

Based on past studies (Reference 2.4.9-1), the applicant indicated that the Delaware River has been flowing in its current channel for approximately 10,000 years. There are no levees or dams on the Delaware River and collapse or breaching of levees or dams on tributaries will have little impact on surface water levels at the PSEG Site as discussed in SSAR Section 2.4.4.

The Staff's Technical Evaluation

The ancestral Delaware River followed a similar course to that of the present day for the last several thousand years (Reference 2.4.9-1), although undoubtedly not always precisely in its current channel. Given the low topographic relief, wide and open marine tidal connection of the Delaware River adjacent to the site, and lack of constricting topography from the PSEG Site to the mouth of the Delaware Bay, dam breaching or levee collapses along tributaries to the Delaware River would not significantly impact Delaware River water levels nor be reasonably expected to impact safety-related SSCs at the PSEG Site. The staff considers the applicant's evaluation adequate.

2.4.9.4.2 Regional Topographic Evidence

Information Submitted by the Applicant

The PSEG site is located in a region of relatively low relief in the Atlantic Coastal plain with highest land surface elevations (approximately 6.1 m (20 ft) NAVD88) in the vicinity of the site corresponding to man-made embankments along the Delaware River. The river is approximately 4.0 km (2.5 mi) wide at the PSEG Site and progressively widens to several kilometers (miles) at the mouth of the Delaware Bay. Given the low topographic relief and lack of constricting topography from the PSEG Site to the mouth of the bay, a blockage downstream causing a channel diversion that could affect the site SSCs designed to the selected DCD specifications is extremely unlikely.

The Staff's Technical Evaluation

Given the low topographic relief, wide and open tidal connection of the Delaware River adjacent to the site, and lack of constricting topography from the PSEG Site to the mouth of the Delaware Bay, topographic characteristics would not be amenable to a downstream blockage that would create any significant flooding at the PSEG Site. The staff finds the applicant's evaluation and information adequate.

2.4.9.4.3 Ice Causes

Information Submitted by the Applicant

Ice blockages are discussed in SSAR Section 2.4.7. Given the wide and open marine connection of the Delaware River to tidal influences, tidal flow could easily supply sufficient cooling water for the proposed plant with upstream river ice blockages. The upstream river ice blockages would not be a threat to site SSCs.

The Staff's Technical Evaluation

The staff reviewed SSAR Section 2.4.7 and the physiographic nature of the tidally influenced Delaware River. Tidal flow at the PSEG Site ranges from 11,327 to 13,366 m^3 (400,000 to 472,000 ft^3) per second (Reference 2.4.9-2 and Reference 2.4.9-3), which is sufficient to supply the required water (approximately 5 m^3 (177 ft^3) per second for the PPE cooling. Therefore, ice blockages causing channel diversions upstream of the PSEG Site would not limit the volume of safety-related water available for cooling. The staff finds the applicant's evaluation adequate.

2.4.9.4.4 Flooding of Site Due to Channel Diversions

Information Submitted by the Applicant

Physiographic characteristics of the PSEG Site and surrounding areas, and the tidal nature of the Delaware River make flooding due to channel diversions extremely unlikely at the PSEG Site. In addition, the tidal nature of the river at the PSEG Site results in ample cooling water availability for the plant.

The applicant indicated that even if (as yet un-designed) drainage ditches at a proposed plant are blocked due to ice formation, blockages will be bypassed as the water rises. Grading in the vicinity of the SSCs will be sloped away from each of the SSCs toward collection ditches. The grading plan will be adapted and designed for the specific technology selected at the COL application stage.

The Staff's Technical Evaluation

The PSEG Site is located in a tidal zone progressively widening into the Delaware Bay, which empties into the Atlantic. Given the wide and open connection with the Atlantic, there are no opportunities for channel diversions to impact the site as flood waters from channel diversions would flow unimpeded to the Atlantic.

Since the applicant has not selected a reactor technology, a site grading plan and storm water management system necessary to establish the maximum site water surface elevation due to collection ditch capacities and blockage has not been determined. A detailed analysis to determine the maximum site water level will be performed once a specific reactor technology is selected at the COL stage. While the staff finds the applicant's evaluation adequate, the staff is tracking the maximum site water level determination need via **COL Action Item 2.4-1** (See Section 2.4.2.4.3 of this report).

2.4.9.4.5 Human-Induced Causes of Channel Diversion

Information Submitted by the Applicant

The Delaware River in the vicinity of the PSEG Site is actively maintained with dredging by the USACE as an established shipping channel. This regular maintenance, coupled with shoreline protection (e.g., river bank armoring) and water resource oversight by the DRBC, reduces the potential for anthropogenic-induced diversions of the Delaware River channel.

The Staff's Technical Evaluation

The Delaware River is an established major navigable waterway that is actively maintained by the USACE with protection and development of the Delaware River Basin water resources within the purview of the DRBC. Given the USACE maintenance and DRBC regulatory oversight, human-induced modifications are carefully monitored and unlikely to threaten the PSEG Site. The staff finds the applicant's evaluation adequate.

2.4.9.4.6 Alternative Water Sources

Information Submitted by the Applicant

The Delaware River safety-related water supply to the proposed plant consists primarily of tidal flow with much lesser contributions from freshwater flow of upstream tributaries. Historically, there are no recorded channel diversions of the Delaware River. Average annual freshwater flow at the Trenton, NJ gauge is 334 m^3 (11,780 ft^3) per second, while the proposed PPE intake is projected as 5 m^3 (175 ft^3) per second. Tidal flow at the PSEG Site ranges from 11,327 to 13,366 m^3 (400,000 to 472,000 ft^3) per second (Reference 2.4.9-2 and Reference 2.4.9-3), which is more than sufficient for the required water supply.

The Staff's Technical Evaluation

The staff reviewed the physiographic characteristics of the PSEG Site area and publicly available tidal flow studies published by the USGS (e.g., Reference 2.4.9-4). The staff finds these characteristics and tidal flow rates to be consistent with those cited by the applicant and therefore, adequate and acceptable.

2.4.9.4.7 Consideration of Other Site-Related Evaluation Criteria

Information Submitted by the Applicant

Channel diversion from severe weather events (SSAR Sections 2.4.2 through 2.4.7) or seismic events is not considered to contribute to a loss of the proposed plant's cooling water supply. The wide Delaware River, low topography and gentle relief in the vicinity of the PSEG Site preclude impacts to SSCs from shoreline collapse due to seismic or severe weather events. The intake forebay will extend into the Delaware River and be dredged to an elevation sufficient to accommodate extreme low water elevation in the river. Periodic maintenance will be performed to remove accumulated silt and sedimentation to maintain the specified invert elevation.

The Staff's Technical Evaluation

The staff reviewed potential impacts from severe weather events (Sections 2.4.2 through 2.4.7 herein), and the potential for seismic events to contribute to a loss of the proposed plant's

cooling water supply. Seismic-induced collapse of already low-lying landforms within the subdued topography in the vicinity of the plant would have no impacts to safety-related SSCs. The applicant will maintain the intake structure to accommodate low water and will perform sediment removal for a specified invert elevation. The staff finds the applicant's evaluation to be adequate.

Based on a review of the applicant's information in the SSAR, the staff finds that the applicant appropriately considered channel-diverting phenomena and their combinations that are relevant for the PSEG Site. Therefore, the staff finds that the requirements of 10 CFR 52.17(a)(1)(vi), 10 CFR Part 100, and 10 CFR 100.23(d), as they relate to identifying and evaluating hydrological features of the site, are met.

2.4.9.5 *Post Early Site Permit Activities*

There are no post ESP activities related to this section.

2.4.9.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the plant parameter envelope, and that there is no outstanding information required to be addressed in the SSAR related to this section.

As set forth above, the applicant has provided information pertaining to channel diversions showing that channel diversion above the PSEG Site is not likely. Therefore, the staff concludes that the applicant has met the requirements regarding channel diversions, with respect to 10 CFR 52.17(a), 10 CFR Part 100. Additionally, the staff concludes that the applicant has considered the most severe natural phenomena that have been historically reported for the site and surrounding area with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated in establishing that channel diversion is not likely at this site.

2.4.10 Flooding Protection Requirements

2.4.10.1 *Introduction*

The flooding protection requirements address the locations and elevations of safety-related facilities and those of structures and components required for protection of safety-related facilities. These requirements are then compared with design-basis flood conditions to determine whether flood effects need to be considered in the plant's design or in emergency procedures. The specific areas of review are as follows: (1) safety-related facilities exposed to flooding; (2) type of flood protection (e.g., "hardened facilities," sandbags, flood doors, bulkheads) provided to the SSCs exposed to floods; (3) emergency procedures needed to implement flood protection activities and warning times available for their implementation reviewed by the organization responsible for reviewing issues related to plant emergency procedures; (4) potential effects of seismic and nonseismic information on the postulated flooding protection for the proposed plant site; and (5) any additional information requirements prescribed in the applicable subparts to 10 CFR Part 52.

2.4.10.2 *Summary of Application*

In SSAR Section 2.4.10, the applicant addressed the need for site-specific information on flood protection requirements.

2.4.10.3 *Regulatory Basis*

The relevant requirements of NRC regulations and the associated acceptance criteria for flood protection are specified in NUREG-0800, Section 2.4.10, “Flooding Protection Requirements.”

The applicable regulatory requirements for identifying and evaluating flood protection are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used the appropriate sections of the following regulatory guides for the acceptance criteria identified in NUREG-0800, Section 2.4.9.

- RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants,” as it relates to providing high assurance that the water sources relied on for the sink will be available where needed

- RG 1.29, “Seismic Design Classification,” as it relates to those SSCs intended to protect against the effects of flooding.

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.

- RG 1.102, “Flood Protection for Nuclear Power Plants,” as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.10.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.10 and confirmed that the information in the application addresses the relevant information related to the flooding protection requirements. The staff’s technical review of this section includes an independent review of the applicant’s information in the SSAR. This section describes the staff’s evaluation of the technical information presented by the applicant in SSAR Section 2.4.10.

Information Submitted by the Applicant

As required by the selected technology, the applicant will conform with required design elevations of the safety-related SSCs corresponding to the DCD for the selected reactor technology. Subsequent to selection of a technology, the applicant will design site grading and drainage systems to drain runoff up to and including the PMP away from safety-related SSCs into swales and pipes toward the Delaware River assuming all site drainage structures are blocked during the PMP event. These site drainage systems will be designed to prevent flooding of safety-related SSCs given the PMP event.

The PMSS (Section 2.4.5 of this report) is the determining event for the design basis flood at the PSEG Site. The PMSS combined with 10 percent exceedance high tide, wave runup and potential sea level rise produces a water level of 9.78 m (32.1 ft) NAVD88 as reviewed in Section 2.4.5 herein. Riprap of the appropriate designation will be placed on site slopes to provide wave runup protection. All safety-related SSCs will be designed with flood protection features to withstand the flood height of the DBF and associated effects as required for a selected reactor technology.

The Staff's Technical Evaluation

The staff reviewed the information submitted by the applicant related to flood protection at the PSEG Site. The maximum water level in the intake forebay is controlled by storm surge (SSAR Section 2.4.5). Appropriate erosion control technology will be implemented, where applicable, to protect the intake structure from wind-induced waves, runup, and associated erosion. Flood protection for the intake structure will be designed as part of the detailed design of the proposed plant at the COL stage. The intake structure will be designed to be protected from the effects of flooding and to withstand the applicable hydrodynamic forces, including wave forces, in accordance with RG 1.27, RG 1.59, and RG 1.102. Flood protection and procedures to address flooding protection requirements will be developed based on the detailed site design for review by staff at the COL stage. Consistent with the applicant's stated intention, the staff identified **COL Action Item 2.4-2** to address this item.

2.4.10.5 *Post Early Site Permit Activities*

The procedure to be developed for addressing flooding protection requirements based on the design-basis flood consistent with the detailed site design is being tracked as **COL Action Item 2.4-2.**

2.4.10.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the plant parameter envelope, and that there is no outstanding information required to be addressed in the SSAR related to this section.

As set forth above, the applicant presented and substantiated information to establish the site description. The staff reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description to allow the staff to evaluate, as documented in Section 2.4.10 of this report, whether the applicant has met the relevant requirements of 10 CFR 52.17(a)(1) and 10 CFR Part 100 with respect to determining the acceptability of the site. The staff concludes that the applicant has provided

sufficient information pertaining to flood protection to satisfy the requirements of 10 CFR Part 52 and 10 CFR Part 100. The COL applicant will address COL Action Item 2.4-2.

2.4.11 Low Water Considerations

2.4.11.1 *Introduction*

This SSAR section addresses natural events that may reduce or limit the available safety-related cooling-water supply. The applicant ensures that an adequate water supply will exist to shut down the plant under conditions requiring safety-related cooling.

This section of the report provides an evaluation of the following specific areas: (1) low-water conditions due to the worst drought considered reasonably possible in the region; (2) the effects of low water surface elevations caused by various hydrometeorological events and a potential blockage of intakes by sediment, debris, littoral drift, and ice because they can affect the safety-related water supply; (3) the effects of low water on the intake structure and pump design bases in relation to the events described in SSAR Sections 2.4.7, 2.4.8, 2.4.9, and 2.4.11, which consider the range of water supply required by the plant (including minimum operating and shutdown flows during anticipated operational occurrences and emergency conditions) compared with availability (considering the capability of the UHS to provide adequate cooling water under conditions requiring safety-related cooling); (4) the use limitations imposed or under discussion by Federal, State, or local agencies authorizing the use of the water; (5) the potential effects of seismic and non-seismic information on the postulated worst-case low-water scenario for the proposed plant site; and (6) any additional information requirements prescribed in the “Contents of Application” sections of the applicable subparts of 10 CFR Part 52.

2.4.11.2 *Summary of Application*

In SSAR Section 2.4.11, the applicant addresses the impacts of low water on safety-related water supply.

2.4.11.3 *Regulatory Basis*

The relevant requirements of NRC regulations and the associated acceptance criteria for low water considerations are specified in NUREG-0800, Section 2.4.11, “Low Water Considerations.”

The applicable regulatory requirements for identifying and evaluating low water considerations are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used the appropriate sections of the following regulatory guides for the acceptance criteria identified in NUREG-0800, Section 2.4.9.

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed.
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding.
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site.

2.4.11.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.11. The staff confirmed that the information in the application addresses the relevant information related to the low water considerations. The staff's technical review of this section includes an independent review of the applicant's information in the SSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented in SSAR Section 2.4.11.

2.4.11.4.1 Historical Low Water Conditions and Effect of Tides

Information Submitted by the Applicant

The applicant reviewed the 22-year period of record (1987 to 2008) and determined an extreme low water level of -2.07 m (-6.8 ft) NAVD88 at Reedy Point which is the closest gauge to the PSEG Site. A 1962 USGS report (Reference 2.4.11-1) describes a cold front with a sustained northwest wind averaging 45 kph (28 mph) blowing for approximately 48 hours that coincided with an extremely low tide and resulted in an elevation of -2.62 m (-8.6 ft) NAVD88 at Reedy Point. The mean low tide at Reedy Point is -0.85 m (-2.8 ft) NAVD88. The applicant noted a 90 percent exceedance low tide (-1.56 m (-5.1 ft)) NAVD88) by interpolation between Reedy Point and Lewes, DE determined by a Department of Interior report (Reference 2.4.11-2).

The Staff's Technical Evaluation

As rationale for determining the 90 percent exceedance low-tide for the 22-year record at Reedy Point, the applicant cited ANSI/ANS 2.8, which is not routinely used for analysis of low water conditions. In RAI 27, Question 02.04.11-1, the staff requested that the applicant provide additional rationale for using ANSI/ANS 2.8 as a basis for determining the 90 percent low tide exceedance for the 22-year record at Reedy Point. In a June 9, 2011, response to RAI 27, Question 02.04.11-1, the applicant noted that Standard Review Plan (SRP) Section 2.4.11

makes reference to using the same general methods of analysis as discussed in SRP Sections 2.4.5 and 2.4.6 that are applicable to low water estimates at coastal sites. In addition, the applicant noted that although SSAR Section 2.4.11 does not cite ANSI/ANS 2.8 directly, ANSI/ANS 2.8 references ANSI/ANS 2.13 as guidance when evaluating low water considerations. Therefore, the applicant's methodology used to develop the conceptual model of low tide was informed by ANSI/ANS methodology in the determination of low water conditions. The staff determined that the applicant provided a reasonable and adequate explanation detailing the rationale and justification for the use of ANSI/ANS methodology in its analysis, and that the determination of the 90 percent exceedance low tide for the Reedy Point record is adequate. Accordingly, the staff considers RAI 27, Question 02.04.11-1 resolved.

2.4.11.4.2 Low Water from Drought

Information Submitted by the Applicant

The applicant reviewed the water resource and management constraints imposed on the Delaware River basin, and the role of the DRBC flow management program for maintaining river flows. The DRBC, which was established in 1961, has a low Trenton flow objective of 84.95 m^3 (3,000 ft^3) per second although this flow rate has been modified by DRBC in times of drought (Reference 2.4.11-3). The minimum daily flow at Trenton for the 89-year period of record (1913 through 2001) is 35.11 m^3 (1,240 ft^3) per second. More recent (1956 through 2001) 20-year daily low flows of 52.10 m^3 (1,840 ft^3) have been estimated by the USGS (Reference 2.4.11-6). The applicant is a co-owner of the Merrill Creek Reservoir, which is used for low flow augmentation during times of drought to allow the applicant to continue water withdrawal from the Delaware River for power generation.

To evaluate low flow conditions at the PSEG plant, the applicant simulated low flow conditions in conjunction with drought effects with the HEC-RAS model (Reference 2.4.11-5) from the USGS Trenton Gauge to the NOAA gauge at Lewes (i.e., the mouth of the Delaware Bay). Channel geometry and floodplain topography (Section 2.4.7) from the USGS, NOAA and the USACE and Manning's *n* coefficients were calibrated using tide data and stage-discharge data for Trenton. Downstream boundary conditions were representative of the 90 percent exceedance low tide and made consistent with the upstream tide cycle. The applicant simulated low flows for the 20-year drought low daily flow at Trenton (43.32 m^3 (1,530 ft^3) per second) and flow at Trenton of 0.03 m^3 (1 ft^3) per second. For the most conservative simulation (Trenton at 0.03m^3 (1 ft^3) per second), the minimum water level at the PSEG Site was estimated at -1.56 m (-5.1 ft) NAVD88, while the low daily flow simulation produced a low water level of -1.52 m (-5.0 ft) NAVD88 at the site. The simulation results indicated that even with negligible flow at Trenton, tidal flow rather than fresh water flow is capable of providing ample and sufficient cooling water supply. The applicant concluded that the 20-year drought low flow simulation is sufficient to simulate the minimum water level at the PSEG Site.

The Staff's Technical Evaluation

The staff reviewed the DRBC policies and USGS information submitted by the applicant. The staff found a longer period of record (1912 through 2013) published by the USGS (Reference 2.4.11-6) than the applicant described; however, the daily low flow estimate for this longer period of record remains 35.11 m^3 (1,240 ft^3) per second as quoted by the applicant. The applicant is co-owner of the Merrill Creek Reservoir which could be used by the applicant for low flow augmentation of Delaware River flow.

The applicant's HEC-RAS (Reference 2.4.11-5) simulation of the 90 percent exceedance low tide in conjunction with 20-year drought low flows demonstrated that negligible freshwater flows make little difference in the minimum water level in the tidally influenced Delaware River at the PSEG Site. The applicant selected the 20-year drought low flow simulation as representative of producing minimum water levels at the PSEG Site. The staff finds the applicant's evaluation and discussion of drought low flow conditions adequate.

2.4.11.4.3 Low Water from Other Phenomena

Information Submitted by the Applicant

The applicant considered low water from other phenomena including hypothetical hurricane effects (SSAR Section 2.4.11), tsunami effects (SSAR Section 2.4.6), and winter low water with ice effects (SSAR Section 2.4.7).

The applicant reviewed the historical record for hurricanes passing within 100 nautical miles while noting that water levels from storm surge are lowest at the upwind area of semi-enclosed water bodies (Reference 2.4.11-7) such as the Delaware Bay. The greater surge upwind is consistent with gauge observations at Reedy Point (an upwind, interior bay location) having higher magnitudes than those observations at Lewes located at the mouth of the Delaware Bay for storm-associated tides. Based on a review of the historical record, in addition to observations of greater surge at Reedy Point, the applicant concluded the following for tropical cyclones passing east of Delaware Bay:

- Negative surge in the Delaware Bay is caused by tropical cyclones passing near to and east of the bay
- Negative storm surge is greater if the storm passes close to the mouth of the bay while remaining offshore
- Increasing maximum sustained winds at the point of closest approach increase the negative surge
- Maximum negative surge occurs 2 to 10 hours after the closest approach and negative surge lasts less than 6 hours

To calculate negative surge, the applicant used an established equation:

$$\text{Negative surge (ft)} = A \times (\text{maximum sustained winds})^2$$

Where A is a constant dependent on the storm center difference from the bay at the closest approach with the sustained winds in units of knots (kt) (Reference 2.4.11-7 and Reference 2.4.11-9). To apply this relationship the applicant established hypothetical meteorological parameters based on NOAA Technical Report NWS 23 (Reference 2.4.11-8) as follows:

- Central pressure, p_0 = 26.65 in mercury (Hg)
- Pressure drop, Δp = 3.5 in Hg
- Radius of maximum winds, R = from 11 to 28 NM

- Forward speed, T = from 26 to 42 kt
- Track Direction (storm coming from) = from 70 to 185 degrees azimuthal
- Coefficient related to density of air, K = 68 (when parameters are in units of kt and in Hg)

To maximize strong northwesterly winds over the bay, the applicant chose a storm track direction of 185 degrees clockwise from true north. A slow forward speed (26 kt) was chosen by the applicant to maximize the duration of high windspeeds over the Delaware Bay, with the largest radius of maximum winds (28 nautical mi (NM) (51.86 km)) chosen to produce maximum negative surge. Based on procedures defined by NOAA (Reference 2.4.11-8), the distance of closest approach to the mouth of the Delaware Bay was determined to be 20 NM (37 km).

Based on the selected parameters, the negative surge calculated is 3.32 m (10.9 ft). Combining this value with the negative surge from the 90 percent exceedance low tide and the 20-year drought low flow (SSAR Section 2.4.2.2) results in an overall negative surge of -4.85 m (-15.9 ft) NAVD88.

Details of the tsunami effect are presented in SSAR Section 2.4.6, Revision 2, reviewed herein and summarized in this section. The applicant analyzed tsunami sources from four sources:

- Currituck submarine landslide
- Currituck submarine landslide without bottom friction
- La Palma, Canary Island submarine landslide
- Hispaniola Trench earthquake

A minimum low water of -1.89 m (-6.2 ft) NAVD88 was determined by the applicant due to the Currituck submarine landslide without bottom friction as detailed in SSAR Section 2.4.6.

The low water effects from ice in conjunction with the 90 percent exceedance low tide and winter low flow, (52.10 m^3 (1,840ft^3)) per second at Trenton), were analyzed by the applicant to produce a minimum winter water level at the PSEG Site. The modified Stefan equation (Reference 2.4.11-10) is used to determine the maximum historical ice thickness. This equation uses a coefficient representative of the body of water and accumulated freezing degree days for ice thickness prediction and assumes a fresh water body. An ice thickness of 45.2 cm (17.8 in.) was determined and assumed to be conservative as the Delaware River near the new plant location is brackish and would actually have a lower freezing point resulting in a thinner ice estimate. The minimum water level from the low flow model was estimated as -1.52 m (-5.0 ft) NAVD88. Based on the above analyses, the applicant will design the intake structure such that surface ice effects occurring during low flow conditions would not prohibit or impede the operations of the intake structure.

The Staff's Technical Evaluation

The staff evaluated the applicant's consideration of low water from other phenomena including storm surge in Section 2.4.5, tsunami effects in Section 2.4.6, and winter low water with ice effects in Section 2.4.7 of this report.

The staff reviewed the applicant's descriptions of the historical negative surges and negative surge caused by a hypothetical hurricane (i.e., a PMH as defined by NOAA, Reference 2.4.11.8) as the most conservative condition for low water at the PSEG Site. The staff also reviewed the PMH parameters used by the applicant for a PMH passing by the Delaware Bay creating northerly winds that could result in significant low water near the PSEG Site and finds the parameters to be reasonable. Consistent with previous studies (Reference 02.04.11-11 and 02.04.11-12), the applicant's surface water model simulations demonstrated that tidal flow dominates surface water levels at the PSEG Site and conservatively included a 90 percent exceedance low tide in the low water evaluation. By assuming this hurricane is coincident with a 20-year low flow in the Delaware River at Trenton, NJ and 90 percent exceedance low tide, the staff agrees that the applicant's evaluation demonstrated appropriately conservative assumptions and finds the applicant's evaluation for a resulting low water level of -4.85 m (-15.9 ft) NAVD88 adequate.

The staff reviewed the physiographic nature of the tidally influenced Delaware River. Tidal flow at the PSEG Site ranges from 11,327 to 13,366 m^3 (400,000 to 472,000 ft^3) per second (Reference 2.4.9-2 and Reference 2.4.9-3), which is sufficient to supply the required water (projected to be 5 m^3 (175 ft^3) per second). Therefore, ice blockages upstream of the site would not limit the volume of safety-related water available for cooling and the staff finds the applicant's evaluation adequate.

In addition, in SSAR Section 2.4.7, the applicant stated that protective measures in accordance with ANSI/ANS-2.8-1992, Section 8.3, "Surface Ice," for the intake structure will be implemented which are acceptable to staff for mitigation of ice effects. Therefore, the staff finds the information and evaluation provided adequate.

2.4.11.4.4 Future Controls

Information Submitted by the Applicant

There are no dams on the main stem of the Delaware River. Its tributaries contain surface water impoundments that are used to manage the water supply, for flood protection, and recreation as overseen by the Delaware River Basin Commission, an independent legal authority.

The surface water elevations of the lower Delaware River (i.e., the upper Delaware Bay) are primarily dependent on tidal fluctuations. Therefore, the cooling water supply need not rely on fresh water flow to maintain an elevation sufficient for cooling water intake. There are no known controls on the Delaware River in the vicinity of the plant that could affect the availability of water or result in extreme low surface water elevations at the PSEG Site.

The Staff's Technical Evaluation

The staff evaluated the applicant's review and finds its assessment of future controls on the Delaware River basin adequate given the DRBC management of the Delaware River Basin and its water resources.

2.4.11.4.5 Plant Requirements

Information Submitted by the Applicant

The PPE intake structure requirements (approximately 5 m^3 (177 ft^3) per second), are far less than the tidal flows of the Delaware River at the PSEG Site ranging from 11,327 to 13,366 m^3 (400,000 to 472,000 ft^3) per second (Reference 2.4.9-2 and Reference 2.4.9-3), which is sufficient for the cooling water intake supply.

The Staff's Technical Evaluation

The staff reviewed the PPE as presented in the SSAR and publicly available studies (References 2.4.9-1, 2.4.9-2, 2.4.9-4) of the Delaware River tidal flows and finds the applicant's characterization of the tidal flows with respect to cooling water requirements adequate.

2.4.11.4.6 Heat Sink Dependability Requirements

Information Submitted by the Applicant

Depending on the technology selected, the intake structure provides either a non-safety-related or a safety-related source of water for the proposed plant. The applicant will design the UHS portion of the intake structure to the requirements of the selected technology to withstand extreme events including flooding from streams and rivers (SSAR Section 2.4.3), the PMSS (SSAR Section 2.4.5), the PMT (SSAR Section 2.4.6), winter ice effects (SSAR Section 2.4.7), and extreme low water conditions (SSAR Section 2.4.11). The invert elevations of the UHS makeup pumps will be set at an elevation sufficient to maintain plant operations during extreme low water conditions.

The Staff's Technical Evaluation

When the specific reactor technology is selected at the PSEG Site, the final design of the intake structure and invert elevation for maintenance of plant operations will be evaluated. The safety-related intake structure for the selected reactor technology will be designed to operate during the low water conditions as described in SSAR Section 2.4.11. The staff finds the applicant's evaluation and discussion adequate.

2.4.11.5 *Post Early Site Permit Activities*

There are no post ESP activities related to this section.

2.4.11.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the PPE, and that there is no outstanding information required to be addressed in the SSAR related to this section.

As set forth above, the applicant has provided information pertaining to low-water considerations, including hydrologic conditions that could lead to low river elevations, conditions that could result in use of a UHS, and potential effects of upstream land-use change in the drainage area. Therefore, the staff concludes that the applicant has met the requirements related to low-water considerations with respect to 10 CFR 52.17(a) and 10 CFR Part 100. Additionally, the staff concludes that, the applicant has considered the most severe natural

phenomena that have been historically reported for the site and surrounding area with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated in establishing low-water conditions for use in design.

2.4.12 Groundwater

2.4.12.1 *Introduction*

SSAR Section 2.4.12 describes the hydrogeological characteristics of the site. A significant safety objective of groundwater investigations and monitoring at this site is to evaluate the effects of groundwater on plant foundations. The evaluation is performed to assure that the maximum groundwater elevation remains within the PPE value. The other significant objectives are to examine whether groundwater provides any safety-related water supply; to determine whether dewatering systems are required to maintain groundwater elevation below the required level; to measure characteristics and properties of the site needed to develop a conceptual site model of groundwater movement; and to estimate the direction and velocity of movement of potential radioactive contaminants.

Section 2.4.12 herein presents an evaluation of the following specific areas: (1) identification of the aquifers, types of onsite groundwater use, sources of recharge, present withdrawals and known and likely future withdrawals, flow rates, travel time, gradients (and other properties that affect the movement of accidental contaminants in groundwater), groundwater levels beneath the site, seasonal and climatic fluctuations, monitoring and protection requirements, and manmade changes that have the potential to cause long-term changes in local groundwater regime; (2) effects of groundwater levels and other hydrodynamic effects of groundwater on design bases of plant foundations and other SSCs important to safety; (3) reliability of groundwater resources and related systems used to supply safety-related water to the plant; (4) reliability of dewatering systems to maintain groundwater conditions within the plant's design bases; (5) potential effects of seismic and non-seismic information on the postulated worst-case groundwater conditions for the proposed plant site; and (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.12.2 *Summary of Application*

This SSAR section addresses groundwater conditions in terms of impacts on structures and water supply. The application section addresses these issues as follows:

- The applicant described geologic formations, and regional and local groundwater aquifers, sources, and sinks.

- The applicant described proposed groundwater use for PSEG Site operations consisting of sanitary/potable use, demineralized makeup water, and fire suppression.

- The applicant described dewatering that will be required during construction, but due to the proposed plant grade elevation, no dewatering will be required when the plant is operational.

- The applicant described the present and projected future regional water use, relying on reports and databases of the USGS, the U.S. Environmental Protection Agency (EPA), the State of New Jersey, and the Delaware River Basin Water Commission.

- The applicant described water levels and flow directions both regionally and onsite. The applicant provided groundwater level maps over the site and regional maps showing major hydrologic features.

2.4.12.3 *Regulatory Basis*

The relevant requirements of the NRC regulations for characterizing groundwater, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.12.

The applicable regulatory requirements for groundwater are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

The staff also used the acceptance criteria identified in NUREG-0800, Section 2.4.12:

- Local and Regional Groundwater Characteristics and Use: The applicant should supply a complete description of regional and local groundwater characteristics and groundwater use, groundwater monitoring and protection requirements, and any man-made changes with a potential to affect regional groundwater characteristics over a long period of time.

- Effects on Plant Foundations and other Safety-Related Structures, Systems, and Components: The applicant should supply a complete description of the effects of groundwater-surface elevations and other hydrodynamic effects on the design bases of plant foundations and other SSCs important to safety.

- Reliability of Groundwater Resources and Systems Used for Safety-Related Purposes: The applicant should supply a complete description of all SSCs important to safety that depend on groundwater, as well as data and analysis regarding the reliability of the groundwater source.

- Reliability of Dewatering Systems: The applicant should supply a complete description of the site dewatering system, including its reliability to maintain the groundwater conditions within the groundwater design bases of SSCs important to safety.

- Consideration of Other Site-Related Evaluation Criteria: The applicant should supply an assessment of the potential effects of seismic and non-seismic information about the postulated worst-case scenario related to groundwater effects for the proposed plant site.

2.4.12.4 *Technical Evaluation*

The staff reviewed the information in SSAR Section 2.4.12. The staff confirmed that the information in the application addresses the relevant information related to the groundwater considerations. The staff's technical review of this section includes an independent review of the applicant's information in the SSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

The applicant identified aquifers, groundwater use categories, sources of recharge, present and future withdrawals, flow rates, travel times and gradients and other properties that affect transport of radionuclides, groundwater levels in the site vicinity including seasonal and climatic variations, monitoring and protection plans, and manmade changes that have the potential to cause long-term changes in the localized flow system. This section of the report provides the staff's evaluation of the technical information presented in SSAR Section 2.4.12.

2.4.12.4.1 Groundwater System

Information Submitted by the Applicant

The applicant supplied a narrative description of the hydrogeology of the region and the site. In the region, the aquifer/aquitard sequence contains the following units (Reference 2.4.12-1):

- Alluvium
- Kirkwood-Cohansey Formation
- Vincentown Formation
- Navesink-Hornerstown Formation
- Mount Laurel-Wenonah Formation
- Marshalltown Formation
- Englishtown Formation
- Woodbury Formation
- Merchantville Formation
- Potomac-Raritan-Magothy (PRM) Formation

The PRM system is the key regional potable groundwater source and is the formation used for onsite water withdrawals. Water withdrawal rates for the Salem Generating Station (SGS) and HCGS were given and estimated withdrawal rates were provided using a PPE as the final technology selection has not been made. The proposed PSEG Site operations will use groundwater for sanitary/potable use, demineralized makeup water, and fire suppression. Makeup to a safety-related ultimate heat sink (UHS) (if necessary) and the non-safety-related circulating water system (CWS) for the new plant will be drawn from the Delaware River. Dewatering will be required during construction but not when the plant is operational.

The Staff's Technical Evaluation

Although the applicant indicated that the aquifer/aquitard sequence for the site includes the Kirkwood-Cohansey Formation, the New Jersey Geological Survey (Reference 2.4.12-2) has indicated that this Formation is absent from the site area. Since the applicant performed field studies and derived parameters from these studies for the interval proposed to be the Kirkwood-Cohansey Formation, the formal name for this interval had no impact on the staff's evaluations and conclusions in this report.

The staff reviewed the information provided in SSAR Section 2.4.12, Revision 0 and determined that additional information was needed to describe the differing hydrologic units to confirm groundwater pathways and flow rates. Therefore, in RAI 29, Question 02.04.12-1, the staff requested that the applicant provide more detail on the hydraulic parameters for the hydrologic units. In a June 14, 2011, response to RAI 29, Question 02.04.12-1, the applicant provided the requested information as summarized here: the groundwater elevation will determine the hydrostatic loading on the plant foundations, which is safety-related. The new plant grade will be at an elevation of 11.25 m (36.9 ft) NAVD88. Based on 1 year of data, the maximum measured groundwater level at the power block was 0.48 m (1.57 ft) NAVD88. After construction, the water level is anticipated to return to slightly higher levels due to the soil retention wall barrier used during dewatering that will be left in place. Anticipated water levels will be on average 0.9 – 1.2 m (3 – 4 ft) NAVD88 with a maximum elevation just above the retention wall at 1.6 m (5.2 ft) NAVD88. For analysis of hydrostatic loading, a water elevation of 1.8 m (6 ft) NAVD88 is used. At this elevation, the hydrostatic loads are much less than the maximum required of the potential technologies. The staff reviewed the applicant's response containing parameters as well as groundwater contour map and finds that the additional information in conjunction with the original description of the hydrological units and flow directions was adequate. The applicant committed to revise the SSAR with this information. The staff verified that Revision 1 of the ESP application (May 21, 2012) contains the applicant's committed information. Accordingly, the staff considers RAI 29, Question 02.04.12-1 resolved.

2.4.12.4.2 Groundwater Modeling During Operation

Information Submitted by the Applicant

The applicant provided a description of groundwater modeling efforts to support routine operations (Reference 2.4.12-3). The modeling study was performed to justify water withdrawal permits for the existing plants (SGS and HCGS) and included an assessment of the potential for saltwater intrusion in the Mount Laurel-Wenonah and PRM aquifers and the impact of water withdrawal on regional groundwater flow. The applicant concluded that there would be no major impact to the salinity of the Upper PRM even at simulated flows twice the level of the current pumping rate. Based on this information and the PPE for water withdrawal at the proposed plant, the applicant concluded that there was sufficient groundwater availability on site to meet the new plant's needs and not induce saltwater intrusion.

The Staff's Technical Evaluation

The staff reviewed the information provided in SSAR Section 2.4.12, Revision 0 and determined that additional information was needed to evaluate the groundwater modeling. Therefore, in RAI 29, Questions 02.04.12-1 through 02.04.12-5, the staff requested that the applicant provide the following information:

- Clarification on the use of site-specific parameters and data (specifically whether porosities values were effective or total porosity)

- Model calibration studies including comparison of model predictions with measured values

- The impacts of a) boundary conditions, b) horizontal grid size, and c) vertical grid size on model accuracy

In a June 14, 2011, response the applicant provided the requested information. The staff reviewed this response and concluded that the information provided was sufficient to evaluate the modeling used to support groundwater withdrawals for plant use. However, the applicant indicated that it believed that no changes to the SSAR were required. After discussion with the applicant, the staff issued RAI 38, Question 02.04.12-6, requesting that this information be included in the SSAR. In a September 22, 2011, response to RAI 38, Question 02.04.12-6, the applicant stated that the information would be incorporated into the next revision of the SSAR. The staff confirmed that these changes have been incorporated in Revision 1 of the ESP application (May 21, 2012). Accordingly, the staff considers RAI 29, Questions 02.04.12-1 through 02.04.12-5, and RAI 38, Question 02.04.12-6 resolved.

2.4.12.4.3 Groundwater Modeling During Dewatering

Information Submitted by the Applicant

The applicant provided a description of groundwater modeling efforts to support dewatering operations required during construction of the new plant. The model simulated dewatering of the plant area down to the Kirkwood Formation over most of the proposed plant boundary and to the deeper Vincetown Formation beneath the safety-related structures. Dewatering will be accomplished by temporary wells around these two regions. The best estimate for the pumping rate to effectively dewater the site was estimated to be between 19,682 liters (l) (5,200 gallons (gal)) and 21,196 l (5,600 gal) per minute for the first year and decreasing afterwards. Sensitivity analysis indicated that the first year pumping rate could range from 11,355 l (3,000 gal) to 28,766 l (7,600 gal) per minute. Potentiometric surfaces based on the modeling were provided and existing structures that may be impacted were identified. During construction, the applicant noted that additional measures, such as sand drains, may be needed to effectively dewater the fill and alluvium. The design for such measures and the dewatering study will be refined when a final technology is selected.

The Staff's Technical Evaluation

The staff reviewed the information provided in SSAR Section 2.4.12, and determined that additional information was needed to evaluate the groundwater modeling used to support dewatering. Therefore, in RAI 29, Question 02.04.12-3, the staff requested that the applicant supply the reference material and information on model specific parameters (porosity, hydraulic conductivity, etc.) used in the analysis. In a June 14, 2011, response to RAI 29, Question 02.04.12-3, the applicant adequately addressed existing conditions as summarized here: during dewatering, the piezometric head is decreased, the effective vertical pressure exerted by the soil column is increased by the amount of that decrease times the unit weight of water. The increase in vertical effective pressure can cause settlement of soils. The settlement, in turn, can affect the performance of structures supported on the soil, or can add downward loads to pile foundations supporting the structures. Current water level contour maps in the vicinity of SGS and HCGS indicate that the safety of these foundations will not be

compromised by dewatering. However, the applicant indicated that no changes to the SSAR were needed. After a teleconference with the applicant, the staff issued RAI 38, Question 02.04.12-6 requesting that this information be included in the SSAR. In a September 22, 2011, response to RAI 38, Question 02.04.12-6, the applicant committed to include the information into the next revision of the SSAR. The applicant further stated that groundwater modeling would be refined after the reactor vendor is selected, and the final excavation geometry is determined, and preparation of the COL application would require additional data, which would be obtained from pumping tests or other methods, to further refine hydrogeologic parameters and model estimates of dewatering rates and drawdowns beneath existing site structures. The staff finds the applicant's information and rationale adequate. The staff also confirmed that the applicant's committed changes have been incorporated in Revision 1 of the ESP application (May 21, 2012). Accordingly, the staff considers RAI 29, Question 02.04.12-3 and RAI 38, Question 02.04.12-6 resolved. Consistent with the applicant's stated intention, the staff identified **COL Action Item 2.4-3** to address the review of future site characterization data, dewatering plans, and groundwater monitoring plans.

2.4.12.4.4 Groundwater Monitoring

Information Submitted by the Applicant

The applicant indicated that best management practices will be used to minimize impacts to the groundwater and that the monitoring programs will be developed once the final technology is selected.

The Staff's Technical Evaluation

Groundwater monitoring information will be reviewed by staff at the COL stage once a reactor technology is selected. The staff will be tracking the applicant's groundwater monitoring program via **COL Action Item 2.4-3**.

2.4.12.5 *Post Early Site Permit Activities*

The review of future site characterization data, dewatering plans, and groundwater monitoring plans at the COL stage is being tracked as **COL Action Item 2.4-3.**

2.4.12.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the plant parameter envelope, and that there is no outstanding information required to be addressed in the SSAR related to this section.

As set forth above, the applicant has provided sufficient information pertaining to groundwater. Therefore, the staff concludes that the applicant has met the requirements related to groundwater in 10 CFR 52.17(a), 10 CFR 100.23, and 10 CFR 100.20(c). The COL applicant will address **COL Action Item 2.4-3**.

2.4.13 Accidental Release of Radioactive Liquid Effluent in Ground and Surface Waters

2.4.13.1 *Introduction*

SSAR Section 2.4.13 considers the potential effects of relatively large accidental releases from systems that handle liquid effluents generated during normal plant operations. Such releases would have relatively low levels of radioactivity, but could be large in volume. Normal and accidental releases are also considered in the applicant's environmental report. The accidental release of radioactive liquid effluents in ground and surface waters is evaluated based on the hydrogeological characteristics of the site that govern existing uses of groundwater and surface water and their known and likely future uses.

The source term from a postulated accidental release is reviewed under NUREG-0800, Section 11.2, following the guidance in Branch Technical Position (BTP) 11-6, "Postulated Radioactive Releases Due to Liquid-Containing Tank Failures." The source term is determined from a postulated release from a single tank outside of the containment. The results of a consequence analysis are evaluated against SRP Section 11.2 and BTP 11-6 guidance and effluent concentration limits (ECLs) of Table 2, Column 2 in 10 CFR Part 20, Appendix B, "Annual Limits on Intake (ALIs) and Derived Air Concentrations (DACs) of Radionuclides for Occupational Exposure; Effluent Concentrations; Concentrations for Release to Sewerage," as SRP acceptance criteria. Under SRP guidance, the effluent concentration limits of 10 CFR Part 20, Appendix B are applied as acceptance criteria only for the purpose of assessing the acceptability of the results of the consequence analysis and are not intended for demonstrating compliance with ECLs.

The following specific areas are reviewed by the staff: (1) alternative conceptual models of the hydrology at the site that reasonably bound hydrogeological conditions at the site inasmuch as these conditions affect the transport of radioactive liquid effluent in the groundwater and surface water environment; (2) bounding set of plausible surface and subsurface pathways from potential points of an accidental release to determine the critical pathways that may result in the most severe impact on existing uses and known and likely future uses of groundwater and surface water resources in the vicinity of the site; (3) ability of the groundwater and surface water environments to delay, disperse, dilute, or concentrate accidentally released radioactive liquid effluent during its transport; (4) assessment of scenarios wherein an accidental release of radioactive effluents is combined with potential effects of seismic and non-seismic events (e.g., assessing effects of hydraulic structures located upstream and downstream of the plant in the event of structural or operational failures and the ensuing sudden changes in the regime of flow); and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.13.2 *Summary of Application*

This section provides an analysis of an accidental liquid release of effluents or radioactive wastes to the groundwater at the PSEG Site. The postulated accident scenario is combined with the conceptual site model to evaluate potential impacts to receptors should a catastrophic tank rupture occur during plant operations and instantaneously release radionuclides to the groundwater environment. The resulting calculated concentrations that would reach the potential surface water receptors are then compared to the ECLs published in 10 CFR Part 20, Appendix B. The calculated results are then assessed using the unity rule where the sum of the

ratios of the calculated concentrations to the corresponding ECLs for all radionuclides in the effluent may not exceed one.

2.4.13.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the pathways of liquid effluents in ground and surface waters, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.13.

The applicable regulatory requirements for evaluating accidental release of radioactive liquid effluents in ground and surface waters are set forth in the following:

- 10 CFR 52.17(a)(1)(vi), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

The staff also used the acceptance criteria identified in NUREG-0800, Section 2.4.13:

- Alternate Conceptual Models: Alternate conceptual models of hydrology in the vicinity of the site are reviewed.

- Pathways: The bounding set of plausible surface and subsurface pathways from the points of release are reviewed.

- Characteristics that Affect Transport: Radionuclide transport characteristics of the groundwater environment with respect to existing and known and likely future users should be described.

- Consideration of Other Site-Related Evaluation Criteria: The applicant’s assessment of the potential effects of site-proximity hazards, seismic, and non-seismic events on the radioactive concentration from the postulated tank failure related to accidental release of radioactive liquid effluents to ground and surface waters for the proposed plant site is needed.

- BTP 11-6 provides guidance in assessing a potential release of radioactive liquids after the postulated failure of a tank and its components, located outside of containment, and effects of the release of radioactive materials at the nearest potable water supply, located in an unrestricted area, for direct human consumption or indirectly through animals, crops, and food processing.

The staff used best current practices to analyze groundwater transport of radioactive liquid effluents. In addition, the hydrologic characteristics should conform to appropriate sections from RG 1.113, “Estimating Aquatic Dispersions of Effluents from Accidental and Routine Reactor Releases for the Purpose of Implementing Appendix I.”

2.4.13.4 *Technical Evaluation*

The staff reviewed the SSAR, Revision 2, and subsequent responses to RAIs related to the accidental release of radioactive liquid effluents in ground and surface waters included under Section 2.4.13 of the application. The staff's review confirmed that the information in the application addresses the relevant information related to this section.

2.4.13.4.1 Release Site Location

This evaluation concerns the location of the spill release site and the conservatism of this assumption as discussed in the SSAR Sections 2.4.13.1.2 through 2.4.13.1.9.

Information Submitted by the Applicant

The applicant assumed in SSAR, Revision 0 that the radioactive release would occur on the western edge of the power block but not the eastern or northern edge. The applicant indicated that this was conservative (shortest travel time, least decay), because the nearest surface water receptor is located on the west side of the power block (Delaware River). In addition, it was determined that travel time to the northeast surface water receptor would take much longer and thus result in lower concentrations at the discharge to surface water.

The Staff's Technical Evaluation

The Delaware River is the closest surface water body to the power block and has a great dilution capacity, while the northern and eastern tidal streams are further away and have a much lower dilution capacity. The northeast migration path was not quantitatively addressed in the SSAR, Revision 0. The staff estimates indicate that concentrations from a spill reaching the tidal streams could be much higher than those estimated in the Delaware River by the applicant.

In RAI 31, Questions 02.04.13-5 (potential for release to the northeast) and 02.04.13-10 (conservatism of receptor locations), the staff communicated to the applicant concerns relating to a release toward the northeast, and requested that the applicant address these concerns. In a June 30, 2011, response, the applicant provided several qualitative arguments, summarized below, as to why a release toward the east side is the most conservative. The applicant also generated a site wide water table contour map (Figure 2.4.13-1 of this report) in response to RAI 29, Question 02.04.12-2, based on monthly water level data measured in 2009 to indicate predominant and sustained westerly groundwater flow across the PSEG Site with easterly components due to tidal fluctuation. These arguments for the conservatism of the westerly path noted that a substantially longer easterly travel time allowed for more radionuclide decay before discharge to surface water.

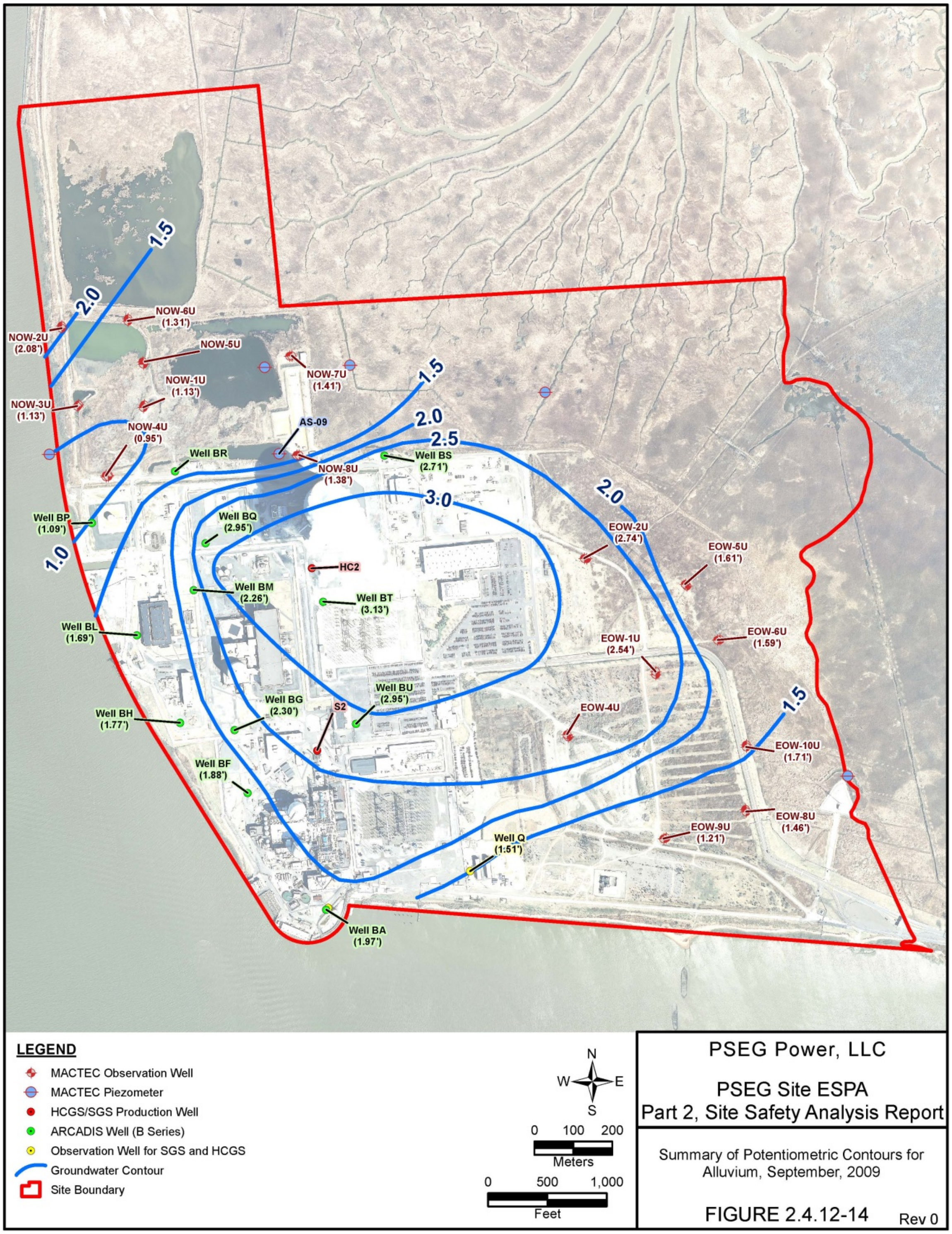

Figure 2.4.13-1 PSEG Site Wide Water Levels September 2009, (from SSAR Revision 3, Figure 2.4.12-14)

In a June 30, 2011, response to RAI 31, Question 02.04.13-10, the applicant explained that concentration attenuation factors in the accidental release scenario include sorption, decay, dilution, and dispersion vertically through migration into a lower, thicker aquifer. The release into the alluvium spans a relatively thin aquifer, whereas any further vertical migration into the next lower aquifer (the Vincentown) toward potential private well receptors (as well as the Delaware River) would be moderated by a longer flow path, additional dilution by dispersion, sorption, and added time for decay to occur which would indicate that migration through the alluvium is the more conservative pathway.

The staff confirmed that these qualitative explanations with respect to RAI 31, Question 02.04.13-10, were included in SSAR, Revision 1, and subsequently, Revision 2. While the staff concurs that the conceptual model contains several conservative assumptions, this does not imply that the entire analysis is conservative. Conservatism in the analysis depends on the choice of parameters that reasonably represent the physical properties of the system.

Further, the applicant did not address the difference in dilution capacity between the Delaware River and the tidal streams toward the northeast nor did it perform a quantitative evaluation of the potential northeasterly flow. A teleconference with the applicant was held on July 25, 2011, to discuss staff's additional information needs after the June 30, 2011, response to RAI 31, Question 02.04.13-5. In a July 25, 2011, response to RAI 31, Question 02.04.13-12, the applicant submitted changes to SSAR Section 2.4.13 and committed to incorporate these into the next SSAR revision. The staff verified that the applicant incorporated the committed changes in Revision 1 of the ESP application (May 21, 2012).

The applicant's July 25, 2011, response to RAI 31, Question 02.04.13-12, stated that flow directions are to the east and south for groundwater in the eastern portion of the facility. The applicant predicted post-construction groundwater mounding (SSAR Section 2.4.12) as a function of impacts due to post-construction features, the hydrogeologic properties of the alluvium, the distribution of recharge across the site, and distances and directions to groundwater sinks (i.e., locations where groundwater would discharge to surface water). A similar groundwater mounding pattern exists in the vicinity of the SGS and HCGS power block area (Figure 2.4-13-1 of this report).

Given that there were no available groundwater monitoring data points in the northeastern marsh area to indicate otherwise, that there is a reasonable possibility for easterly flow in the eastern portion of the power block area, and that the final plant design may result in a release on the east side of the power block, the staff requested supplemental information in RAI 60, Question 02.04.13-14, after a September 27, 2011, teleconference with the applicant.

The applicant addressed flow to the eastern portion of the site in a May 3, 2012, response. After reviewing the applicant's response, the staff communicated to the applicant in RAI 68, Question 02.04.13-15 further staff concerns related to the incorporation of post-construction groundwater conditions into the release scenarios, and requested that the applicant address these concerns. In a December 20, 2012, response, the applicant adequately and acceptably addressed staff concerns regarding post-construction groundwater conditions and release scenarios, which are discussed below in Section 2.4.13.4.4, "Migration Scenarios" of this report. Accordingly, the staff considers RAI 68, Question 02.04.13-15; RAI 31, Questions 02.04.13-5 and 02.04.13-10; and RAI 29, Question 02.04.12-2 resolved.

2.4.13.4.2 Postulated Release to Alluvium

This evaluation concerns the release of a spill directly to the Alluvium aquifer that bypasses the hydraulic fill and whether this is a sufficiently conservative assumption as initially postulated in SSAR Revision 0, Section 2.4.13.1.3.

Information Submitted by the Applicant

The applicant stated that the release will be to the alluvium and no groundwater travel time for downward migration through the hydraulic fill is provided. Once in the alluvium, the contaminants migrate to the edge of the Delaware River where they discharge to the surface waters. The effect of dredging cutting into the alluvium along the river bank that would create a shorter travel time is conservatively incorporated into the conceptual model through the applicant's assumption of groundwater discharge through the west river bank immediately adjacent to the site rather than further offshore.

The Staff's Technical Evaluation

The applicant's evaluation in SSAR, Revision 0, did not fully explain the detail of transport of contaminants after being released into the structural fill of the power block area that will replace the existing and hydraulic fill and alluvium to be excavated. Therefore in RAI 31, Questions 02.04.13-6 and 02.04.13-7, the staff, requested that the applicant further clarify the hydraulic relationships between the structural fill, hydraulic fill, alluvium, and Delaware River to verify that the assumptions are conservative.

In a June 30, 2011, response, to RAI 31, Questions 02.04.13-6 and 02.04.13-7, the applicant provided further clarification indicating the spill would be released to the proposed structural fill and that this material would provide additional retention and decay time that is not included in the assumptions. In addition, the impact of dredging was included by assuming the discharge from the alluvium would be at the Delaware River bank (87 m (285 ft)) from the western edge of the proposed power block area and the assumed conservative release location).

The staff verified that based on the relatively low hydraulic conductivity of the hydraulic fill, groundwater will preferentially flow from the structural fill into the alluvium rather than the hydraulic fill at the edge of the power block area. Therefore, the assumption of an instant release of a surface spill to the alluvium ignores additional retention and decay within the structural fill that would be the case in an actual spill. This assumption is conservative because it transports radionuclides to the Delaware River faster than could actually occur. Therefore, the applicant provided an adequate and acceptably conservative response for the staff's evaluation, and the staff confirmed that the information was incorporated in Revision 1 of the ESP application (May 21, 2012). Accordingly, the staff considers RAI 31, Questions 02.04.13-6 and 02.04.13-7 resolved.

2.4.13.4.3 Impacts of Tidal Flushing on the Groundwater Flow Rate in the Alluvium

This evaluation concerns the applicant's calculation of hydraulic gradients in the alluvium towards the river without including explicit tidal effects as initially discussed in SSAR, Revision 0, Section 2.4.13.1.3.

Information Submitted by the Applicant

The applicant presented the probable maximum and average hydraulic gradients calculated for the alluvium as discussed in the SSAR, Revision 0. With the additional (hydraulic conductivity, porosity and discharge area) site data, the average and maximum groundwater velocities and flow rate are calculated.

The Staff's Technical Evaluation

Since the impacts from tidal influences were not explicitly discussed in the SSAR, Revision 0, the staff could not verify these calculations. Within the shallow groundwater system, the influence of the tidal cycle will continuously change the groundwater velocity and an estimate of the resultant velocity and groundwater flow direction needed to be made by the applicant.

In RAI 31, Question 02.04.13-3, the staff requested that the applicant provide clarification of how tidal influences from the Delaware River were taken into account and the rationale to support the premise that the predominant groundwater flows in the alluvium and the Vincentown are westward toward the Delaware River.

On June 30, 2011, the applicant provided initial response to RAI 31, Question 02.04.13-3, and following a clarifying teleconference with the staff, the applicant provided a supplemental response on July 25, 2011. The applicant modified the SSAR to explain that tidal flushing within the alluvium would lead to increased dilution, which is not accounted for in the model, and therefore, adds to the conservatism of the assumptions. Tidal flushing will impact the instantaneous groundwater flow into the river, but not the average flow. The average flow is controlled by the hydraulic gradients on land that are toward the Delaware River on the west side of the power block. The applicant's explanation and additional information was an adequate basis for the staff's evaluation, and the staff confirmed that this information was incorporated in Revision 1 of the ESP application (May 21, 2012). Accordingly, the staff considers RAI 31, Question 02.04.13-3 resolved.

2.4.13.4.4 Migration Scenarios

This evaluation relates to the staff's assessment of the applicant's consideration of the easterly release scenario as discussed in SSAR, Revision 0, Section 2.4.13.1.4, and a westerly release as initially described in SSAR, Revision 1, Section 2.4.13.1.4.

Information Submitted by the Applicant

The applicant stated in the SSAR, Revision 0, that the easterly migration scenario is less conservative than the westerly because the distance to tidal streams is longer than to the Delaware River; the release would need to migrate vertically upward through sediments of low permeability; and there would be dilution in the surface water due to tidal influences.

The Staff's Technical Evaluation

This scenario was not completely evaluated by the applicant and did not contain quantitative estimates as to travel time, dilution, and decay of the contaminants. Due to the lower flows and subsequent dilution capacity of the eastern tidal areas, there is a possibility that even with a longer travel time the concentration at the tidal streams could eventually be higher than that in the Delaware River. Without additional information the staff could not adequately evaluate this scenario.

Therefore in RAI 31, Question 02.04.13-4, the staff requested that the applicant provide clarification on this issue. After reviewing the applicant's initial and supplemental responses, dated June 30, 2011, and July 25, 2011, respectively, the staff had further questions. Subsequently, following a clarifying teleconference with the applicant on September 27, 2011, the staff requested supplemental information in RAI 60, Question 02.04.13-14.

Following a related March 1, 2012, teleconference with the applicant, the applicant committed to address this issue based on the final design at the COL stage. However, the applicant responded on May 3, 2012, to RAI 60, Question 02.04.13-14, with a proposed revision to the SSAR that addressed both the easterly and westerly release pathways.

In the release analysis, the applicant chose the northeast corner of the power block as the potential release site and Fishing Creek in a northeasterly direction as the groundwater discharge point. The staff independently verified from piezometer and well water level data the applicant's conceptualization that the streams closer to the postulated release point are not in contact with the groundwater but are tidally driven only.

Given the staff's hydrogeologic understanding of the site as well as the location of Fishing Creek at the old Delaware River shoreline, (i.e., the shoreline prior to the creation of Artificial Island with hydraulic fill), the applicant's choice of Fishing Creek as the discharge point of an accidental release into the alluvium is reasonable.

Using the highest measured groundwater elevation at the power block in the alluvium, (approximately 0.48 m (1.57 ft) NAVD88); the applicant calculated that 0.015 m^3/day (0.53 ft^3/day) would be needed to achieve the sum of radionuclide to ECL ratios of unity at the discharge point in Fishing Creek. Fishing Creek is estimated from aerial photographs to be approximately 61 m (200 ft) wide, therefore, this low flow rate is easily achieved. The staff confirmed that the analysis and discussion of an easterly release was incorporated in Revision 2 of the ESP application (March 31, 2013).

The staff noted that the groundwater elevation, 0.48 m (1.57 ft) NAVD88, used to calculate the hydraulic gradient to Fishing Creek is inconsistent with the mounded elevations 1.8 - 3 m (6 - 10 ft) NAVD88, assumed for the structural analysis and post construction modeling presented in the SSAR, Revision 1 (pages 2.4-164, 2.4-167, and 2.5-306). This inconsistency was discussed with the applicant during a November 15, 2012, conference call and subsequently documented in RAI 68, Question 02.04.13-15. The applicant responded to this RAI on December 20, 2012. For the release toward Fishing Creek, the applicant addressed with appropriate conservatism, a groundwater mound resulting from the construction of the power block by shortening the flow pathway from 1,280 m (4,200 ft) to 1,158 m (3,800 ft) (i.e., a transport pathway beginning just outside the influence of the mound). This raised the dilution capacity required to meet the Unity Rule at Fishing Creek from 0.015 m^3 (0.53 ft^3) per day to 0.06 m^3 (2.12 ft^3) per day. The staff finds the applicant's characterization of transport pathways and of the dilution capacity of Fishing Creek as a tidal stream adequate.

For a westerly transport flow path to the Delaware River considering post-construction mounding, the applicant determined that instead of 3.17 m^3 (112 ft^3) per second (without mounding), 9.52 m^3 (336 ft^3) per second of river flow for dilution would be required to meet the Unity Rule. There is no reasonably conceivable scenario in which the Delaware River adjacent to the site could fall below either flow rate. The staff finds the applicant's determination that groundwater mounding influencing westerly flow to the Delaware River would not significantly change, and calculations presented in the SSAR adequate. Accordingly, the staff considers

RAI 31, Question 02.04.13-4, RAI 60, Question 02.04.13-14, and RAI 68, Question 02.04.13-15 resolved.

2.4.13.4.5 Average and Maximum Groundwater Velocities

This topic concerns the level of confidence the applicant has in both the average and maximum groundwater velocities in the contaminant migration estimates given the length of the monitoring record.

Information Submitted by the Applicant

In SSAR, Revision 0, Section 2.4.13.1.6, the applicant initially presented the estimated average and maximum groundwater flow velocities from the power block to the Delaware River bank (0.0024 and 0.029 meters per day (m/day) (0.00788 and 0.094 feet per day (ft/day)), respectively).

The Staff's Technical Evaluation

The specific methodology used to determine these migration estimates was not presented by the applicant; however, the applicant stated in the SSAR, Revision 0, that the net migration rates are estimated from site-specific data including the tidal influences.

In RAI 31, Question 02.04.13-8, the staff requested that the applicant present flow velocity calculations. In a June 30, 2011, response, the applicant provided information on the method used to calculate groundwater flow velocities and hydraulic data to calculate flow velocities. The staff confirmed the velocities calculated using the same method. The staff calculated a range of potential water velocities based on hydraulic potential maps supplied by the applicant and these values were always less than the maximum value determined by the applicant although based on existing conditions. In addition, existing conditions are inconsistent with the post-construction mounded elevations anticipated by the applicant. The applicant's post construction water levels of 1.8 to 3 m (6 to 10 ft) NAVD88 were presented in the SSAR markup. These higher post-construction groundwater elevations were incorporated into release scenarios as described in the applicant's December 20, 2012, response to RAI 68, Question 02.04.13-15 and included in Revision 2 of the ESP application (March 31, 2013). The staff considered the applicant's response to RAI 31, Question 02.04.13-8 in conjunction with the response to RAI 68, Question 02.04.13-15, and determined to be adequate. Accordingly, the staff considers RAI 31, Question 02.04.13-8 resolved.

2.4.13.4.6 Delaware River Dilution Calculations

This evaluation concerns the specific dilution estimates for groundwater discharge to the Delaware River.

Information Submitted by the Applicant

In SSAR, Revision 0, Sections 2.4.13.1.7 and 2.4.13.1.8, dilution factors for the discharge of groundwater to the Delaware River were presented as "substantial" but no supporting calculations were provided by the applicant. Only the comparison of the groundwater discharge to two thirds of the total average river flow is presented. The only quantitative estimate given is that a minimum of 3.17 m^3 (112 ft^3) per second is required to dilute the estimated radionuclide concentrations to below the unity rule in the groundwater flux of 0.24 m^3 (8.59 ft^3) per day.

The Staff's Technical Evaluation

In RAI 31, Question 02.04.13-11, the staff requested that the applicant address the rationale for calculating the minimum flow rate needed to achieve the unity rule while also presenting the much larger dilution capacity of two thirds of the Delaware River flow. In a June 30, 2011, response, the applicant included the changes to the SSAR to clarify that 3.17 m^3 (112 ft^3) per second flow rate in the river is the minimum needed to achieve the concentrations satisfying the unity rule. Dilution by two thirds of the river flow was presented in Tables 2.4.13-3 and 2.4.13-5 as a qualitative example of the potential ultimate dilution capacity of the river. The value of 3.17 m^3 (112 ft^3) per second is based on existing groundwater conditions and potential flow rates. The staff considers RAI 31, Question 02.04.13-11 resolved.

Potential effects of radionuclide retardation were also addressed in the applicant's June 30, 2011, response to RAI 31, Question 02.04.13-11. The applicant demonstrated that if retardation coefficients from NUREG/CR-5512, "Residual Radioactive Contamination From Decommissioning: User's Manual DandD Version 2.1," were applied to the PSEG Site, the required flow to achieve the unity rule is reduced to about 0.2 cubic feet per second (cfs). In the response to RAI 68, Question 02.04.13-15, the applicant's calculation used flow rates based on the applicant's anticipated post-construction groundwater levels (1.8 - 3 m (6 - 10 ft) NAVD88). This was evaluated and included in the SSAR, Revision 2.

In RAI 31, Question 02.04.13-9, the staff requested that the applicant address its rationale for using the subset of all possible radionuclides that could contribute to the unity rule calculations. In a June 30, 2011, response, the applicant explained that very short half-lives of some of the radionuclides or low activity levels lead to their exclusion from further calculations. This information is incorporated into SSAR, Revision 2, Table 2.4.13-1, which contains the bounding activity concentrations for all the radionuclides initially considered, and identified the radionuclides eliminated due to short half-life or low activity concentration. Accordingly, the staff considers RAI 31, Question 02.04.13-9 resolved.

2.4.13.4.7 Potential Migration into Deeper Aquifers

Information Submitted by the Applicant

In SSAR, Revision 0, Section 2.4.13.1.9, the applicant stated that the spill could migrate into the Vincentown Formation in the power block area where the alluvium and underlying formation (also known as "Kirkwood") are excavated during construction or are thin or absent. The applicant stated that the initial dilution in the Vincentown would be 10 times greater than in the alluvium because the Vincentown is 10 times thicker than the alluvium. The applicant also stated that groundwater migration in the Vincentown is believed to be toward the Delaware River.

In SSAR Section 2.4.13.1.9 the applicant also stated that migration into the PRM aquifers could be induced by pumping wells but radial groundwater influx to the well due to pumping would dilute radionuclides below detectable levels.

The Staff's Technical Evaluation

If contaminated liquid migrates from the alluvium into the Vincentown the flow will be laminar and the assumption that the full aquifer thickness will be available for dilution is not appropriate. More likely, a thin layer of contaminated water will form at the top of the potentiometric surface in the Vincentown. The analysis required a realistic dilution assumption and a determination of

the direction of flow within the Vincentown formation. In RAI 31, Questions 02.04.13-12 (1), (2), (3), (4), (5), and (6), the staff requested that the applicant address these issues. RAI 31, Question 02.04.13-12 (1) was discussed in Section 2.4.13.4.1, "Release Site Location" of this report.

In a June 30, 2011, response to RAI 31, Question 02.04.13-12 (2), the applicant indicated agreement that the full thickness of the Vincentown Formation should not be considered for dilution and revised SSAR Section 2.4.13.1.9 to clarify the assumptions inherent in a release that could reach and migrate through the Vincentown.

The applicant provided a description of the flow paths to the Vincentown Aquifer and stated that the flow paths in the Vincentown Aquifer would lead to the Delaware River either just to the west after migrating upward through the alluvium and Kirkwood units or 1.6 km or 3.2 km (1 or 2 mi) northwest of the site, where the Vincentown sub-crops beneath the Delaware River. In both cases, the groundwater travel distances are much greater than distances through the alluvium. Hydraulic conductivity and gradient data suggest that the groundwater velocity in the Vincentown is about 15 percent faster than in the alluvium; however, this small difference is not enough to make the groundwater travel time through the Vincentown less than through the alluvium. The longer travel times in the Vincentown aquifer lead to greater radioactive decay and therefore lower predicted concentrations. Based on this reasoning, the applicant stated the alluvium was the most conservative release pathway and, therefore, did not address releases to the Vincentown or deeper aquifers and removed SSAR Section 2.4.13.1.9, Revision 0, while placing the discussion in SSAR, Revision 2, Section 2.4.13.1.3.

The staff verified that the travel time to the river was fastest through the alluvium and therefore, the most conservative assumption for a release scenario contamination to the Delaware River. The staff considers the applicant's evaluation reasonable and adequate. Accordingly, the staff considers RAI 31, Question 02.04.13-12 (2) resolved.

In a June 30, 2011, response to RAI 31, Questions 02.04.13-12 (3) and (4), the applicant described the two possible pathways for contaminants to enter the Delaware River from the Vincentown formation. One path is directly to the northwest where the Vincentown outcrops in the Delaware River (Reference 2.4.13-1), and the other is upward through the overlying alluvium. The staff finds this explanation adequate and reasonable. Accordingly, the staff considers Questions 02.04.13-12 (3) and (4) resolved.

In RAI 31, Question 02.04.13-12 (5), the staff requested that the applicant provide information on potential migration pathways into the deeper PRM aquifer system. In a June 30, 2011, response, the applicant explained that the lack of downward hydraulic head and the presence of intervening aquitards would lessen the possibility of downward migration. The applicant stated that SSAR Section 2.4.13.1.9 would be updated but this section was removed and incorporated within SSAR, Revision 1, and subsequently Revision 2, Section 2.4.13.1.3. Accordingly, the staff considers RAI 31, Question 02.04.13-12 (5) resolved.

In RAI 31, Question 02.04.13-12 (6), the staff requested that the applicant provide justification for why dilution of radionuclide concentrations in a pumping well to less than detectable levels is compliant with requirements. In a June 30, 2011 response, the applicant explained that the pumping well discussion was only qualitative and that it would be removed from the SSAR. The staff verified that the discussion was removed from the SSAR, Revision 1, and considers RAI 31, Question 02.04.13-12 (6) resolved.

Based on an independent evaluation of the local hydrogeologic units, lack of deep pumping, and the higher hydraulic conductivity of the alluvium, the staff finds the applicant's responses and discussions to RAI 31, Question 02.04.13-12 in its entirety adequate, and considers all parts of this RAI Question resolved.

2.4.13.4.8 Groundwater Model, Release Calculations, and Slug Test Calculations provided in Digital Format

In RAI 31, Question 02.04.13-13, the staff requested digital files for the dewatering modeling, transport analysis, and slug testing performed by the applicant. On July 25, 2011, the applicant submitted a response to RAI 31, Question 02.04.13-13, along with digital files for groundwater model that refers to the dewatering during construction. The aquifer slug test data needed to obtain hydraulic conductivity values to perform the transport calculations was received on discs and found to be consistent with the aquifer parameters provided by the applicant and those used to calculate results included in Revision 1 of the ESP application. The staff finds the applicant's response adequate and, therefore, considers RAI 31, Question 02.04.13-13 resolved.

2.4.13.5 *Post Early Site Permit Activities*

There are no post ESP activities related to this section.

2.4.13.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant addressed the relevant information and there is no outstanding information required to be addressed in the SSAR related to this section. As set forth above, the applicant has provided sufficient information pertaining to liquid pathways. Therefore, the staff concludes that the applicant has met the requirements related to liquid pathways of 10 CFR 52.17(a) and 10 CFR 100.20(c).

2.4.14 Site Characteristics and Bounding Design Parameters

This section of the report lists site characteristics and bounding design parameters as given in Tables 2.4.14-1 and 2.4.14-2 below that the staff has determined should be included in the ESP that may be granted for the PSEG Site:

Table 2.4.14-1 Proposed Site Characteristics Related to Hydrology

Site Characteristic	PSEG Site Value	Definition
Proposed Facility Boundaries	Figure 2.4.14-1 depicts the proposed facility area boundaries.	PSEG site boundary areas within which all safety-related SSCs will be located.
Maximum Groundwater	3.05 m (10 ft) NAVD88	The maximum elevation of groundwater at the PSEG Site.
Maximum Stillwater Flood Elevation (Storm Surge) + 10% Astronomical High Tide	7.53 m (24.7 ft) NAVD88	The stillwater elevation, without accounting for wind-induced waves, the water surface reaches during a flood event.

Site Characteristic	PSEG Site Value	Definition
Wave Runup (Storm Surge)	2.26 m (7.4 ft) NAVD88	The height of water reached by wind-induced waves running up on the site.
Combined Effects Maximum Flood Elevation (Design Basis Flood)	9.78 m (32.1 ft) NAVD88	The water surface elevation at the point in time where the combination of the still water level and wave runup is at its maximum.
Local Intense Precipitation	46.7 cm (18.4 in.) per hour	The depth of PMP for duration of 1 hour on a 1 square-mile drainage area. The surface water drainage system should be designed for a flood produced by the local intense precipitation.
Frazil, Surface or Anchor Ice	The PSEG Site has the potential for frazil and surface ice.	Potential for accumulated ice formation in a turbulent flow condition.
Minimum River Water Surface Elevation	-4.85 m (-15.9 ft) NAVD88 for less than 6 hours	The river surface water elevation and duration for which the low water level conditions exist at the PSEG Site.
Maximum Ice Thickness	45.2 cm (17.8 in.)	Maximum potential ice thickness on the Delaware River at the PSEG Site.
Hydraulic Conductivity	SSAR Table 2.4.12-9	Groundwater flow rate per unit hydraulic gradient.
Hydraulic Gradient	SSAR Tables 2.4.12-7 and 2.4.12-8	Slope of groundwater surface under unconfined conditions or slope of hydraulic pressure head under confined conditions.

Table 2.4.14-2 Bounding Design Parameters

Bounding Design Parameter	Value	Definition
Site Grade	11.25 m (36.9 ft) NAVD88	Finished plant grade for the power block area on the PSEG Site.

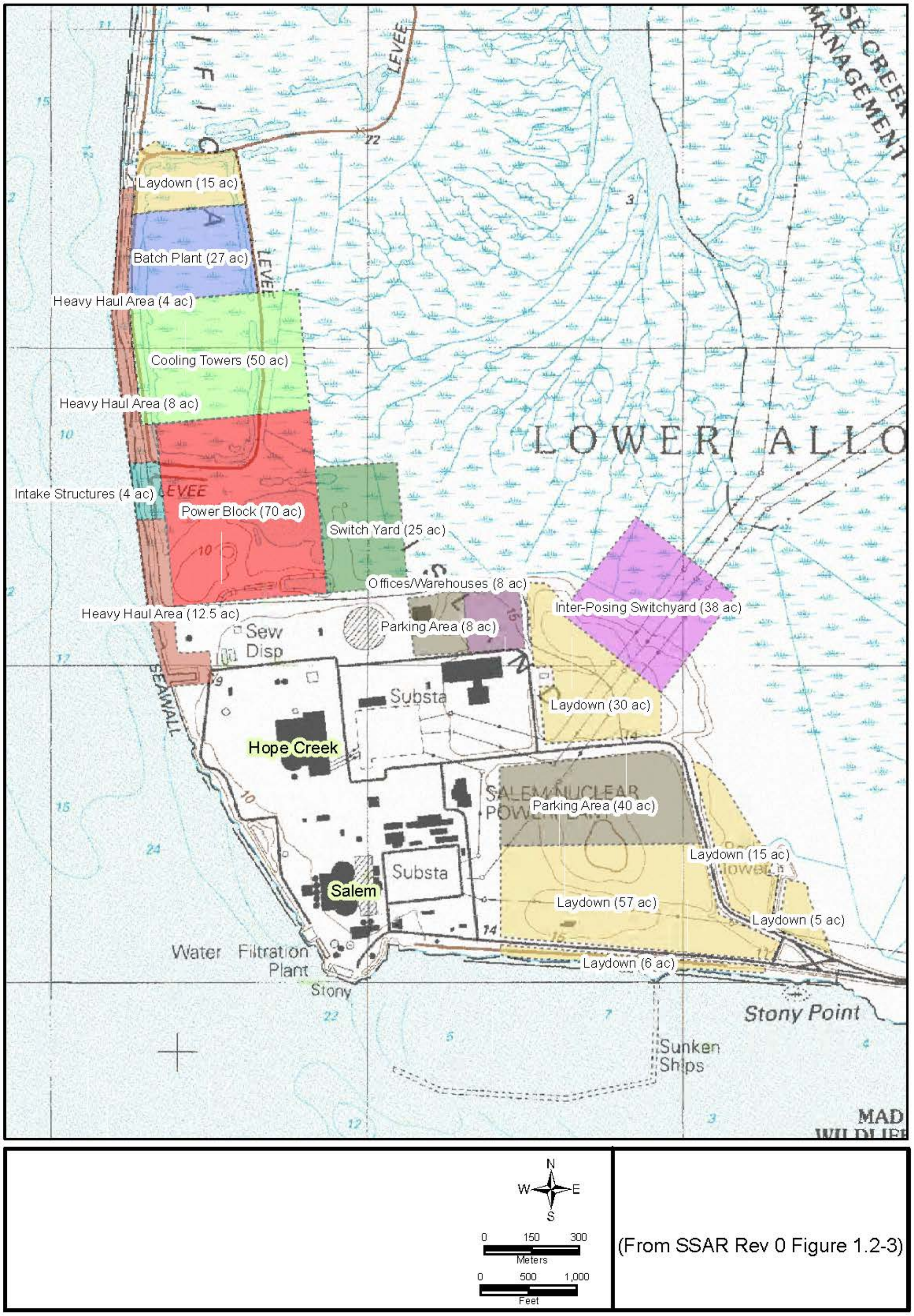

Figure 2.4.14-1 Proposed PSEG Site Layout (based on SSAR Revision 3, Figure 1.2-3)

When referenced by a COL applicant pursuant to 10 CFR 52.73, “Relationship to Subparts A and B,” this ESP is subject to these COL action items:

<u>COL Action Items 2.4-1 through 2.4-3</u>

2.4-1 An applicant for a combined license (COL) or construction permit (CP) that references this early site permit should design the site grading to provide flooding protection to safety- related structures at the ESP site based on a comprehensive flood water routing analysis for a local PMP event without relying on any active surface drainage systems that may be blocked during this event. (See Section 2.4.2.4 of this report.)

2.4-2 An applicant for a combined license (COL) or construction permit (CP) that references this early site permit should address whether the intake structure of the selected design is a safety-related SSC. If so, the applicant should address necessary flooding protection for a safety-related intake structure at the ESP site based on the design basis flooding event and associated effects. (See Section 2.4.10.4 of this report.)

2.4-3 An applicant for a combined license (COL) or construction permit (CP) that references this early site permit should refine hydrogeologic parameters and model estimates of dewatering rates and drawdowns beneath existing site structures after determination of the final excavation geometry consistent with a selected reactor technology. (See Section 2.4.12.4 of this report.)