Technical Evaluation Report
Salem Generating Station
Storm Surge Hazard Reevaluation Review

Prepared for

U.S.NRC

By

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TECHNICAL EVALUATION REPORT

STORM SURGE

The licensee reported in the FHRR that the reevaluated hazard, including associated effects, for site flooding due to storm surge is 22.5 ft-NAVD88 for stillwater and 29.9 ft-NAVD88 for total water level at the auxiliary building. At the intake structure, the licensee reported in the FHRR that the reevaluated hazard, including associated effects, for site flooding due to storm surge is 22.5 ft-NAVD88 for stillwater and 36.3 ft-NAVD88 for total water level. This flood-causing mechanism is described in the licensee's current design basis. The current design basis hazard for site flooding due to storm surge is 24 ft-NAVD88 for stillwater and 30.6 ft-NAVD88 for total water level at the auxiliary building. At the intake structure, the current design basis hazard for site flooding due to storm surge is 24 ft-NAVD88 for stillwater and 37.5 ft-NAVD88 for total water level

To provide additional information in support of the summaries and conclusions in the FHRR, the licensee made certain calculation packages available to the staff via an electronic reading room (ERR). The staff did not rely directly on this calculation package in its review; it was found only to expand upon and clarify the information provided in the FHRR, and so those calculation packages were not docketed or cited.

In connection with the staff's FHRR review, electronic copies of the computer input/output (I/O) files used in the numerical modeling were provided to the staff and cited as part of the Nuclear Regulatory Commission Report for the Audit of PSEG Nuclear LLC's Flood Hazard Revaluation Report Submittals Relating to the Near-Term Task Force Recommendation 2.1-Flooding for Salem Nuclear Generating Station Units 1 and 2 (NRC, 2016) [ML15364A073].

The staff describes its evaluation of site flooding from storm surge, including associated effects, against the relevant regulatory criteria based on present-day methodologies and regulatory guidance below.

1.0 Historical Storm Surge Data.

Information Submitted by the Licensee

Table 1 (FHRR Table 2.4-10) provides a list of historical storms with headings towards the PSEG Salem Generating Station (SGS) site. The list includes storms passing over a line from a point at latitude 36.5 degrees north, longitude 76 degrees east to a point at latitude 41.5 degrees north, 71 degrees east (Figure 1, FHRR Figure 2.4-11). Table 1 includes important meteorological parameters for 15 storms dating back to 1869 and includes Hurricane Sandy (2012). FHRR Section 2.4.3.4.1 describes how the licensee applied the data from the 15 historical storms to develop the complimentary probability distribution of storm central pressure applied by the study.

Table 1. Historical storm headings towards PSEG SGS site (FHRR Table 2.4-10)

Central Pressure (mb)	Longitude Deg. East	Latitude Deg. North	Forward Velocity (kt)	Track Angle (deg)[b]	Year	Name
963	71.9	41.0	44.4	67.9	1869	Unnamed
984	73.4	39.5	27.2	41.9	1879	Unnamed
990	72.5	41.0	22.8	49.1	1916	Unnamed
941	72.9	40.7	14.0	90.0	1938	Unnamed
966	71.5	42.1	30.1	57.1	1944	Unnamed
976	71.8	43.1	30.1	67.5	1954	Carol
969	75.9	36.6	10.0	84.3	1955	Connie
980	73.4	40.2	20.1	98.5	1972	Agnes
977	73.8	38.8	22.5	77.2	1976	Belle
951	74.5	38.4	41.0	67.5	1985	Gloria
990	75.2	37.4	14.8	46.2	1986	Charley
964	71.4	41.4	27.3	55.3	1991	Bob
980	73.5	40.6	29.4	55.3	1999	Floyd
958	75.0	38.1	15.0	63.4	2011	Irene
943	74.0	38.8	8.0	148.0	2012	Sandy

a) Storms passing over a Line from a point at Latitude 36.5 degrees North, Longitude 76 degrees East to a point at Latitude 41.5 degrees North, 71 degrees East (see Figure 2.4-11).
b) Track Angle convention is 0 degrees denotes a storm moving east, 90 degrees denotes a storm moving north and 180 degrees denotes a storm moving west.
c) References 2.4-2 and 2.4-16.

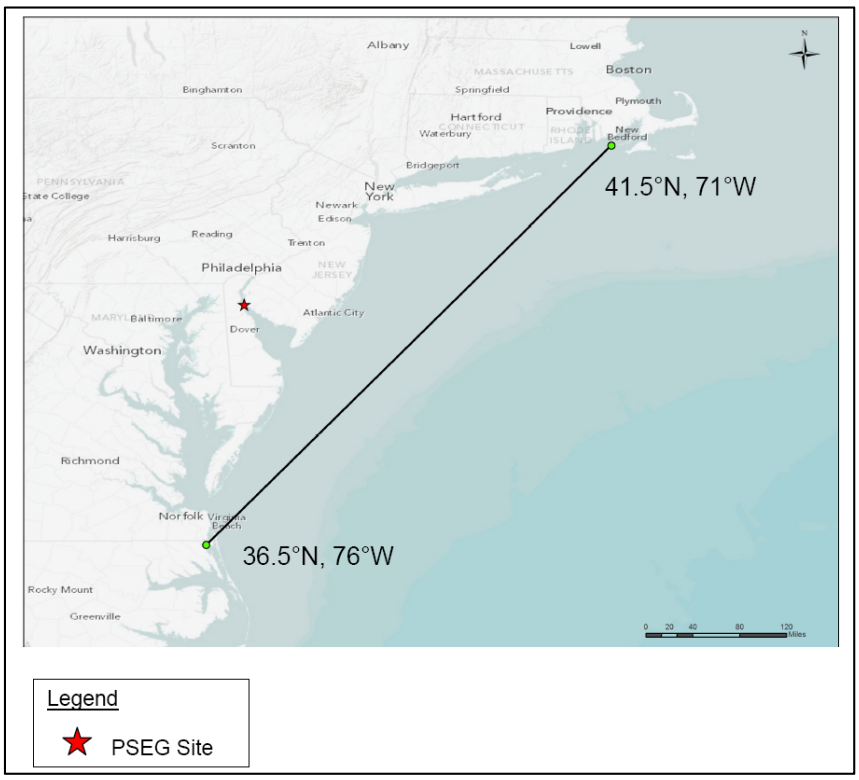

Figure 1. Demarcation line for historical storm central pressure analysis (FHRR Figure 2.4-11)
The *Response to RAI Regarding Flooding Hazard Reevaluation — Question 5* document submitted by PSEG on September 23, 2014, contains a revised demarcation line and storm set based on the revised line. The revised demarcation line captures storms passing over a line

from a point at latitude 37 degrees north, longitude 76 degrees west to a point at latitude 41 degrees north, 72 degrees west (Figure 2, FHRR Figure RAI-5-2). Table 2 (FHRR Table RAI 5-2) contains the list of 14 storms which entered into the defined area and had a minimum central pressure less than or equal to 980 mb. The angle in the table follows a standard mathematical convention with zero degrees denoting a storm moving east and 90 degrees denoting a storm moving north.

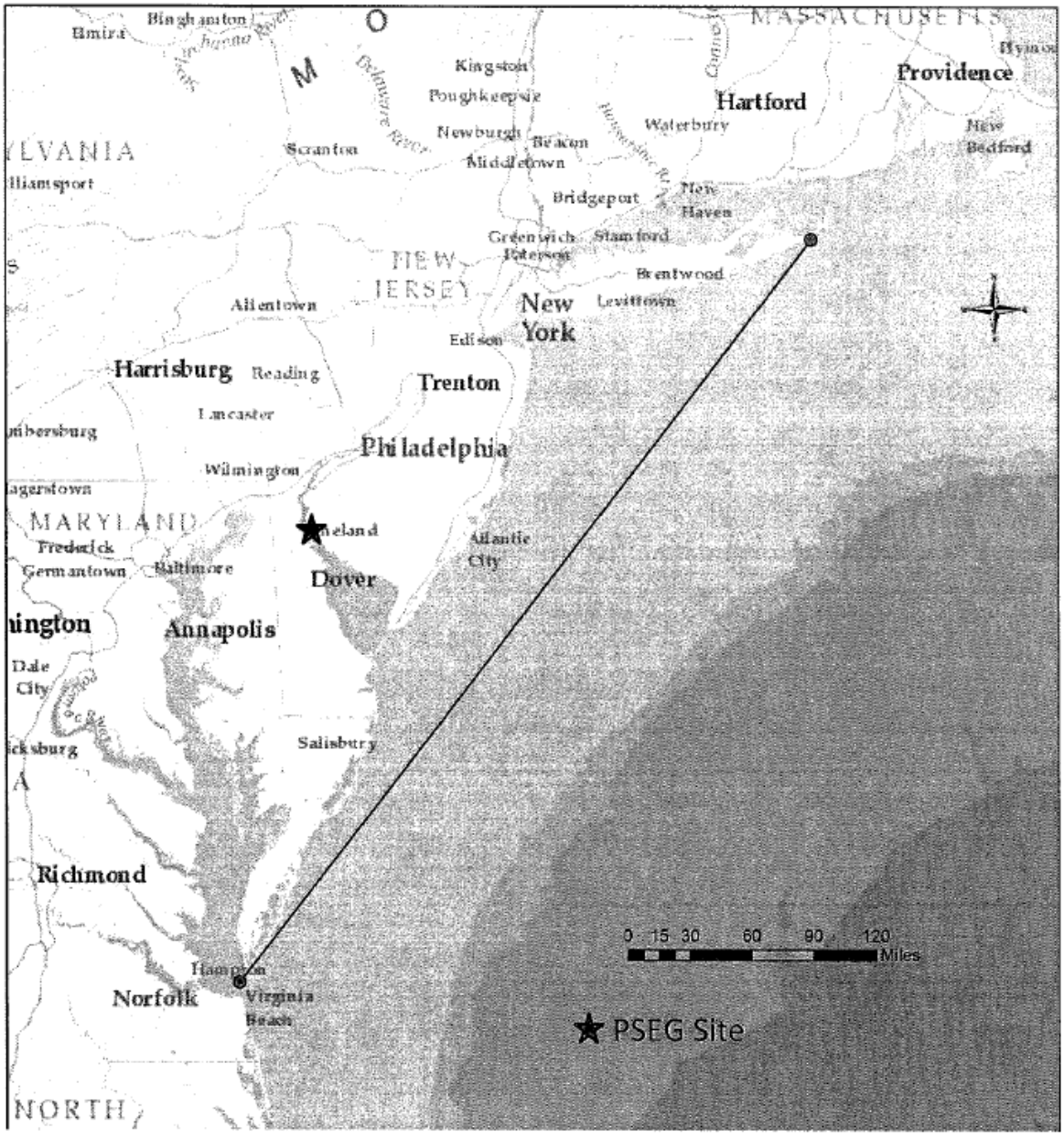

Figure 2. Revised demarcation line for historical storm central pressure analysis (FHRR Figure RAI-5-2)

Table 2. Revised list of storms and characteristics at closest location to landfall that crossed demarcation line in Figure 2 (FHRR Figure RAI-5-2) [FHRR Table RAI-5-2]

Year	Name	Central Pressure (mb)	Heading (deg)	Forward Speed (knots)
1867	Unnamed	969	61.9	16.3
1869	Unnamed	950	79.6	38.4
1879	Unnamed	979	63.4	26.9
1936	Unnamed	968	90.0	13.0
1938	Unnamed	940	87.2	41.0
1958	Daisy	970	58.6	20.0
1960	Donna	965	47.6	30.0
1972	Agnes	977	47.3	20.1
1976	Belle	977	79.7	22.2
1985	Gloria	951	71.0	30.0
1991	Bob	953	58.0	26.7
1999	Floyd	974	56.9	25.9
2011	Irene	958	63.4	13.8
2012	Sandy	940	127.5	6.7

Staff Technical Evaluation

The storm list in Table 1(FHRR Table 2.4-10) provides pertinent information for storms with headings towards the PSEG SGS site. The transect applied for the analysis (Figure 1, FHRR Figure 2.4-11) provides a reasonable capture area given the PSEG SGS site location. The capture area applied covers storms with various headings that could cause significant storm surge near the PSEG SGS site. The storms in Table 1 and Figure 1 provide the basis for the FHRR analysis.

Within the response to RAI Question 5, PSEG provides the storm listing and demarcation line in Table 2 and Figure 2. As part of the response to RAI Question 5, PSEG completes a revised probabilistic-deterministic storm surge analysis based on the revised storm set and revisions to other analysis parameters. The response to RAI Question 5 states that revised analysis produces a lower 10^{-6} Annual Exceedance Probability (AEP) storm surge water elevation. Thus, the revised analysis indicates that modifications to the storm suite do not increase the estimated 10^{-6} AEP storm surge water elevation at the site.

Staff notes that hurricanes Carol (1954) and Connie (1955) were on the original FHRR storm list, but are not on the RAI response storm list. Hurricane Donna, was not on the FHRR storm list, but is on the RAI response storm list.

2.0 Probable Maximum Hurricane.

Information Submitted by the Licensee
FHRR Section 1.2.4 describes the methodology applied to develop the Probable Maximum Hurricane (PMH) within the current licensing basis (CLB) storm surge level analysis. The CLB methodology applied bathystropic storm tide theory described by Marinos and Woodward (FHRR Reference 1-2) with the hurricane surge computed at the mouth of Delaware Bay. The surge was propagated up Delaware Bay in accordance with a method described by Bretschneider (FHRR Reference 1-1).

Components of the stillwater level included:
1) Mean low water depth
2) Astronomical tide
3) Rise in water level resulting from the hurricane's atmospheric pressure reduction
4) Wind stress component perpendicular to the bottom contours (onshore wind components)
5) Wind stress component parallel to the bottom contours which produces a longshore flow deflected to the right (in the northern hemisphere) by the Coriolis forces
6) Initial surge (a slow general rise in sea level existing before the actual hurricane winds arrive)

In the CLB calculations, HUR 7-97 (U.S. Dept of Commerce, 1968; FHRR Reference 1-5) provided the storm parameters used to calculate maximum storm surge. HUR 7-97 grouped storm parameter values according to defined coastal zones and by latitude within each zone. As stated in the FHRR, HUR 7-97 produced the parameters and characteristics based on empirical observations, assumptions, and experience.

The PMH used in the CLB storm surge analyses featured a large radius, moderate forward speed hurricane which generated the maximum surge on the open coast. The PMH quantitative meteorological parameters were
- Central pressure: 27.09 inches Hg (917.4 millibar (mb))
- Peripheral pressure: 30.72 inches Hg (1040.3 mb)
- Radius of maximum winds: 39 nautical miles (nmi) (72.2 kilometers (km))
- Maximum wind speed: 132 miles per hour (MPH) (212.4 kilometers/hour (km/hr))
- Forward speed: 27 knots (kts) (50.0 km/hr)

The FHRR analysis applies a deterministic-probabilistic approach that does not specifically develop storm parameters for a PMH storm. The deterministic-probabilistic approach develops a suite of storms designed to produce the water level associated with various return periods.

Staff Technical Evaluation
The FHRR does not develop parameters values to define a specific PMH. Review of the deterministic-probabilistic approach will include comments on the appropriateness of the values applied in FHRR storm suite.

For reference, the recently submitted PSEG Early Site Permit (ESP) application (PSEG, 2010; PSEG 2015; NRC, 2015) developed and applied a PMH storm for a deterministic storm surge analysis. The NWS 23 report (NOAA, 1979) provided the basis for the PSEG ESP application values. The PSEG ESP application PMH storm featured the following meteorological parameters:

- Central pressure: 26.65 inches Hg (902 mb)
- Peripheral pressure: 30.15 inches Hg (1021 mb)
- Radius of maximum winds: 28 nmi (51.9 km)
- Forward speed: 26 kts (48.2 km/hr)

Compared to the CLB storm parameters, the PSEG ESP application PMH storm parameter values show a relatively similar pressure drop and forward velocity, a smaller radius to maximum winds, and a higher maximum wind speed. Overall, given the different sources of information applied to develop the CLB PMH and the PSEG ESP application PMH, the storm parameters are relatively similar.

3.0 Probable Maximum Wind Storm.

Information Submitted by the Licensee
Section 2.4.7 of the FHRR states the storm events analyzed in developing the hurricane-induced storm surge bound the Probable Maximum Wind Storm (PMWS) that could cause flooding at the PSEG SGS site. For the FHRR, a 31-year record (1978 through 2008) of wind speed and direction data from Dover, DE (11 mi (17.7 km) west of the center of Delaware Bay) was analyzed. The licensee opines the Dover weather station is the closest to the center of Delaware Bay, and thus provides the most appropriate evaluation location. For the FHRR analysis, winds at Dover were averaged over four hours, which the FHRR states provides a sufficient duration to cause wind setup of Delaware Bay. Analysis of historical records shows that four hour average winds parallel to the long axis of Delaware Bay did not exceed 35 mph (30 kts, 55.6 km/hr)) at Dover. Over-water winds are expected to be 50 kts (92.6 km/hr) when overland winds are 30 kts (55.6 km/hr) (FHRR Reference 2.4-20). Based on this, the FHRR states that winds of sufficient duration to cause wind setup or seiche did not exceed 50 kts (92.6 km/hr) over Delaware Bay during the period 1978 through 2008. By comparison, the one hour wind speeds associated with the storm events analyzed in developing the storm surge are approximately 80 kts (148 km/hr) near the PSEG SGS site. Therefore, the FHRR concludes that the hurricane-induced storm surge exceeds any potential surge associated with a PMWS for the PSEG SGS site.

Staff Technical Evaluation
NRC staff evaluated the ESP applicant's analyses completed for the PMWS calculations as presented in FHRR Section 2.4.7. The assessment proves reasonable and the results align with the Dover data. The review verified the relevant proximity of Dover to the proposed project site. Based on the analyses, NRC staff agrees that the PMWS forcing at the site does not approach the extreme forcing generated by the PMH event.

4.0 Antecedent Water Levels.

Information Submitted by the Licensee
In the simulation of FHRR production storms for the storm surge analysis, the ADCIRC +SWAN model was run with a constant tide set at 0 ft-NAVD88 (FHRR Section 2.4.3.2).

The FHRR states that tidal effects were included in a probabilistic manner using linear superposition. To create a synthesized tidal record at the site, the astronomic tide signal was obtained from the ADCIRC+SWAN model by simulating a tides-only condition. The licensee applied the validated ADCIRC+SWAN model to develop the tidal constituents from a 45-day

period within a 90-day simulation. The ADCIRC+SWAN model allowed for calculation of the important tidal constituents at the PSEG SGS site. Analysis of the tidal constituents allowed calculation of tidal levels at the site for a 14-day period that included a full Spring-Neap tidal cycle at the site. Table 3 (FHRR Table 2.4-4) presents the mean displacements and standard deviations associated with five different surge levels.

Table 3. Mean displacement and standard deviation of tidal effects (FHRR Table 2.4-4) — Conversion from meters to feet is accomplished by multiplying by a factor of 3.2808

Surge Value (m)	Mean Displacement (m)	Standard Deviation (m)
5	0.22	0.47
6	0.18	0.49
7	0.16	0.50
8	0.14	0.51
9	0.13	0.51

To probabilistically evaluate the effect of tides on the 10^{-6} AEP storm surge water elevation, a mean tidal displacement of 0.59 ft (0.18 m) was added to each storm surge result and a standard deviation of 1.6 ft (0.49 m) was added to the epistemic uncertainty term. (FHRR Sections 2.4.3.1.2.1.4 and 2.4.3.2)

Sea level rise was set at 0.5 ft (0.15 m) for the 32 years remaining on the longest operating license for the three units at this site. The 0.5 ft (0.15 m) was based on the sea level trend at the NOAA Reedy Point tidal gage. The trend is an increase of 1.14 ft (0.35 m) in 100 years with an upper limit 95% confidence limit of 1.35 ft (0.41 m) in 100 years. (FHRR Section 2.4.4)

Staff Technical Evaluation

The methodology applied within the production runs allows the addition of a mean tide level to each storm surge simulation and application of a tide-based epistemic uncertainty term within the JPM-OS integral. FHRR Table 2.4-4 presents the mean displacements and standard deviations associated with five different surge levels.

The PSEG ESP application (PSEG, 2010; PSEG 2015; NRC, 2015) presents results from ADCIRC+SWAN sensitivity simulations designed to evaluate the effect of applying linear superposition for tides and sea level rise (SLR) versus including the tide and SLR as an initial water level adjustment in ADCIRC+SWAN. The sensitivity analysis results indicate that adding the 10% annual exceedance high tide and the SLR to the initial water level can increase the maximum water level, including wave runup, by about 1 ft (0.31 m). However, the PSEG ESP application notes that increasing the initial water surface in the entire model is not physically realistic and develops a conservative estimate of how adding the tide alters the maximum surge level. Given the sensitivity analysis results and the conservative nature of increasing the water level everywhere in the model domain, NRC staff agrees that applying linear superposition for the tide and SLR change provides a reasonable procedure to evaluate the maximum water level across the various ADCIRC+SWAN simulations.

FHRR Figure 1.1-9 indicates a 10% exceedance high tide level of 1.8 m (4.5 ft) at the PSEG SGS site. However, the probabilistic analysis applied in the FHRR storm surge evaluation

applies a different, probabilistic-based, approach that does not require the 10% exceedance high tide level to include conservatism in the timing of storm surge and high tide.

JLD-ISG-2012006 recommends using the 10% exceedance high tide, calculated sea level anomaly, and the expected SLR in the calculation of antecedent water levels. The licensee used a probabilistic method rather than the method in the Interim Staff Guidance (ISG). Staff concurs with the SLR of 0.5 ft (0.15 m) based on the remaining time in the plant operating licenses.

5.0 Storm Surge Models

5.1 Surge Propagation Models

Information Submitted by the Licensee
The FHRR describes that a computer program developed by Dames and Moore provided the CLB PMH analysis tool — described by Marinos and Woodward (FHRR Reference 1-2). Input data to the computer program describing the storm and the bathymetric conditions included the basic parameters of the hurricane, an initial surge of one foot, wind friction factor, bottom friction factor (0.008), wind speed at various radial distances and angles of wind direction relative to the translational velocity vector of the hurricane, bathymetric traverse data, and astronomical tide (5.6 ft).

As part of the CLB PMH storm surge calculation, the computed maximum surge elevation at the mouth of the Delaware Bay was 21.9 ft (6.7 m) above mean low water (MLW) — equal to 19.7 ft (6.0 m) Mean Sea Level (MSL). This surge included the effects of the astronomical high spring tide. The model surge hydrographs for Delaware Bay computed were then used to determine hurricane surge values at the SGS site as a function of time. The maximum stillwater elevation at the site was determined using a combination of the storm surge and the crosswind setup or drawdown for the six fetches chosen.

Independent surge estimates were developed for the SGS Safety Evaluation Report (SER) by NRC consultants. The NRC consultant estimated maximum stillwater was at elevation 113.8 ft Public Service Electric and Gas Company datum (PSD) (24.8 ft MSL [7.6 m MSL]), considering the complete range of PMH parameters coincident with the local high spring tide. The site hydrologic design parameters were developed using the recommended value — notably, the complete range of parameters and the source of the recommended values is not provided in the discussion.

The CLB PMH was located so as to produce maximum waves. In the vicinity of the site, the PMH winds had a maximum sustained wind velocity of 85 mph from the southeast. With the surge level at 113.8 ft PSD (24.8 ft MSL [7.6 m MSL]), wave runup elevations on safety-related structures inside the sea wall were calculated to be a maximum of 120.4 ft PSD (31.4 ft MSL) [9.6 m MSL]). Maximum wave run up elevation on the service water intake structure was calculated to be 127.3 ft PSD (38.3 ft MSL [11.7 m MSL]).

FHRR Section 2.4 describes the process to reevaluate the storm surge at the PSEG SGS site. The FHRR states the process to evaluate the storm surge combined a state-of-the-art high resolution numerical modeling platform with the Joint Probability Method – Optimal Sampling (JPM–OS) method. The high resolution numerical modeling platform applied the FEMA Region

III storm surge study model mesh and the ADCIRC+SWAN fully-coupled model (Figure 3, FHRR Figure 2.4-1) (USACE 2011b, 2013). ADCIRC is based on the two-dimensional, vertically integrated shallow water equations that are solved in Generalized Wave Continuity Equation form (UNC, 2015). The equations are solved over complicated bathymetry encompassed by irregular seashore boundaries using an unstructured finite-element method.

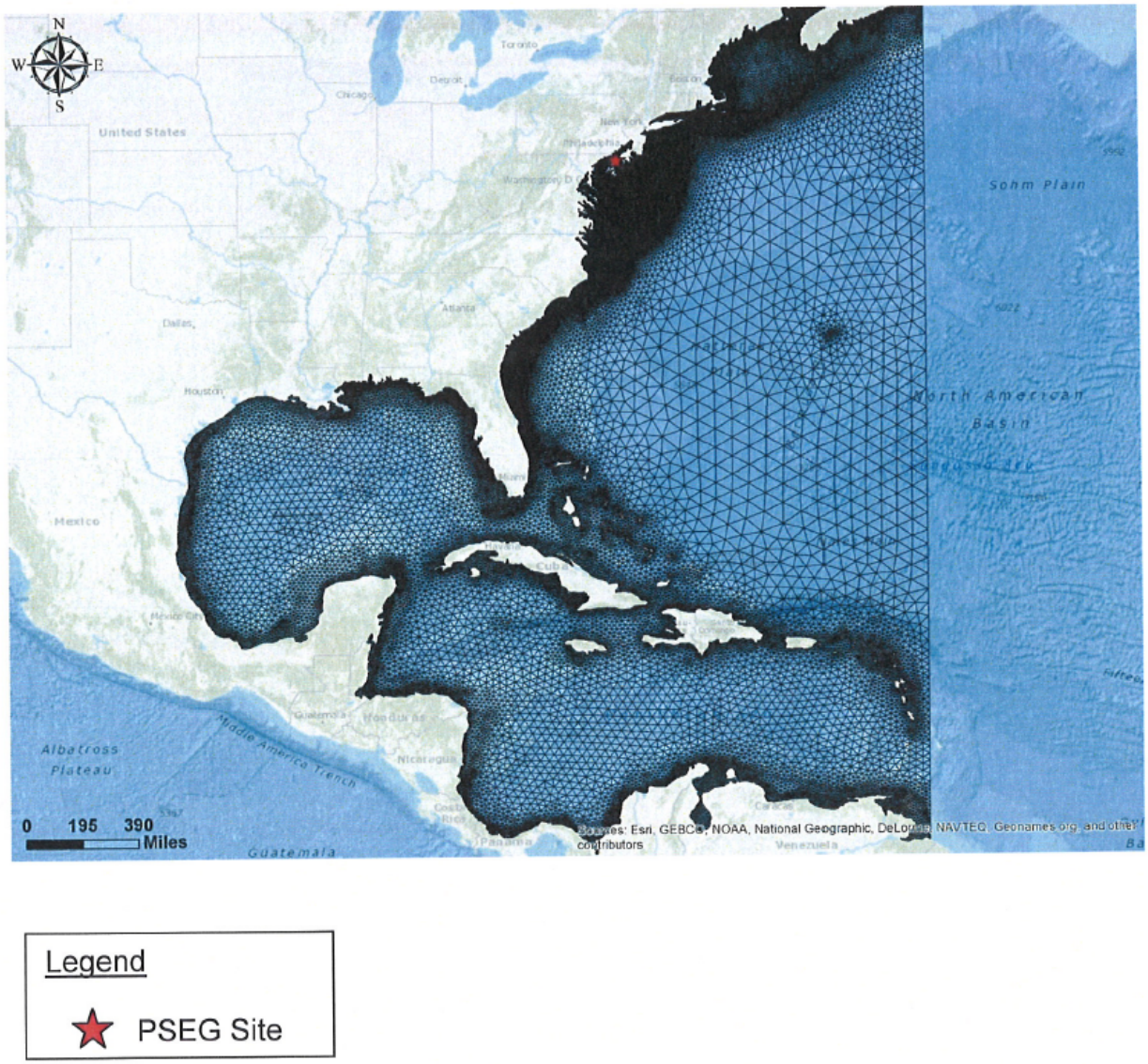

Figure 3. Entire ADCIRC+SWAN mesh (FHRR Figure 2.4-1)

The recent FEMA Region III storm surge study developed a high-resolution ADCIRC mesh that covers the entire Delaware Bay and PSEG SGS site region (Figure 3). The ADCIRC mesh consists of nodes that create a high-resolution grid covering FEMA Region III. The FEMA Region III mesh was appended to a previously developed grid of the western North Atlantic, the Gulf of Mexico, and the Caribbean Sea. Specifically, the mesh covers the area from the 60 degrees west meridian to the U.S. mainland. Within FEMA Region III, the mesh extends inland to the 49.2 ft-NAVD88 (15 m) contour to allow for inland storm surge flooding. In the Region III

study area, the mesh was designed to resolve major bathymetric and topographic features such as: inlets, dunes, and river courses, as identifiable on the detailed FEMA digital elevation model (DEM), satellite images, and NOAA charts. (USACE 2011a and 2011b).

The model mesh featured a refined mesh for the PSEG SGS area (Figure 4, FHRR Figure 2.4-2). After inserting the refined model mesh section into the overall Region III mesh, the model was revalidated with comparison of results to measurements from Hurricane Isabel. The simulation with the refined mesh shows results that agree well with results from the unmodified FEMA Region III mesh.

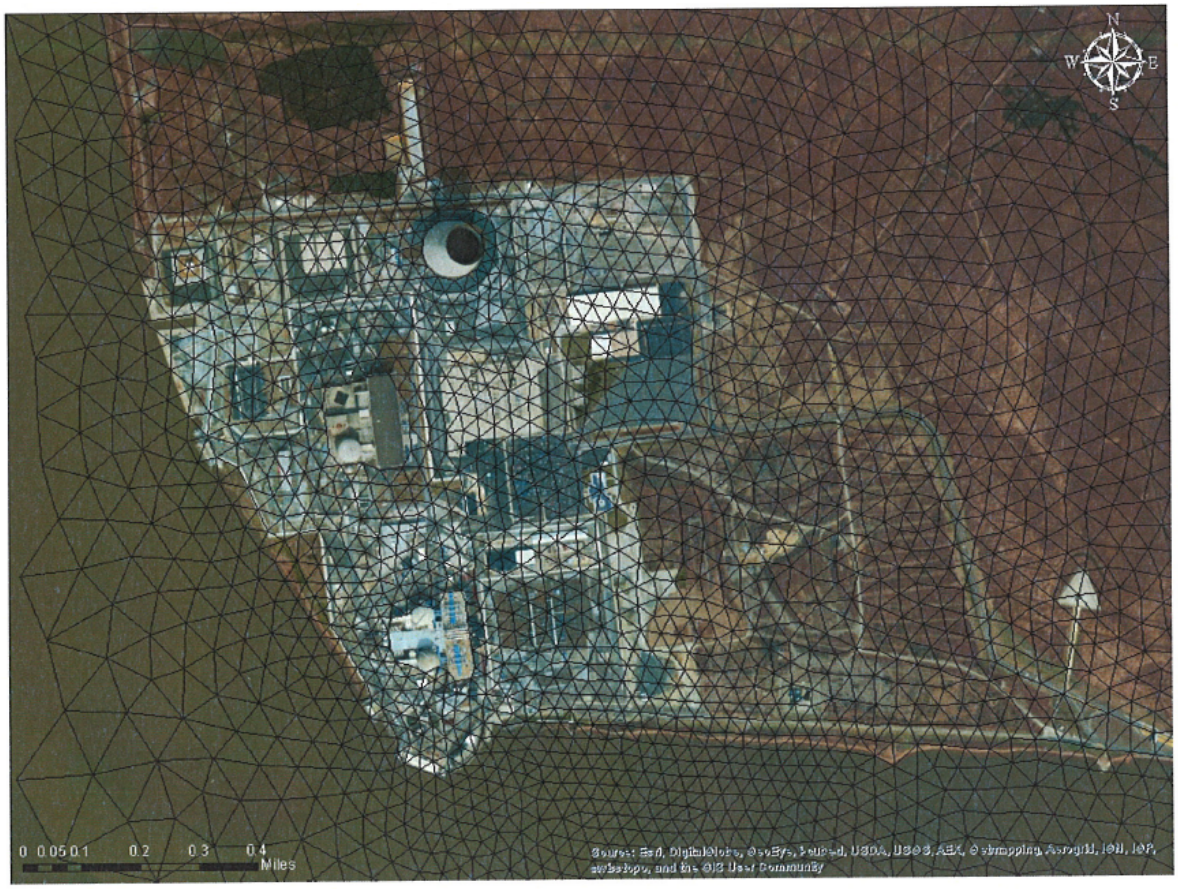

Figure 4. Refined ADCIRC+SWAN mesh near the PSEG SGS site (FHRR Figure 2.4-2)

Staff Technical Evaluation

The FHRR analysis applies the FEMA Region III ADCIRC+SWAN mesh (USACE 2011a, 2011b, 2013) with some additional detail near the PSEG SGS site. The ADCIRC and coupled ADCIRC+SWAN models have been used extensively in recent FEMA coastal storm surge studies and the models provide a robust and state-of-the-art platform to develop estimates of coastal storm surge levels. FEMA requires DEM and model mesh products to apply the best available topographic and bathymetry data and undergo several layers of review. Given the efforts expended by FEMA Region III to develop the comprehensive mesh and the goal of providing a platform to allow evaluation of storm surge caused by powerful storms, the Region III mesh provides a good platform for the FHRR analysis. The mesh covers the PSEG SGS site and surrounding area to adequately simulate surge from very intense storms that may make

landfall near the site or bypass the New Jersey coast. The mesh extends inland a sufficient distance — 49.2 ft-NAVD88 (15 m) contour — to not induce boundary effects during the most powerful storms. It is recognized that the FHRR evaluation considers less frequent (more powerful) storms than the FEMA study; however, application of inland boundary at the 49.2 ft (15 m) contour should prove appropriate for even the most powerful FHRR storms.

The FEMA Region III study had a regional scope and developed a mesh with resolution limits appropriate for the regional view. By adding resolution near the SGS site, the FHRR analysis improves the ADCIRC+SWAN models capability to resolve important local features. Review of the FHRR mesh features in the modified area and comparisons to the Region III mesh shows proper implementation of the increased mesh resolution. Figure 5 shows a comparison of the FEMA Region III mesh resolution and the PSEG FHRR mesh resolution near the PSEG SGS site. The figure shows the level of increased resolution near the PSEG SGS site and the increased level of detail in the topographic features allowed by the increased resolution.

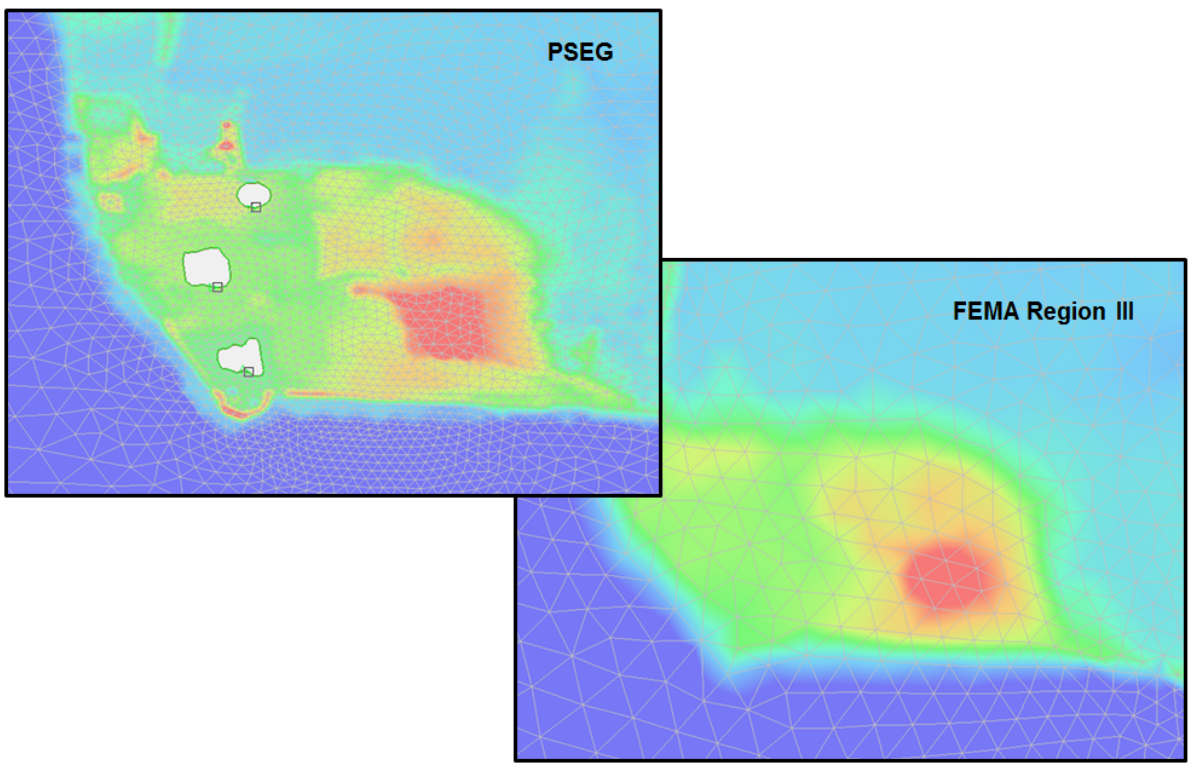

Figure 5. Comparison of FEMA Region III Mesh and PSEG FHRR Mesh near the PSEG SGS site

Review of the PSEG FHRR ADCIRC+SWAN mesh shows the model provides a highly-resolved model grid that applies the FEMA Region III model mesh — built with the best-available bathymetry and topographic data.

5.2 Wave Models

Information Submitted by the Licensee
To develop a detailed analysis of wave conditions on the PSEG SGS site, a high resolution nested SWAN model was applied. The nested SWAN model featured a very high-resolution

mesh with approximately 3 m (10 ft) spacing (Figure 6, FHRR Figure 2.4-12). Figure 7 (FHRR Figure 2.4-13) shows the boundary between the nested SWAN grid and the ADCIRC+SWAN mesh. The figure also shows comparisons of the ADCIRC+SWAN and nested SWAN wave results with good agreement shown for the significant wave height time series plotted at four locations.

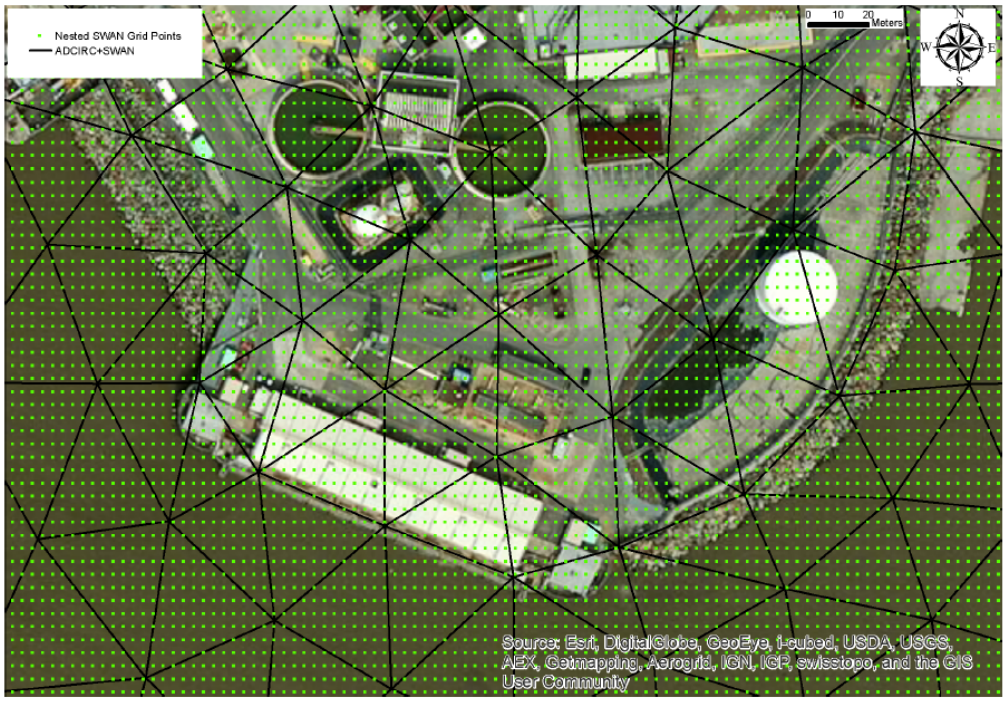

Figure 6. Nested SWAN grid resolution comparison (FHRR Figure 2.4-12)

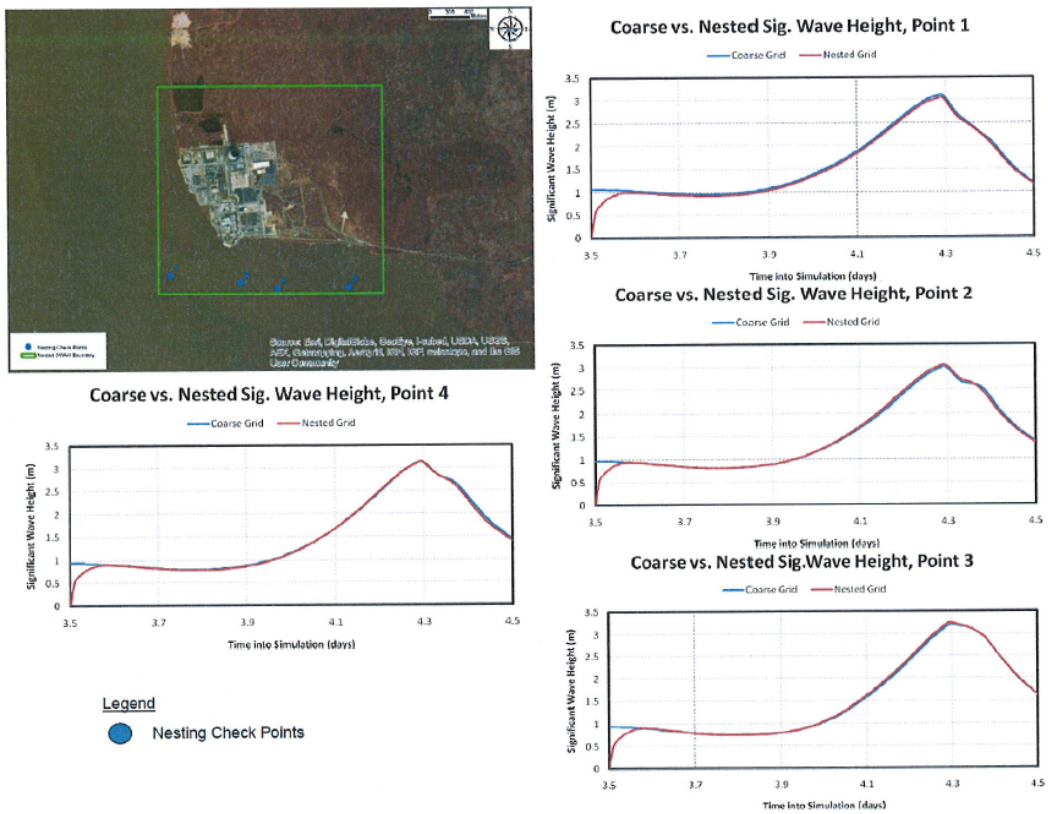

Figure 7. Nested SWAN grid extents and model result comparisons (FHRR Figure 2.4-13)

The wave parameters from the coupled ADCIRC+SWAN model provide the boundary condition parameters applied by the nested SWAN model. The coupled ADCRIC+SWAN model was validated to available wave data during the FEMA Region III study.

Staff Technical Evaluation

The FHRR analysis applies the FEMA Region III ADCIRC+SWAN mesh (USACE 2011a, 2011b, 2013) with a nested SWAN model for additional detail near the PSEG SGS site. The SWAN and coupled ADCIRC+SWAN models have been used extensively in recent FEMA coastal storm surge studies and the models provide a robust and state-of-the-art platform to develop estimates of offshore and nearshore waves. Inclusion of the nested SWAN model (Figures 6 and 7) allows for an additional level of site-specific modeling detail not found within the FEMA coastal surge study that featured a regional perspective. Figure 7 shows that the nested model produces nearly identical results to the coupled ADCIRC+SWAN model for the wave height at selected output locations. The FHRR does not present details of the how the SWAN wave periods compare for the nested model and the coupled ADCIRC+SWAN mesh.

5.3 Topography and Bathymetry

Information Submitted by the Licensee

As stated in TER Section 5.2, the FHRR analysis applied the ADCIRC+SWAN model developed and applied in the recent FEMA Region III storm surge study — which included Delaware Bay, Chesapeake Bay, Delaware-Maryland-Virginia Eastern Shore, Virginia Beach, and all tidal tributaries and waterways connected to these systems. The FEMA Region III study developed a high-resolution ADCIRC mesh that covers the entire Delaware Bay and PSEG SGS site region

(Figure 3) with the mesh extending inland to the 49.2 ft-NAVD88 (15 m) contour. In the Region III study area, the grid was designed to resolve major bathymetric and topographic features such as: inlets, dunes, and river courses, as identifiable on the detailed FEMA digital elevation model (DEM), satellite images, and NOAA charts. (References 2.4-30 and 2.4-31).

The PSEG FHRR study refined the ADCIRC+SWAN mesh in the vicinity of the PSEG SGS site (Figures 4 and 5). Section 2.4.2.2 of the FHRR states that "To properly describe the topographic features important to the hydrodynamic and wave characteristics at the PSEG SGS site, high resolution, site-specific topographic data including the controlling vertical features important to surge conveyance and wave propagation were incorporated into the finite element mesh." The ADCIRC+SWAN model with the increased resolution near the PSEG SGS site was revalidated by comparing water surface levels from Hurricane Isabel and Nor'easter Ida to the original FEMA Region III mesh results. The comparison showed nearly identical results for the refined and original mesh files (Figure 8, FHRR Figure 2.4-3).

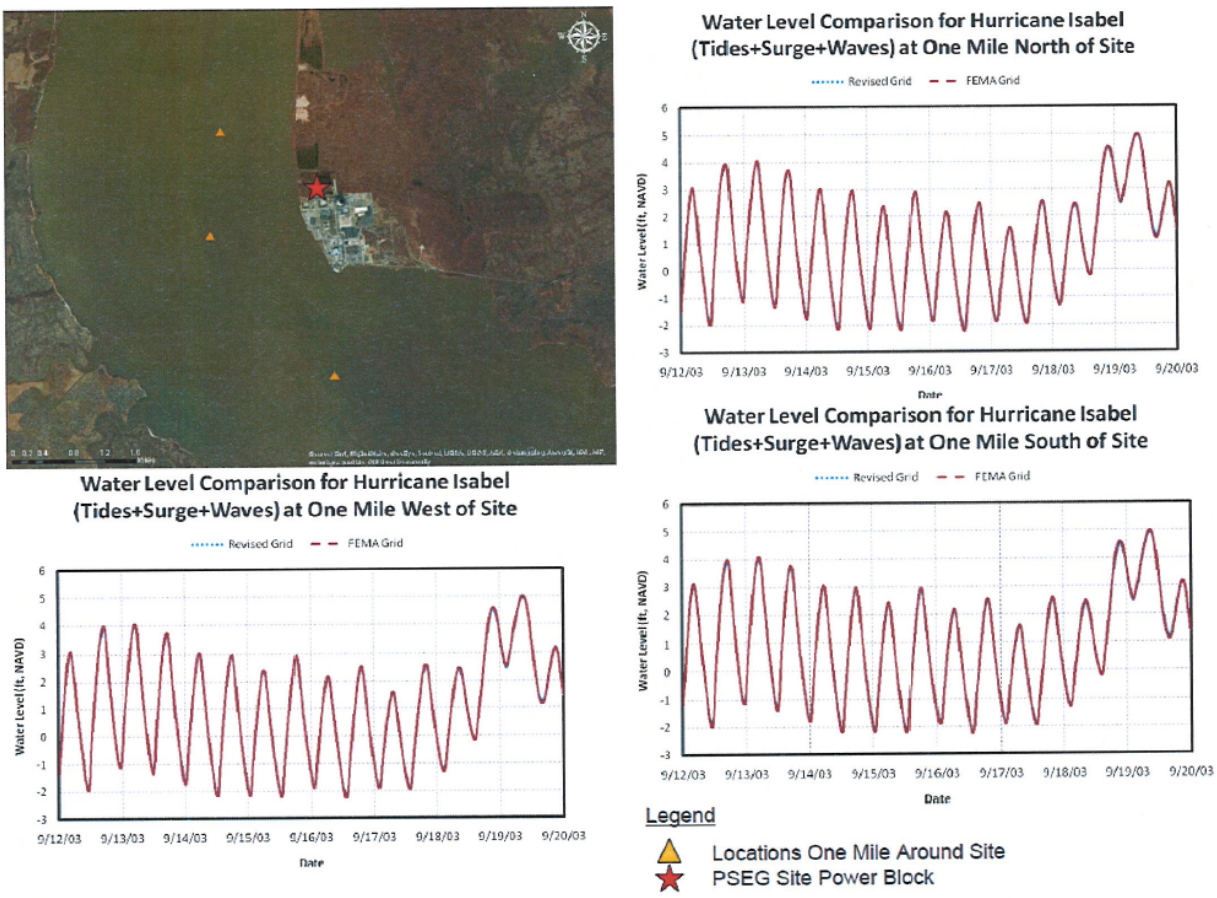

Figure 8. Comparison of refined PSEG SGS site mesh and unmodified FEMA Region III mesh results (FHRR Figure 2.4-3)

Staff Technical Evaluation

The Staff Technical Evaluation for Section 5.1 discusses the features of the ADCIRC+SWAN mesh applied in the FEMA Region III and PSEG FHRR analyses. Review of the PSEG FHRR ADCIRC+SWAN mesh shows the model provides a highly-resolved model grid that applies the

best-available bathymetry and topographic data. Revalidation of refined SWAN+ADCIRC mesh (Figure 8) and the nested SWAN model (Figure 7) demonstrates that changes to the FEMA Region III mesh do not alter model results away from the PSEG SGS site and allow for increased resolution of site-specific features.

6.0 Numerical Model Validation

Information Submitted by the Licensee
FHRR Section 2.4.1 states "Validate the modeling system in the PSEG site region" as the second step in the procedure to develop the storm surge elevations at the PSEG SGS site. FHRR Section 2.4.2.2 states the ADCIRC+SWAN model, with the refined mesh near the PSEG SGS site, is revalidated using the same Hurricane Isabel and Nor'easter Ida test storm input files as the FEMA model validation report prepared by USACE (2013). Figure 8 (FHRR Figure 2.4-3) presents a graphical comparison of water levels from the Hurricane Isabel storm simulation on the refined PSEG SGS site mesh and unmodified FEMA Region III mesh at locations around the PSEG SGS site. The FHRR states that this process confirms the refined mesh produces results that are essentially the same as the unmodified FEMA Region III mesh in the vicinity of the PSEG SGS site.

FHRR Section 2.4.2.3 presents comparisons of the nested SWAN model output with the coupled ADCIRC+SWAN model output for several locations (Figure 7, FHRR Figure 2.4-13). The comparisons demonstrate that the nested SWAN model can produce similar wave height results as compared the ADCIRC+SWAN model with the refined mesh near the PSEG SGS site.

Staff Technical Evaluation
The FHRR analysis applies the FEMA Region III ADCIRC+SWAN mesh and model with a refined mesh near the PSEG SGS site and a nested SWAN model for additional resolution in the wave results. The FEMA III ADCIRC+SWAN mesh and model underwent several layers of review during the FEMA study process. Comparisons of SWAN+ADCIRC water levels from the resolved PSEG mesh model shows good agreement with the FEMA Region III results for Hurricane Isabel (Figure 8, FHRR Figure 2.4-3). USACE (2013) shows good agreement between the FEMA Region III water level results and the measured hydrograph and high water mark data for Hurricane Isabel, Hurricane Ernesto, and Extra-tropical Storm Ida (Nor-Ida). Notably, the maximum measured and modeled water levels in USACE (2013) do not exceed 2.5 m (8.2 ft), which does not approach the levels simulated for the PMH or 10^{-6} AEP storm surge water elevation.

The comparisons of the nested SWAN model results with the ADCIRC+SWAN model results in Figure 7 (FHRR Figure 2.4-13) shows the nested grid model produces results similar to the ADCIRC+SWAN model with the refined mesh near the PSEG SGS site. The FHRR report does not provide any direct comparisons of ADCIRC+SWAN model results versus measured data. Review of USACE (2013) shows comparisons of ADCIRC+SWAN model results versus measured wave data for Hurricane Isabel, Hurricane Ernesto, and Extra-tropical Storm Ida (Nor-Ida). The USACE (2013) report states that, despite a significant effort to locate data, only limited wave parameter validation data exist for the three storms. The USACE (2013) comparisons of measured and modeled significant wave height, peak wave period, and mean wave direction show good agreement.

The FHRR contains limited presentation of model validation results. However, review of the USACE (2013) model validation indicates the FEMA Region III model and mesh, which was applied in the FHRR analysis, produces water levels and waves that match well with measured data for recent significant storms.

7.0 Numerical Model Error and Uncertainty

Information Submitted by the Licensee

The FHRR states the probabilistic storm surge analysis developed a total epistemic uncertainty of 2.6 ft (FHRR Section 2.4.3.5) and an aleatory uncertainty equal to 2.3 ft (FHRR Section 2.4.3.8).

Staff Technical Evaluation

Total epistemic uncertainty is 2.6 ft (FHRR Section 2.4.3.5) and aleatory uncertainty is 2.3 ft (FHRR Section 2.4.3.8). As Staff developed an independent deterministic storm surge analysis, staff did not perform a detailed review of the licensee's methodology to develop the epistemic or aleatory uncertainty associated with the licensee's probabilistic storm surge estimate. .

8.0 Storm Surge Water Levels.

Information Submitted by the Licensee
FHRR Section 2.4.1 describes the procedure to estimate storm surge at the PSEG SGS site. The analysis applies a modified Joint Probability Method (JPM) approach that follows the USACE's Response Surface Method (FEMA, 2012). The Response Surface Method selects storms for simulation in such a way as to accurately cover the entire storm parameter space through optimal parameter selection with associated weighting and interpolation methods.

Based on recent analysis conducted by USACE and FEMA, PSEG established the following storm surge estimation procedure for developing the 10^{-6} AEP storm surge elevation at the PSEG SGS site.

- Develop a high resolution state-of-the-art coupled wave-surge modeling system with accurate bathymetric-topographic data
- Validate the modeling system in the PSEG site region
- Establish the method of performing the wave runup analysis to determine the total water surface elevation (total WSEL) at the site during each storm event modeled
- Develop a synthetic suite of storms which include a range of storm parameters and combinations of those parameters
- Simulate the synthetic suite of storms in the modeling system
- Output surge and wave results at critical locations around the Salem and Hope Creek Generating Stations to determine a maximum surge still WSEL and total WSEL (with wave runup) for each storm

- Develop the surge response function for the PSEG site
- Estimate the probabilities for the storm parameters
- Integrate the probabilities within the JPM-OS integral, including epistemic uncertainty and the effect of tides
- Determine the effects of aleatory uncertainty and develop the 10^{-6} still WSEL
- Estimate the correlation between still WSEL and total WSEL, and determine the 10^{-6} total WSEL, including wave runup.

Staff Technical Evaluation

The FHRR storm surge methodology focuses on the development of a probabilistic WSEL —10^{-6} AEP storm surge elevation — at the PSEG SGS site. The staff's evaluation focuses on the development of an independent estimate for the storm surge elevation at the PGEG site generated by a PMH event. Development of the independent PMH storm surge elevation allows for an assessment of the general storm surge results from the PSEG FHRR probabilistic approach — designed to develop the 10^{-6} AEP storm surge elevation.

NRC staff applied the PSEG ADCIRC+SWAN model as the platform for the independent storm surge estimate. As discussed in Sections 5.1 and 5.3, the PSEG ADCIRC+SWAN model includes increased resolution near the PSEG SGS site to allow for more refined solutions in the area of interest. As a first step, NRC 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, NRC staff confirmed the model's capability to reproduce the PSEG study model results near the project site for similar model settings and storm forcing. Staff executed the PSEG study Hurricane Isabel validation simulation and the PSEG ESP application (PSEG, 2010; PSEG 2015; NRC, 2015) PMH storm simulation on Taylor Engineering's "Merlin" high-performance computer (HPC). The results from the independent Hurricane Isabel and PMH storm simulations showed nearly identical values near the project site as compared to the PSEG ESP application results with differences in maximum water levels on the order of 0.03 ft (0.01 m). 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 a water level instability in the small channel located over 40 mi (64.4 km) 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 SGS 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 PSEG SGS site, NRC staff next developed and executed simulations for storms with variations in the PMH forcing. NWS 23 (NOAA, 1979) provided the PMH meteorological parameter ranges for the PSEG SGS site. The NWS 23 meteorological parameters appropriate for the PSEG SGS site are:

- Central pressure, p0 = 26.65 in. of Mercury (Hg) (902.5 mb)
- Pressure drop, Δp = 3.5 in. of Hg (118.52 mb)
- Radius of maximum winds, R = from 11 to 28 nmi (20.4 to 51.9 km)
- Forward speed, T = from 26 to 42 kts (48.2 to 77.8 km/hr)

- Track direction, from 138 degrees (moving northwest)

These parameter ranges match those applied in the PSEG ESP application. The PSEG ESP application PMH storm applies NWS 23 value for p0, Δp, and track direction along with the largest R value (28 nmi (51.9 km)) and slowest forward speed (26 kts (48.2 km/hr)). Importantly, the PSEG PMH simulations applied a landfall location offset 28 nmi (51.9 km) southwest from the center of the Delaware Bay mouth (PSEG ESP SSAR Figure 2.4.5-1). These parameter values provide the input forcing the PSEG ESP application PMH storm — results replicated in an NRC independent ADCIRC+SWAN simulation (Figure 9). At the PSEG SGS site, the PSEG ESP application and NRC staff independent PMH simulations produced maximum still WSEL near 18.6 ft-NAVD88 not including the tide, SLR, or runup.

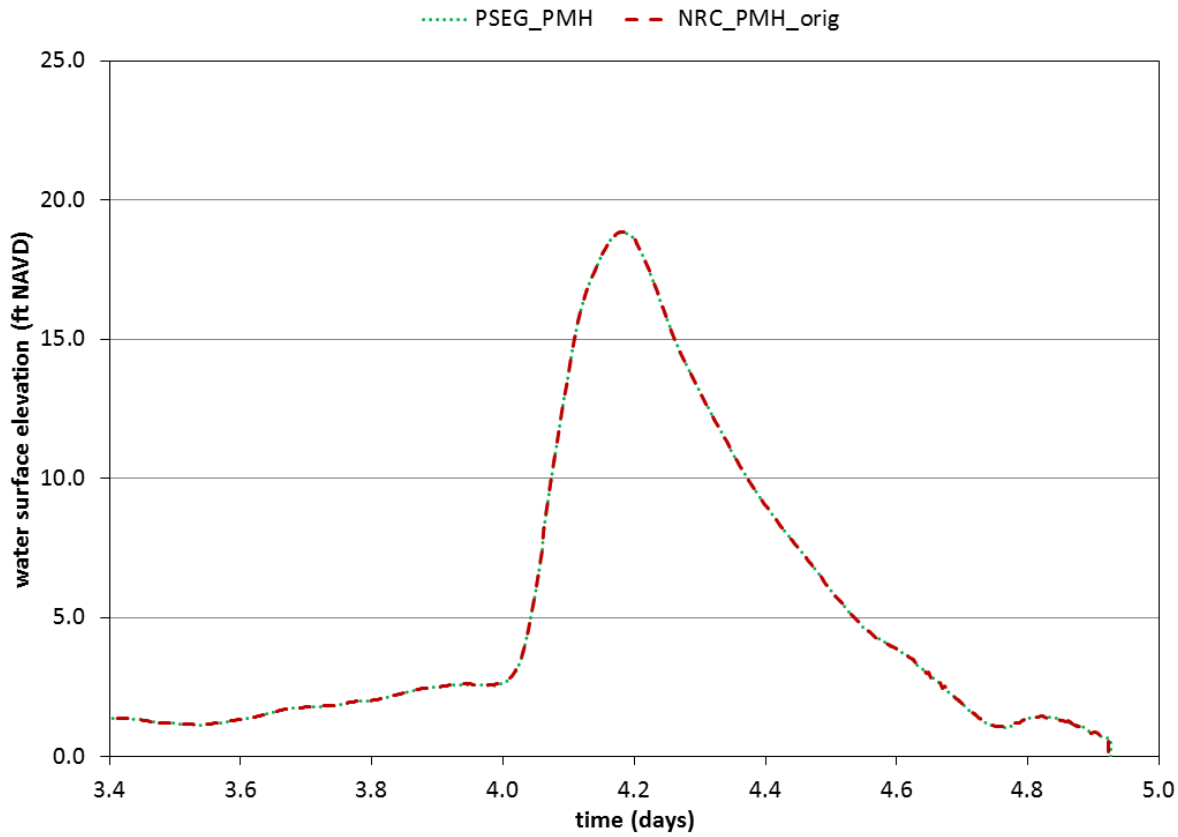

Figure 9. Comparison of PSEG ESP application and NRC independent PMH simulations at western edge of the ESP site

Given the complexity of the bay shape, selection of the landfall location could significantly influence the storm surge values at the PSEG SGS 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 SGS 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 SGS site. Given the bay geometry and the NWS 23 parameter ranges, NRC staff investigated the sensitivity of the landfall location and forward speed on the WSEL near the site.

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

1. PMH storm track shifted 14 nmi (25.9 km) to the southwest (SW_14_nmi)
2. PMH storm track shifted 14 nmi (25.9 km) to the northeast (NE_14_nmi)

Near the PSEG SGS site, the SW_14_nmi simulation showed increased maximum WSEL as compared to the PSEG PMH simulation with differences near 0.75 ft (0.23 m). The NE_14_nmi simulation showed decreased maximum WSEL near the PSEG SGS site as compared to the PSEG PMH simulation with differences near 5 ft (1.52 m). Given the counter-clockwise rotation of the hurricane wind field, having the northwest-shifted wind produce lower water levels seems reasonable. Based on this information, NRC staff executed additional shifted track simulations with the storm track shifted 7 nmi to the southwest (SW_7_nmi) and 21 nmi to the southwest (SW_21_nmi) . The SW_7_nmi and SW_21_nmi simulations produced maximum WSEL values between 0 and 1 ft higher than the PSEG PMH simulation; however, the maximum WSEL increase was less than that of the SW_14_nmi simulation. These simulations show that modifying the track landfall location can produce higher WSEL at the PSEG SGS site, but the increase in maximum WSEL is less than 1 ft (0.31 m).

To investigate the sensitivity of the storm surge results to the storm forward velocity, NRC staff executed two additional simulations with the PMH storm velocity increased to 30 kts (55.6 km/hr) and 34 kts (63.0 km/hr). 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 NRC staff to leverage the existing two-dimensional wind and pressure fields developed for the PMH (with a 26 kts (48.2 km/hr) forward velocity). As compared to the PMH storm forcing results, the model results for the 30 kts (55.6 km/hr) and 34 kts (63.0 km/hr) forward velocities indicate reduced maximum WSEL values near the PSEG SGS site. The maximum WSEL values are reduced by about 2 ft (0.61 m) for the 30 kts (55.6 km/hr) simulation and by about 4 ft (1.22 m) for the 34 kts (63.0 km/hr) simulation. These results indicate the 26 kts (48.2 km/hr) forward velocity — the slowest forward velocity in the range provided by NWS 23 — produces the largest WSEL at the PSEG SGS site.

Detailed review of the ADCIRC+SWAN model PMH simulation results in the immediate vicinity of the PSEG SGS site revealed some notable maximum water level features that NRC staff considered in need of 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 SGS site and extended into Delaware Bay (Figure 10). 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 SGS site to the node string, NRC 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 show 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 SGS site range from approximately +/- 0.1 ft (0.03 m) for the with land boundary versus without land boundary simulations. At times other than at maximum WSEL, differences can exceed 0.3 ft (0.91 m). These results indicate that the

land boundary nodes, while not having a documented purpose, cause only a minor effect on the maximum water level values near the project.

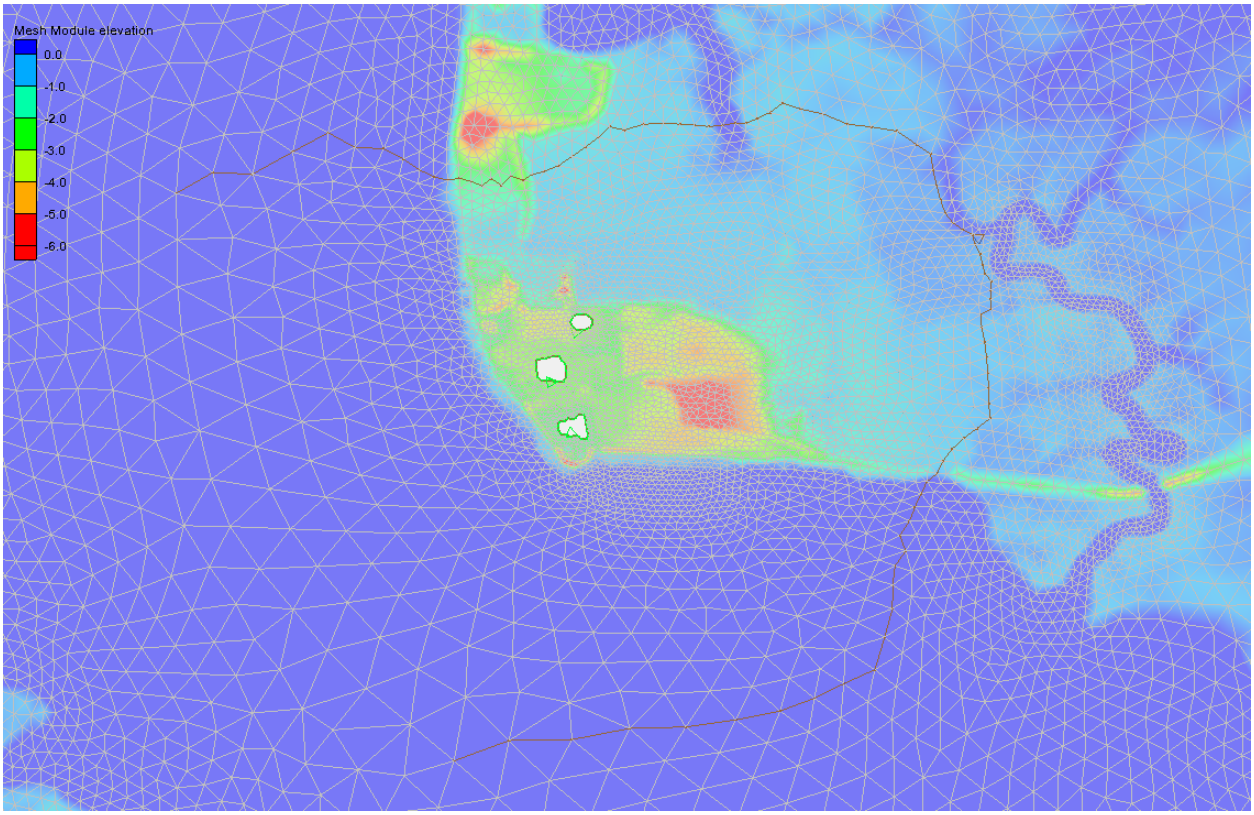

Figure 10. PSEG ADCIRC+SWAN mesh with elevation contours (m) and location of land boundary string.

Figure 11 compares still WSEL time series for several PMH simulations at the west side of the SGS site. The plot includes the results of the PSEG PMH simulation and independent PMH simulation conducted by the NRC staff; the simulations show almost identical results. Figure 11 also contains additional sensitivity model simulations. Removing the land boundary node string from the PSEG ESP application mesh does not significantly alter the simulated water levels at the site. Shifting the PMH storm track approximately 14 nmi (25.9 km) to the southwest increases the maximum water level at the site by about 0.7 ft (0.21 m). Increasing the number the maximum number of iterations to 8 in the SWAN model didn't significantly change the water levels at the site. The final confirmatory simulation (MaxAll) shifted the landfall location 14 nmi (25.9 km) to the southwest, removed the land boundary arc, and set the SWAN MAXITNS parameter equal to 8. This simulation shows slightly higher maximum water levels than the simulation with the storm track shifted 14 mi (25.9 km) southwest.

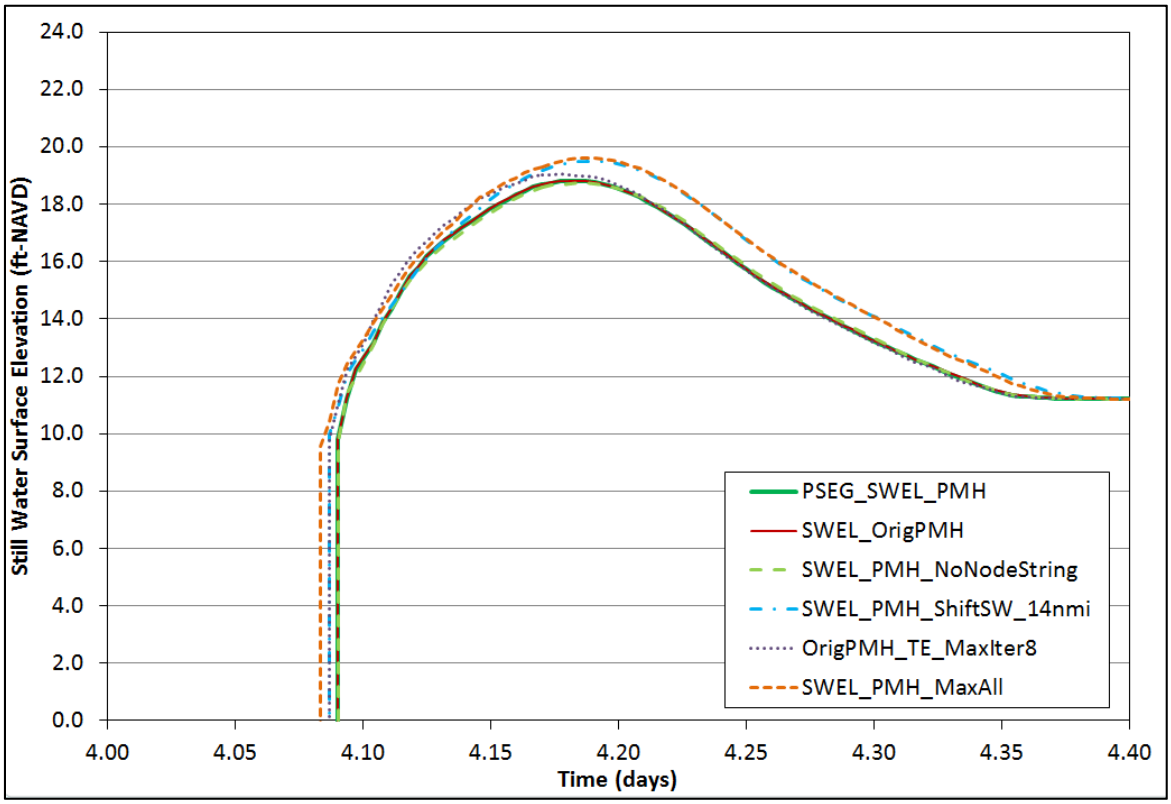

Figure 11. NRC staff confirmatory ADCIRC+SWAN model WSEL time series compared with the licensee results at the west side of the SGS site

Staff also compared wave height results for the PSEG ESP application PMH simulation and the sensitivity ADCIRC+SWAN simulations. Figure 12 compares significant wave height time series for several PMH simulations at the west side of the SGS site. The plot includes the results of the PSEG PMH simulation and independent PMH simulation conducted by the NRC staff. The simulations shows wave heights that differ by approximately 1 ft (0.31 m). Review of the SWAN model control files shows similar model settings and review of the ADCIRC water levels (Figure 11) confirms similar water levels for the PMH simulations. The difference in wave heights could relate to settings during the model compiling procedure or differences in the computing platforms applied. As shown in Figure 11, the ADCIRC model, which applies the same forcing, does not indicate any significant differences between the PSEG ESP application water levels and the NRC staff independent PMH simulation.

As previously mentioned, the PSEG FHRR models included a nested SWAN model that should further refine the wave results near the PSEG SGS site. The independent NRC staff simulations did not apply a nested SWAN grid. Comparison of significant wave height (Hs) time series at locations near the PSEG SGS site show similar wave heights and mean periods for the PSEG ESP application PMH and NRC staff PMH simulations for most comparisons. The NRC staff's PMH simulation with unexplained land boundary nodes removed produces slightly larger wave heights at some locations near the PSEG SGS site (locations adjacent the land boundary). The Hs results for the simulation with the land boundary nodes removed reach approximately 0.5 ft (0.15 m) to 1 ft (0.31 m) higher than the PSEG PMH simulation results. For locations closer the PSEG SGS site in areas that feature depth limited waves, the difference in Hs is negligible. Because the larger differences in Hs do not exceed one ft and locations nearer the PSEG SGS

site show negligible difference, the effect of the land boundary should not cause significant effects on water levels or wave runup.

NRC 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. NRC 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 SGS site with values that exceed those of the MXITNS = 2 simulation (original ESP application PMH simulation). At the west side of the PSEG SGS site (location with largest SWAN waves), the higher MXITNS simulations have maximum significant wave heights equal to 8.3 ft (2.53 m) versus 6.7 ft (2.04 m) 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 1 ft (0.31 m) to 4.5 ft (1.37 m) 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. Importantly, the results for the increased MXITNS values alter the timing of the maximum wave heights, but do not change the timing of the maximum water levels (Figure 11). Timing of waves and water level is important for the wave runup calculations.

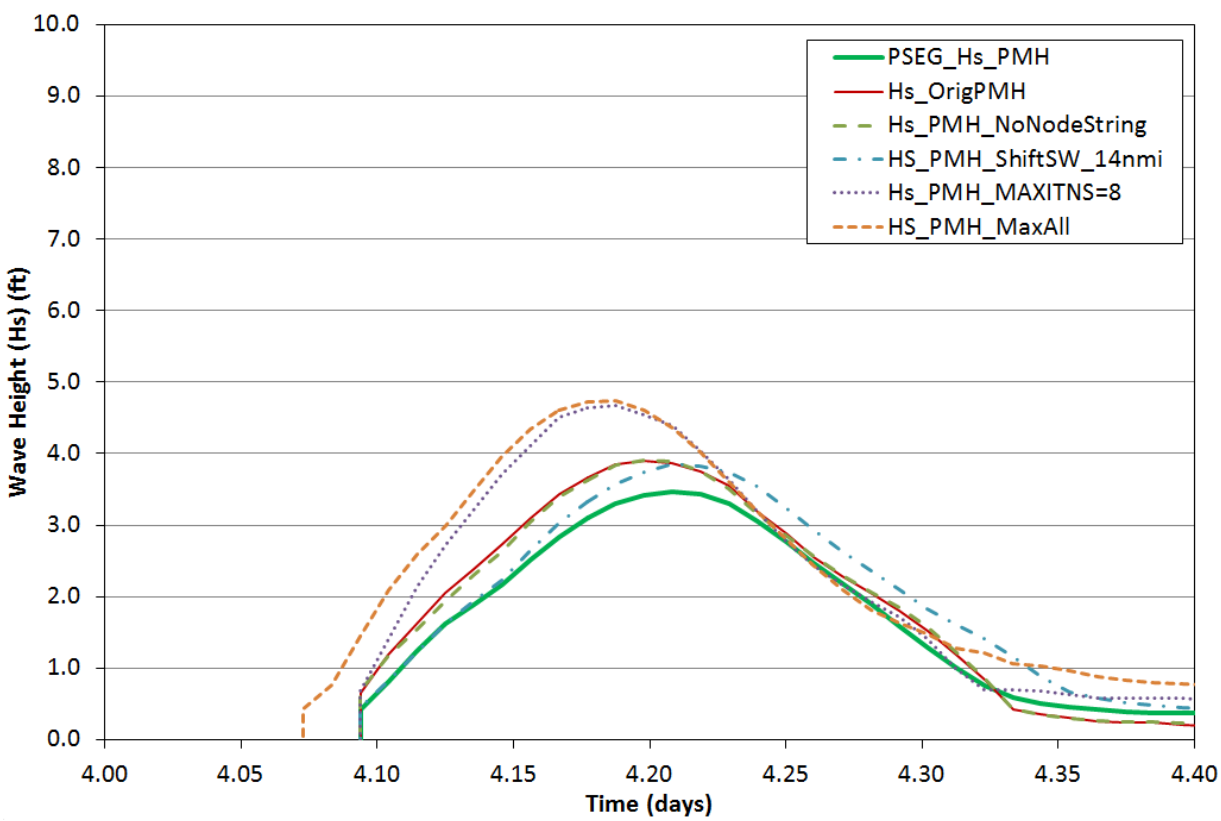

Figure 12. NRC staff confirmatory ADCIRC+SWAN model significant wave height time series compared with the licensee results at the west side of the SGS site

Notably, changing the MXITNS value did not significantly alter the maximum water levels near the PSEG SGS site — maximum differences near 0.4 ft (0.12 m) with most differences near 0.1 ft (0.31 m). However, changing the wave height can alter the wave runup (Section 8).

8.0 Wave Runup, Inundation, and Drawdown.

Information Submitted by the Licensee

FHRR Section 2.4.2.4 presents the wave runup methodology applied as part of the FHRR analysis. The methodology applies water level and wave results from the ADCIRC+SWAN and nested SWAN simulations. Each simulation produces storm still WSEL and wave data (significant wave height, period and direction) over the course of each storm surge event. The nested SWAN model provides wave field data at critical points around the plant (Figure 13, FHRR Figure 2.4-4) at 15-minute intervals. FHRR Section 2.10.1 notes that the nested SWAN modeling indicates wave blocking, diffraction, and scattering by the buildings. The procedure applied in the FHRR calculated the wave runup at each time step and at each of locations around the plant. After the calculations are performed, the peak total WSEL value, defined as the still WSEL plus wave runup, at each point is captured as the maximum value for that storm event at that point.

Figure 13. Output locations within the nested SWAN model applied in the wave runup calculations (FHRR Figure 2.4-4)

Wave runup calculations for the SGS site are based upon the latest design guidance found in the USACE Coastal Engineering Manual (CEM) (USACE, 2002, Chapter VI-5). The FHRR

analysis considers the waves impact vertical surfaces on the plant's critical surfaces; therefore, the FHRR applies the Goda equations to determine the wave runup height (USACE, 2002 Table VI-5-53, Equation VI-5-147). The CEM prescribes the use of 1.8 times the significant wave height for use as the design wave with the Goda equations.

The FHRR calculation of wave runup on the vertical surfaces of plant structures follows the below procedure:

- Using the nested SWAN model output, results for significant wave height (Hs) and wave period (T) are acquired from the grid node closest to the location of interest at 15 minute increments

- The procedure checks the significant wave height (H_s) for both steepness and depth-limited conditions in accordance with Smith (2007), and, if necessary, reduces the wave height using the below equation:

  $$Hs(applied) = minimum\ [0.1*L*\tanh((2\pi/L)h), Hs] \quad \text{(FHRR Equation 2.4-1)}$$

  where:
  L is the local wave length computed from the peak period and the local water depth using linear wave theory.

- The design wave height is established by using the below equation in accordance with Goda equations (USACE, 2002, Table VI-5-53).

  $$H_{design} = 1.8*Hs(applied) \quad \text{(FHRR Equation 2.4-2)}$$

- The wave runup elevation is established using USACE (2002, Equation VI-5-147) as presented below:

  $$\eta^* = 0.75(1+\cos\beta)\lambda_1*H_{design} \quad \text{(FHRR Equation 2.4-3)}$$

  where:
  η^* is the runup elevation
  β is the angle of incidence of waves
  λ_1 is the modification factor based on structure type, taken as 1 for conventional vertical wall structures

The FHRR runup calculations applied the above procedure for each time step at each point of interest at the plant. The procedure recorded the maximum value for each storm event analyzed and a corresponding total WSEL was established, as further discussed in Subsection 2.4.3.7.

FHRR Table 2.4-5 shows the maximum still WSEL for each of the storm simulations. The maximum total WSEL plotted as a function of the maximum still WSEL for each storm are shown in FHRR Figures 2.4-14 through 2.4-26. Figure 14 shows an example of the FHRR maximum still versus maximum total water level plot for SGS SWIS North Wall (FHRR Figure 2.4-14). Table 4 (FHRR Table 2.4-13) provides the linear regression equations which closely fit the data shown in the corresponding figures for each critical location around the PSEG SGS site.

The FHRR states the regression equations in Table 4 (FHRR Table 2.4-13) are developed without the mean tidal value. Therefore, to determine a total WSEL at the 10^{-6} AEP, the influence of the tides must be removed from the 10^{-6} AEP still WSEL. The FHRR procedure removes, from the 10^{-6} AEP still WSEL, the 0.59 ft (0.18 m) mean tide value described in FHRR Subsection 2.4.3.1.2.1.4. The procedure then computes the total WSEL, adds the mean tide value back in, and determines a 10^{-6} AEP total WSEL with tides for each point.

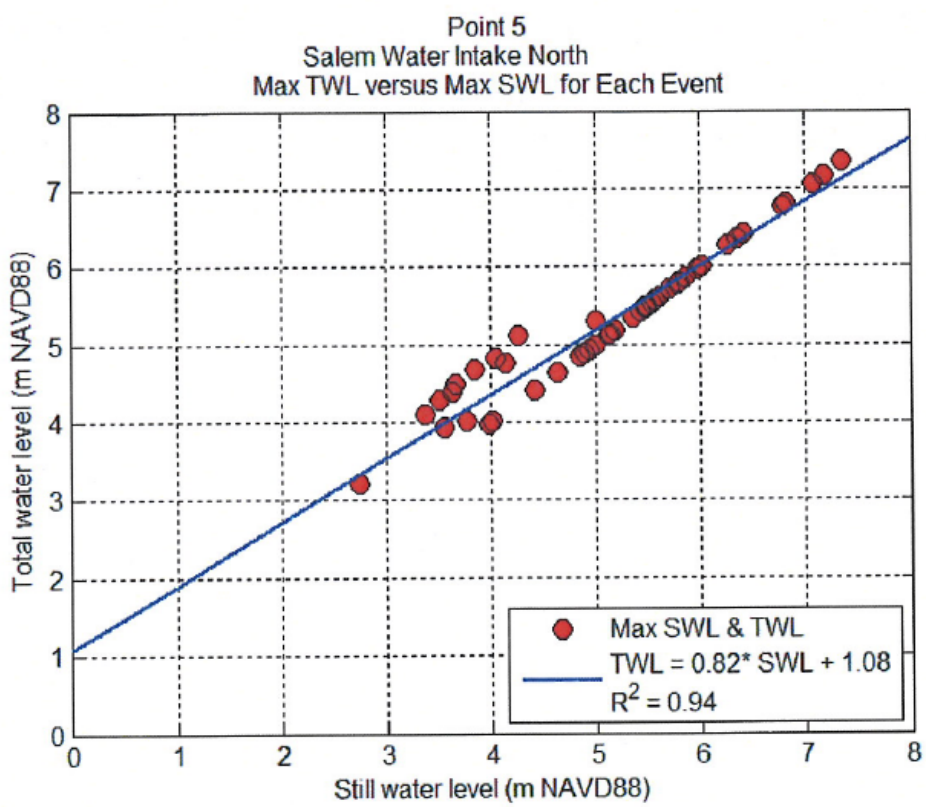

Figure 14. Comparison of still water level and total water level at Point 5 — SGS SWIS North Wall (FHRR Figure 2.4-14)

Table 4. Still water level and total water level relationship at each location (FHRR Table 2.4-13)

Location	Regression Equation	R^2	Regression Plot Figure Number
5	TWL=0.82*SWL+1.08	0.94	2.4-14
6	TWL=1.64*SWL+0.03	0.99	2.4-15
7	TWL=2.11*SWL-3.09	1.00	2.4-16
8	TWL=1.27*SWL-0.61	0.99	2.4-17
9	TWL=1.23*SWL-0.68	0.99	2.4-18
10	TWL=1.32*SWL-1.03	0.95	2.4-19
11	TWL=1.69*SWL-2.23	0.99	2.4-20
12	TWL=1.68*SWL-2.18	0.99	2.4-21
13	TWL=1.48*SWL-1.44	0.99	2.4-22
14	TWL=1.51*SWL-1.53	0.99	2.4-23
15	TWL=1.37*SWL-1.13	0.99	2.4-24
16	TWL=1.07*SWL-0.13	0.98	2.4-25
17	TWL=SWL	1.00	N/A
18	TWL=1.14*SWL-0.33	0.98	2.4-26

Staff Technical Evaluation

NRC staff applied the independent PMH storm analysis ADCIRC+SWAN model results to evaluate the run-up at the PSEG SGS site. Specifically, the analysis included output points at the south, west, north, and east sides of the Salem and Hope Creek facilities. Staff concurs with the selection and application of the USACE (2002) Goda methodology for runup on vertical structures. Application of the 1.8*Hs factor provides a conservative wave height threshold.

Similar to the FHRR analysis procedure, NRC staff applied the Goda method equations at the output points located around the SGS site with application of the water level and wave data at each time step. The analysis determined the appropriate wave height after checking for both steepness and depth-limited breaking criteria.

As discussed in Section 8.0, independent PMH simulations conducted by the NRC produced very similar still water levels near the PSEG SGS site. Additional PMH simulations with modified landfall locations and forward velocities tested the sensitivity of the model results to these parameters. Table 5 presents the results of the wave runup and total water level analysis for the NRC Independent ADCIRC+SWAN simulations. The table contains results for three separate NRC PMH simulations:

1) Confirmatory Run 1: Original PSEG ESP application PMH parameters
2) Confirmatory Run 2: Original PSEG ESP application PMH parameters with landfall location shifted 14 nmi to the southwest
3) Confirmatory Run 3: Original PSEG ESP application PMH parameters with landfall location shifted 14 nmi to the southwest, land boundary arc removed, and SWAN MAXITNS parameter set equal to 8.

The NRC independent confirmatory analysis results contain separate rows for the still water elevation, wave runup, and total water level for output locations on the south and west sides of the PSEG SGS site.

Maximum Still WSEL Results

Table 5 shows, for similar forcing, the independent PMH simulations maximum still WSEL values at the Salem West and South locations are within 0.5 ft of each other. Given the relative proximity of the locations, the similarity in maximum still WSEL values seems reasonable. The maximum still WSEL values in Table 5 for the Salem West location differ by less than 1 ft across all three simulations (23.8 to 24.6 ft-NAVD88). Similarly, the maximum still WSEL values for the Salem South location differ by less than 1 ft across all three simulations (24.1 to 25.0 ft-NAVD88). This shows that the modifications to the PMH forcing — shifting track, removing landing boundary nodes, change in MAXITNS value — have a relatively minor influence on the maximum still WSEL values at the PSEG SGS site.

Table 5 contains comparisons of the NRC independent PMH simulation maximum still WSEL results with the PSEG SGS FHRR water levels. Notably the FHRR contains the CLB values from the UFSAR (PSEG, 2013a) — based on a deterministic PMH forcing — and a reevaluated surge level — based on a 10^{-6} AEP still WSEL. FHRR Table 3-1 compares the SGS CLB storm surge elevations and reevaluated storm surge elevations at the auxiliary building. Table 3.1 lists the CLB still water height as 113.8 ft-PSD (24.0 ft-NAVD88) and revaluated flood level as 112.3 ft-PSD (22.5 ft-NAVD88). Table 5 shows that the NRC independent PMH maximum still water levels range from 0.2 ft below (Confirmatory Run #1 at Salem West) to 1.0 ft above (Confirmatory Run #3 at Salem South) the CLB results of 24.0 ft-NAVD88. The NRC independent PMH maximum still water levels range from 1.3 ft above (Confirmatory Run #1 at Salem West) to 2.5 ft above (Confirmatory Run #3 at Salem South) the FHRR reevaluated Stillwater level of 22.5 ft-NAVD88 for the 10^{-6} AEP still WSEL at the PSEG SGS site. The NRC independent PMH simulation results show maximum still WSEL values very near the CLB values and approximately 2 ft above the 10^{-6} AEP still WSEL from the reevaluation.

Runup Results

The NRC independent PMH simulation runup results show more variation than the maximum still WSEL values across the various forcing and the two locations. Runup values for the Salem West location range from 7.4 ft to 9.6 ft and for the Salem South location from 6.7 ft to 10.8 ft. The runup estimates are lowest for the original PMH forcing and highest for the PMH forcing that includes the shifted track, removal or the land boundary nodes, and increase of MAXITNS.

Table 5 contains comparisons of the NRC independent PMH simulation runup results with the PSEG SGS FHRR runup results. FHRR Sections 1.2.4 and 2.4.2.4 provide details of the runup methodology applied in the PSEG SGS CLB analysis and FHRR reevaluation. Subtracting the still water height from the max flood height in FHRR Table 3-1 allows calculation of the runup computed for the SGS CLB storm surge elevations and reevaluated storm surge elevations at the auxiliary building. Based on the data shown in FHRR Table 3.1 the CLB wave runup at the auxiliary building equals 6.6 ft and the revaluated runup equals 7.4 ft for the 10^{-6} AEP still WSEL. Table 5 shows that the NRC independent PMH runup estimates range from 0.1 ft above (Confirmatory Run #1 at Salem South) to 4.2 ft above (Confirmatory Run #3 at Salem South) the CLB results of 6.6 ft. The NRC independent PMH maximum still water levels range from 0.7 ft below (Confirmatory Run #1 at Salem South) to 3.4 ft above (Confirmatory Run #3 at Salem South) the FHRR reevaluated runup value of 7.4 ft for the 10^{-6} AEP still WSEL.

Notably, the NRC independent PMH simulations applied the ADCIRC+SWAN model with the refined mesh near the PSEG SGS site; however, even with the increased resolution, the refined mesh cannot capture the varied infrastructure and topography that occur between the open water of Delaware Bay and the PSGE SGS auxiliary building. The FHRR revaluation analysis applies a nested SWAN grid to further resolve the varied topography and infrastructure near the PSEG SGS site. However, even with this extra resolution, the interaction of the storm-driven surge and waves with the PSEG SGS site infrastructure challenges the state-of-the-art wave modeling capability. The LiDAR data applied to develop the model mesh topography represents a bare-earth condition (with buildings removed) so the runup estimates do not include any breaking or other wave transformation induced by the structures other than the main auxiliary buildings and cooling tower (area cut out of the refined PSEG mesh).

Given the differences in model resolution, the NRC independent PMH simulation run values (from 6.7 to 10.8 ft) agree reasonably well with the FHRR revaluation analysis wave runup estimate of 7.4 ft for the 10^{-6} AEP still WSEL.

Total Still WSEL Results

Table 5 contains estimates of the total water level that combines the maximum still water elevation, wave runup, tide effects, and SLR near the SGS auxiliary building. The tide effects are set at 4.5 ft to match the 10% exceedance high tide level based on the NOAA tidal gage at Reedy Point, DE (Station 8551910). The 10% exceedance level comes from ANS 2.8 guidance (ANS, 1992). NRC staff confirmed the magnitude of the 10% exceedance high tide value that is also applied in the PSEG ESP application PMH storm surge calculations (PSEG, 2010; PSEG 2015; NRC, 2015). The SLR magnitude equals 0.5 ft as discussed in Section 4.0. Table 6 contains estimates of the total water level that combines the maximum still water elevation, wave runup, tide effects, and SLR near the SGS intake structure.

Table 5. Runup and total water elevation results for NRC Independent ADCIRC+SWAN simulations near the SGS auxiliary building

Parameter	**Staff ADCIRC** Confirmatory Run#1[3]		**Staff ADCIRC** Confirmatory Run#2[4]		**Staff ADCIRC** Confirmatory Run#3[5]	
Peripheral Pressure (in. of Hg)	30.15		30.15		30.15	
Central Pressure (in. of Hg)	26.64		26.64		26.64	
Radius of Maximum Winds (nmi)	28		28		28	
Forward Speed (kt)	26		26		26	
Max Wind Speed (kt)	126.3		126.3		126.3	
10% Astronomical High Tide (ft)	4.5 [1]		4.5 [1]		4.5 [1]	
Sea Level Rise (SLR) (ft)[2]	0.5 [1]		0.5 [1]		0.5 [1]	
Location	Salem West	Salem South	Salem West	Salem South	Salem West	Salem South
Maximum Still Water Level (ft-NAVD88) [Includes tide/SLR]	23.80	24.05	24.52	24.90	24.61	25.04
Wave Runup (ft)	7.35	6.70	7.82	8.14	9.55	10.83

Maximum Total Water Surface Elevation (ft-NAVD88)	31.15	30.75	32.34	33.04	34.16	35.87

FHRR Table 3-1 Auxiliary Bldg UFSAR Stillwater (ft-NAVD88)[6]	24.0	24.0	24.0
FHRR Table 3-1 Auxiliary Bldg Reevaluated Stillwater (ft-NAVD88)	22.5	22.5	22.5

FHRR Table 3-2 WSE Auxiliary Bldg UFSAR Hazard (ft-NAVD88)	30.6	30.6	30.6
FHRR Table 3-2 WSE Auxiliary Bldg Reevaluated Hazard (ft-NAVD88)	29.9	29.9	29.9

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

[2)]Apply projection for 32 years (longest operating license of Salem / Hope Creek plants)

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

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

[5)]Shift track 14 nmi to the SW, removed land boundary string in ADCIRC+SWAN mesh, set SWAN MXITNS = 8 (was 2)

[6)]Conversion between NAVD88 and PSEG Datum: PSEG Datum = NAVD88+89.8 ft

Table 6. Runup and total water elevation results for NRC Independent ADCIRC+SWAN simulations near the SGS intake

Parameter	**Staff ADCIRC** Confirmatory Run#1[3]	**Staff ADCIRC** Confirmatory Run#2[4]	**Staff ADCIRC** Confirmatory Run#3[5]
Peripheral Pressure (in. of Hg)	30.15	30.15	30.15
Central Pressure (in. of Hg)	26.64	26.64	26.64
Radius of Maximum Winds (nmi)	28	28	28
Forward Speed (kt)	26	26	26
Max Wind Speed (kt)	126.3	126.3	126.3
10% Astronomical High Tide (ft)	4.5 [1]	4.5 [1]	4.5 [1]
Sea Level Rise (ft)[2]	0.5 [1]	0.5 [1]	0.5 [1]
Location	Salem Intake	Salem Intake	Salem Intake
Maximum Still Water Level (ft-NAVD88) [Includes tide/SLR]	23.47	24.20	24.25
Wave Runup (ft)	13.81	14.58	17.18
Maximum Total Water Surface Elevation (ft. NAVD88)	37.28	38.78	41.43

FHRR Table 3-1 Service Water Intake Structure UFSAR Stillwater (ft-NAVD88)[6]	24.0	24.0	24.0
FHRR Table 3-1 Service Water Intake Structure Reeval. Stillwater (ft-NAVD88)	22.5	22.5	22.5

FHRR Table 3-2 WSE Service Water Intake Structure UFSAR Hazard (ft-NAVD88)	37.5	37.5	37.5
FHRR Table 3-2 WSE Service Water Intake Structure Reevaluated Hazard (ft-NAVD88)	36.3	36.3	36.3

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

[2)]Apply projection for 32 years (longest operating license of Salem / Hope Creek plants)

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

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

[5)]Shift track 14 nmi to the SW, remove land boundary string in ADCIRC+SWAN mesh, set SWAN MXITNS = 8 (was 2)

[6)]Conversion between NAVD88 and PSEG Datum: PSEG Datum = NAVD88+89.8 ft

The maximum total WSEL values in Table 5 for the Salem West location differ by 3 ft across all three simulations (31.2 to 34.2 ft-NAVD88). The maximum total WSEL values for the Salem South location differ 5 ft across all three simulations (30.8 to 35.9 ft-NAVD88) owing mainly to the difference in wave runup for the Confirmatory Run #3. As stated in the wave runup discussion, the simulation of nearshore waves in and around the PSEG SGS infrastructure increases the uncertainty of the wave model and wave runup results at the site. Table 5 shows that the NRC independent PMH maximum total water levels range from 0.5 ft above (Confirmatory Run #1 at Salem West) to 5.2 ft above (Confirmatory Run #3 at Salem South) the CLB results of 30.6 ft-NAVD88. The NRC independent PMH maximum total water levels range from 1.2 ft above (Confirmatory Run #1 at Salem West) to 5.9 ft above (Confirmatory Run #3 at Salem South) the FHRR reevaluated maximum total water level of 29.9 ft-NAVD88 for the 10^{-6} AEP still WSEL at the PSEG SGS site.

Table 6 shows that the NRC independent PMH maximum total water levels at the SGS intake range from 0.2 ft below (Confirmatory Run #1) to 3.9 ft above (Confirmatory Run #3) the CLB results of 37.5 ft-NAVD88. The NRC independent PMH maximum total water levels range from 1.0 ft above (Confirmatory Run #1) to 5.1 ft above (Confirmatory Run #3) the FHRR reevaluated maximum total water level of 36.3 ft-NAVD88 for the 10^{-6} AEP still WSEL at the PSEG SGS site.

Figure 13 shows the PSEG SGS site and the infrastructure that occurs to the west and south of the auxiliary structure. Figure 15 shows a contour plot of the PSEG SGS SWAN+ADCIRC model mesh and topography with elevations in meters (positive values below 0 ft-NAVD88). Figure 15 shows that the model mesh includes topographic features on the Delaware Bay shoreline, but does not capture buildings or infrastructure near the main SGS auxiliary building or Hope Creek auxiliary building or cooling tower — identified by the white areas “cut-out” of the mesh.”

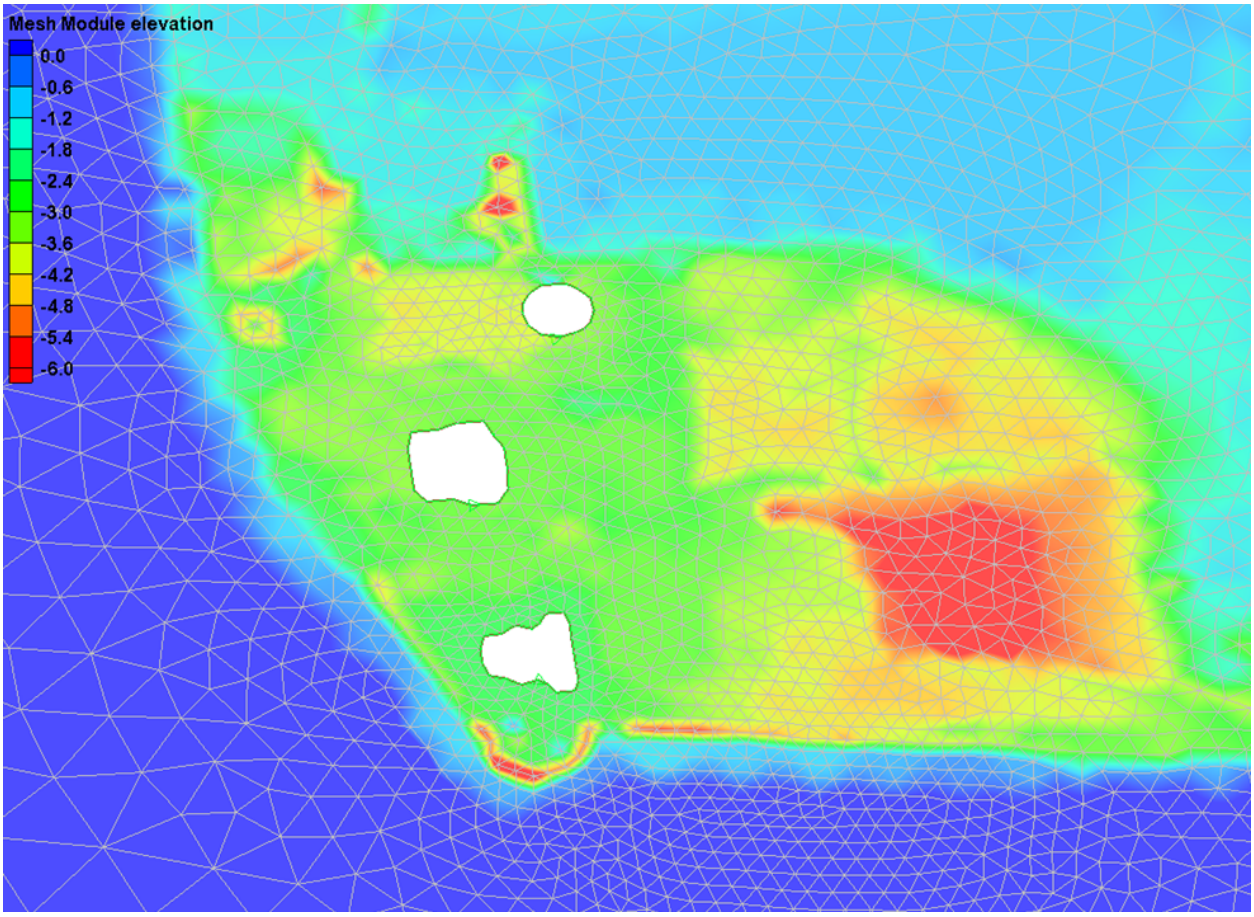

Figure 15. Contour plot of PSEG SGS SWAN+ADCIRC model mesh and topography (elevations in meters; positive values below 0 ft-NAVD88)

10.0 Flood Event Duration

Information Submitted by the Licensee

FHRR Section 2.10.6 states that flood event duration is defined by NRC (2012) as the length of time the flood event affects the site, beginning with conditions being met for entry into a flood procedure or notification of an impending flood (e.g., a flood forecast or notification of dam failure), including preparation for the flood and the period of inundation, and ending when water has completely receded from the site and the plant has reached a safe and stable state that can be maintained indefinitely.

Section 2.10.6.2 lists the specific aspects related to site preparation for a flood event at the PSEG SGS site. The site preparation prior to a severe weather event begins 48 hours before the potential onset of the event. PSEG (2012) summarizes a list of the trigger procedures and associated actions for providing flood protection and mitigation. Closure of watertight doors is the primary flood protection activity taken in the hours proceeding the period of inundation. Watertight doors are required to be closed based on a river level of 10.5 ft MSL or 99.5 ft PSD per Technical Specification 3/4.7.5.1.

The FHRR states the reevaluated storm surge hazard does not introduce any new limitations on the current site preparation for the flood event. The Technical Specification 3/4.7.5.1 watertight door closure time of two hours after the river level of 10.5 ft MSL or 99.5 ft PSD is reached is challenged by the rate of inundation, but SGS programmatically closes the doors prior to this river level.

The FHRR states the period of inundation for the storm surge event is developed by assessing the six storm events discussed in FHRR Section 2.10.1 and determining the duration the storm surge WSEL exceeds site grade, nominally and conservatively taken as 9 ft-NAVD88 (98.8 ft. PSD). Results from the ADCIRC+SWAN simulations indicate the average period of inundation equals 10.9 hours with a standard deviation of 1.3 hours. FHRR Section 3.10.6 compares the storm surge duration with the currently licensing basis.

The FHRR states the recession of water from the site is considered in the values provided in the period of inundation duration. The ADCIRC+SWAN storm surge simulations were run with sufficient duration to assure any effect of the surge water traveling back towards the site is not encountered. Therefore, once the surge water recedes during the period of inundation, activities outside the plant's flood protected structures can proceed as required.

FHRR Section 3.10.6 compares the storm surge duration with the current licensing basis. The FHRR states that implementation of active flood protection features at SGS (e.g., watertight door closure) is currently initiated based on river level readings at the service water intake structure. Watertight door closure is initiated at river level 97.5 ft. PSD (7.7 ft-NAVD88), as indicated in the control room, in accordance with PSEG (2013b) and required by PSEG (2013c) at river level 99.5 ft. PSD (9.7 ft-NAVD88). The credited watertight door closure mission time is two hours per PSEG (2013c). The FHRR states that the SWAN+ADCIRC simulation results for a storm capable of producing the 10^{-6} AEP still WSEL at the PSEG SGS site, can feature a hydrograph where water levels could increase approximately five ft in two hours after reaching 97.5 ft. PSD (7.7 ft-NAVD88). Therefore, the FHRR states, the site may become inundated before operators can complete the watertight door closure process. This aspect of the flood hazard is not considered bounded by the CLB, and proposed interim actions based on the results of this assessment are described in FHRR Chapter 4.

Staff Technical Evaluation

The FHRR states that watertight doors are required to be closed at a river level of 10.5 ft MSL or 99.5 ft PSD (9.7 ft-NAVD88) per Technical Specification 3/4.7.5.1. The FHRR duration analysis applies a threshold river level of 9.0 ft-NAVD88 to include some conservatism in the duration analysis. To provide consistency with the FHRR analysis, the staff water surface duration analysis also applied a river level of 9.0 ft-NAVD88. Review of the ADCIRC+SWAN hydrographs near the PSEG SGS site for the three NRC independent runs provides time series information for when the total water level exceeds 9.0 ft-NAVD88 at the SGS site. The SGS intake location was selected as the most appropriate model output location because of the intake's location along the river. Notably, the FHRR report and review does not specify the location for the ADCIRC+SWAN model output applied in the FHRR duration analysis.

Review of the NRC independent PMH ADCIRC+SWAN model output at the SGS intake shows still water levels exceed 9.0 ft-NAVD88 for 7.9, 8.7, and 8.6 hours for the three confirmatory simulations discussed in Section 8.0. These durations are shorter than the average value determined by the licensee for the six representative storms analyzed in the FHRR duration

calculations. Based on review of the NRC staff confirmatory simulation surge durations, the FHRR surge durations seem reasonable and conservative.

11.0 Hydrostatic and Hydrodynamic Forces.

Information Submitted by the Licensee
FHRR Section 2.10.1 describes the analyses conducted to estimate hydrostatic and hydrodynamic loading at the site. The storm surge analysis and resulting 10^{-6} AEP still and total water levels, described in FHRR Section 2.4, provided the basis for the hydrostatic and hydrodynamic loading evaluation. The FHRR states that no guidance currently exists on how to apply the probabilistic water levels, so the analysis applied a suite of storms selected to represent the 10^{-6} event. Table 7 (FHRR Table 2.10-1) presents the subset of storms selected to bracket the 10^{-6} AEP water level with a tidal adjustment of 0.18 m (0.59 ft) included. The storms in Table 7 feature an average flood still water level of 6.7 m (21.98 ft), which matches the 10^{-6} AEP still water level (6.7 m-NAVD88, 22.0 ft). The values in Table 7 (FHRR Table 2.10-1) do not include the effects of aleatory uncertainty; however, the average still WSEL equals the 10^{-6} AEP water level including the effects of aleatory uncertainty.

The licensee states that selecting several storms allows for varied storm forcing conditions, and therefore, varied water level and wave conditions at the site. The licensee opines that this is advantageous over selecting a single storm that matches the 10^{-6} AEP water level.

Table 7. Subset of storms selected for wave and debris load calculations (FHRR Table 2.10-1)

Storm Number	Central Pressure (millibars)	Radius to Maximum Winds (nm)	Holland B Parameter	Landfall Displacement	Angle of Storm	Forward Speed (knots)	Still WSEL (m)
11	918	45	1.00	2.00	0.00	30	6.71
20	918	45	1.10	2.00	0.00	30	7.01
21	918	45	1.10	3.00	0.00	30	6.75
25	918	45	1.10	1.00	0.00	30	6.24
33	918	30	1.10	2.00	-22.50	30	6.28
60	918	30	1.10	2.00	-11.25	30	6.37

The FHRR analysis follows the NRC (2013) regulatory guidance and the Goda methodology provided in the USACE CEM (USACE, 2002) to calculate hydrostatic and hydrodynamic loads against inundated vertical surfaces. The method assumes the waves are not experiencing local breaking. The licensee opines that this is a reasonable assumption given wave blocking, diffraction, and scattering by the buildings and as indicated by the nested SWAN model results. The Goda method does not include the hydrostatic contribution to the total force on the wall, so the licensee added the force in to develop the total force on the wall.

For the six storms determined to bracket the 10^{-6} AEP storm surge event, Table 8 (FHRR Table 2.10-2) lists the maximum calculated total wave loading (hydrostatic and hydrodynamic) in kips/ft of wall for critical points. The table includes for each point wave loading in kips/ft and the average load for the point. The table shows locations on the river side of the intake structures, — directly front Delaware Bay — have the highest wave loads.

Table 8. Maximum wave loading for six storms and calculated 10^{-6} probability loads (FHRR Table 2.10-2)

Location	Max Wave Force for Each Storm (kips/ft)						Average Total Load[(b)] (kips/ft)
	11	20	21	25	33	60	
1	6.27	9.33	6.82	6.15	5.07	6.08	6.62
2	16.81	17.45	16.83	13.02	13.48	13.36	15.16
3	15.49	16.54	15.80	11.72	12.07	11.79	13.90
4	6.79	8.41	6.98	5.34	5.43	5.46	6.40
5	8.31	9.31	8.43	6.66	6.94	7.11	7.80
6	24.26	23.87	24.14	18.44	19.41	19.18	21.55
7	16.21	18.43	16.46	12.57	13.05	13.50	15.04
8	5.62	6.70	5.80	4.25	4.33	4.42	5.19
9	6.10	7.29	6.24	4.71	4.95	5.09	5.73
10	6.61	8.00	6.80	5.16	5.41	5.58	6.26
11	10.54	11.42	10.62	7.75	8.25	8.38	9.49
12	10.60	11.51	10.68	7.93	8.38	8.52	9.60
13	8.14	8.99	8.16	6.44	6.87	7.04	7.61
14	7.90	8.84	7.93	5.91	6.55	6.64	7.29
15	7.39	8.48	7.49	5.23	5.79	5.82	6.70
16	6.39	7.35	6.49	4.38	5.00	5.17	5.80
17			Not Applicable[(a)]				5.35
18	5.61	6.85	5.76	4.29	4.53	4.66	5.28
19	5.30	6.30	5.40	3.53	3.91	4.02	4.74
20	8.46	9.97	8.67	5.36	6.24	6.33	7.51
21			Not Applicable[(a)]				3.86
22	4.91	6.41	5.09	3.31	3.71	3.77	4.53

a) Hydrodynamic wave forces are not applicable at this location as wave action is blocked by the turbine building. Therefore, only the hydrostatic component of the total wave force is presented.
b) Total load includes hydrostatic forces and hydrodynamic wave loads.

FHRR Subsection 3.10.1 shows a comparison to the existing design total wave loading combinations at critical locations around the site (Tables 9 and 10, FHRR Table 3.3 Sheets 1–3). Tables 9 and 10 (FHRR Table 3.3 Sheets 1–3) indicate that, in all cases, the FHRR total loads are less than the loads applied in the critical safety-related structure analysis. Therefore, the licensee concludes that the reevaluated hydrostatic and hydrodynamic loads are bounded by the CLB.

Table 9. Comparison of Storm Surge Wave Loading (FHRR Table 3-3, Sheet 1 of 3)

Building	Wall ID	Breaking Wave Loads (kips/ft)			Broken Wave Loads (kips/ft)			Reference	Corresponding Location Number	Total Wave Force (kips/ft)
		Static load	Dynamic load	Total Load	Static load	Dynamic load	Total Load			
Unit 1 Fuel Handling	AB	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	13	7.61
	BC	12.60	53.90	66.50	16.93	3.24	20.17	Reference 3-5	14	7.29
	CD	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	15	6.70
	DE	12.60	53.90	66.50	16.93	3.24	20.17	Reference 3-5	15	6.70
	AF	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	13	7.61
	FG	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	13	7.61
	GH	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	13	7.61
	HJ	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	13	7.61
Unit 2 Fuel Handling	AB	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	11	9.49
	BC	12.60	53.90	66.50	16.93	3.24	20.17	Reference 3-5	10	6.26
	CD	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	9	5.73
	DE	12.60	53.90	66.50	16.93	3.24	20.17	Reference 3-5	9	5.73
	AF	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	11	9.49
	FG	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	11	9.49
	GH	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	11	9.49
	HJ	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	11	9.49
Unit 1 Penetration Area	AB	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	15	6.70
	BC	12.60	53.90	66.50	16.93	3.24	20.17	Reference 3-5	15	6.70
	CD	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	15	6.70

Table 10. Comparison of Storm Surge Wave Loading (FHRR Table 3-3, Sheet 2/3 of 3)

CLB PMH Loads									Reevaluated Total Loads	
Building	Wall ID	Breaking Wave Loads (kips/ft)			Broken Wave Loads (kips/ft)			Reference	Corresponding Location Number	Total Wave Force (kips/ft)
		Static load	Dynamic load	Total Load	Static load	Dynamic load	Total Load			
Unit 2 Penetration Area	AB	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	18	5.28
	BC	12.60	53.90	66.50	16.93	3.24	20.17	Reference 3-5	18	5.28
	CD	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	18	5.28
Auxiliary Building	AB	3.66	14.55	18.21	N/A	N/A	N/A[b]	Reference 3-5	12	9.60
	BC	9.82	45.90	55.72	13.19	3.00	16.19	Reference 3-5	12	9.60
	CD	3.66	14.55	18.21	N/A	N/A	N/A[b]	Reference 3-5	13	7.61
	DE	3.66	14.55	18.21	N/A	N/A	N/A[b]	Reference 3-5	13	7.61
	EF	3.66	14.55	18.21	N/A	N/A	N/A[b]	Reference 3-5	13	7.61
	GH	12.60	53.90	66.50	N/A	N/A	N/A[b]	Reference 3-5	15	6.70
	HI[c]	9.82	20.00	29.82	13.19	1.31	14.50	Reference 3-5	17	5.35
	IJ	3.66	14.55	18.21	N/A	N/A	N/A[b]	Reference 3-5	18	5.28
	KL	3.66	14.55	18.21	N/A	N/A	N/A[b]	Reference 3-5	11	9.49
	LM	3.66	14.55	18.21	N/A	N/A	N/A[b]	Reference 3-5	11	9.49
	MN	3.66	14.55	18.21	N/A	N/A	N/A[b]	Reference 3-5	11	9.49
	AN	3.66	14.55	18.21	N/A	N/A	N/A[b]	Reference 3-5	10	6.26
SWIS	AB[a]	49.90	----	81.40	N/A	N/A	N/A[a]	Reference 3-5	6	21.55
	BC	13.51	76.00	89.51	N/A	N/A	N/A[b]	Reference 3-5	7	15.04
	CD	9.82	2.60	12.42	13.20	0.17	13.37	Reference 3-5	8	5.19
	AD	2.01	6.24	8.24	N/A	N/A	N/A[b]	Reference 3-5	5	7.80

a) Non-breaking wave forces are considered on the full length of SWIS west wall.
b) Breaking wave force values used for the full length of the wall.
c) Per Reference 3-5, Segment HI (east wall of the auxiliary building) is sheltered by the service and turbine buildings; however, wave loads were conservatively calculated from fetch F1.

FHRR Section 3.10.1 states that per SGS UFSAR (PSEG, 2013a) Section 3.4.3.1, "All watertight doors and structural walls can withstand the static and dynamic effects associated with a storm that produces a stillwater level of Elevation 113.8 feet PSD with wave runup to Elevation 120.4 feet PSD" (24.0 ft-NAVD88 and 30.6 ft-NAVD88). In addition, SGS UFSAR (PSEG, 2013a) Section 2.4.5.6 states "maximum wave runup elevation was calculated to be 120.4 feet PSD on critical structures inside the sea wall and 127.3 feet PSD on the service water intake structure" (30.6 ft-NAVD88 and 37.5 ft-NAVD88). Therefore, the licensee concluded that external watertight doors can withstand the same wave loading combinations the respective walls experience, and as discussed above, the reevaluated loads are lower than the CLB load combinations and the SGS site is not adversely impacted.

Staff Technical Evaluation
Staff reviewed the licensee's technical analysis and approach for the hydrostatic and hydrodynamic loads at the SGS site.

Staff developed independent analyses for the hydrostatic and hydrodynamic loads near the SGS auxiliary building based on results of the staff independent ADCIRC+SWAN PMH model simulations. The staff hydrostatic and hydrodynamic load analyses applied similar equations to develop the maximum wave force (kips/ft). The staff analysis applied the independent ADCIRC+SWAN results for the PMH and PMH shifted 14 nmi southwest simulations. The staff analysis shows the independent wave forces for the PMH and shifted PMH runs feature lower total forces as compared to the FHRR analysis results. Therefore, the FHRR analysis forces seem reasonable and the FHRR forces are below the CLB values (Table 10, FHRR Table 3-3).

Notably, the FHRR analysis reports the average total load with the average load calculated from the six storms selected as representative of producing 10^{-6} AEP water levels. Staff feels that, to provide a conservative loading estimate, the analysis should have reported the maximum wave force produced by any of the six storms (and not the average value) at each location. However, review of the maximum wave loading at each location shows the maximum values are below the CLB values (Table 10, FHRR Table 3-3) for every location reported, except for Location 5, which was within 1 kip/ft.

12.0 Debris and Water-Borne Projectiles.

Information Submitted by the Licensee
The FHRR describes the CLB and updated analyses conducted to estimate debris loading at the site. FHRR Section 1.2.10.1 states that the CLB applied a probabilistic analysis of floating missiles impacting plant operations (Little, 1984). The HCGS Safety Evaluation Report (NRC, 1986) describes the NRC staff's assessment of floating missiles. The staff's analysis indicates that the potential for waterborne missiles being transported onto the Hope Creek site and endangering the power block safety-related structures or intake structures is acceptably low.

FHRR Section 2.10.2 provides an updated analysis of the potential impact loads due to floating debris during a severe storm. The analysis first establishes a floating debris spectrum and then applies an impact load methodology for critical locations round the PSEG SGS site. The floating debris spectrum analysis applies the study by Little (1984), which developed a floating debris spectrum for the PSEG SGS site. The licensee applies the floating debris spectrum in FHRR Section 2.10.2.2 (Waterborne Debris Impact Forces).

FHRR Section 2.10.2.1 suggests the Little (1984) study developed a conservative analysis of the probability of marine vessel traffic in the Delaware River impacting the power block or intake structures. To assess the applicability of Little (1984) analysis, the updated analysis compared the results to present day conditions. The comparison indicated the Little (1984) analysis contained such conservatism that the values are still conservative compared to present day conditions. The reanalysis with current day marine vessel traffic levels, and considering the increased hurricane preparations undertaken today by the United States Coast Guard (USCG) suggest the probability conclusions in Little (1984) are considered to remain applicable today. Therefore, the licensee opines further consideration of the impact of large marine vessels is not required.

FHRR Section 2.10.2.2 applies the debris spectrum with the equations from Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers (ASCE) 7-10 (2011). ASCE 7-10 (2011) provides requirements for general structural design and includes means for determining dead, live, soil, flood, snow, rain, atmospheric ice, earthquake, and wind loads, as well as their combinations, which are suitable for inclusion in building codes and other documents. ASCE 7-10 (2011), Section C5, specifically ASCE 7-10 Equation C5-3 shown below (Figure 16, FHRR Equation 2.10-1), presents a methodology for determining impact loads for flood borne debris. The licensee applied the ADCIRC+SWAN model storm surge and velocity results from six storms with surge values near the 10^{-6} AEP level. Tables 11 and 12 (FHRR Tables 2.10.3 and 2.10.4) present the debris spectra and results from the analyses. The results indicating the highest velocities occur at points on the south side of facilities. Within the above equation, the licensee applied the average of the maximum velocity component normal to the structure from the six storms considered. Tables 13–16 (FHRR Tables 2.10-5 and 2.10-6) present the debris force (in kips) and the debris impact pressure (in kips/ft^2).

$$F = \frac{\pi\, W\, V_b\, C_i C_o C_D C_B R_{max}}{2\, g\, \Delta t}$$

Equation 2.10-1

where:

F	resulting force in lb
W	weight of the object in lb, taken from Table 2.10-3
V_b	velocity of the object in fps (assumed equal to the velocity of water per Reference 2.10-13), taken as the maximum average current velocity perpendicular to the structure at each critical point around the PSEG Site, see Table 2.10-4
C_i	importance coefficient, taken as 1.3 per Table C5-1 of Reference 2.10-13
C_O	orientation coefficient, taken as 0.8 per Reference 2.10-13
C_D	depth coefficient, taken as 1 per Table C5-2 of Reference 2.10-13 for a still water depth greater than 1.5 m (5 ft)
C_B	blockage coefficient, taken as 1 per Table C5-3 of Reference 2.10-13
R_{max}	maximum response ratio, conservatively taken as 1.8 per Table C5-4 of Reference 2.10-13
Δt	impact duration, taken as 0.03 seconds per Reference 2.10-13
g	acceleration due to gravity, 32.2 ft/s^2

Figure 16. Equation applied by the licensee to evaluate impact loads for floodborne debris (FHRR Equation 2.10-1)

Table 11. Waterborne debris spectra (FHRR Table 2.10-3)

Debris Type	Diameter (ft.)	Height (ft.)	Length (ft.)	Width (ft.)	Contact Area[(a)] (ft^2)	Tare Weight (lb)	Gross Weight (lb)
1.Cryogenic Dewar	2.17	4.42	N/A	N/A	3.7	300	1,700
2.Referigerant Recovery Tank	2.50	4.78	N/A	N/A	4.9	334	1,334
3.IBC Tank	N/A	4.50	4.5	6.3	20.3	1,047	13,456
4.Bullet Resistant Enclosure	N/A	7.33	8	6	44.0	N/A	12,250
5. Intermodal Shipping Container, 53'	N/A	9.50	53	8.5	80.8	10,310	67,200
6. Intermodal Shipping Container, 45'	N/A	9.50	45	8.5	80.8	10,910	74,960
7. House	N/A	10.0	20	10	50.0	N/A	4,000
8. Recreational Boat	N/A	4.0	34	6	0.55	N/A	25,000
9. Automobile	N/A	4.5	16.4	6.6	20.0	N/A	4,000
10. Telephone Pole	1.125	N/A	35	N/A	0.99	N/A	1,490

a) Contact area is defined as the smallest surface area of the object. Contact area for items 7 through 9 are from Reference 2.10-10.

Table 12. Maximum impact current velocity at each location (FHRR Table 2.10-4)

Location	Max Impact Current Velocity for Each Storm (ft/sec) 11	20	21	25	33	60	Average Velocity (ft/sec)
1	0.31	0.45	0.44	0.56	0.59	0.49	0.48
2	0.12	0.22	0.17	0.15	0.33	0.22	0.20
3	2.88	2.77	2.81	2.48	2.44	2.46	2.64
4	0.67	3.26	0.62	0.77	0.58	0.61	1.09
5	0.09	1.05	0.00	0.82	0.44	0.99	0.57
6	0.89	0.80	0.85	0.72	0.73	0.90	0.82
7	2.82	2.78	2.79	2.07	2.11	2.29	2.48
8	2.26	2.62	2.26	1.51	1.74	2.59	2.16
9	0.39	0.33	0.36	0.36	0.40	0.39	0.37
10	0.45	0.39	0.42	0.29	0.36	0.35	0.38
11	0.18	0.12	0.14	0.16	0.17	0.13	0.15
12	0.18	0.12	0.14	0.16	0.17	0.13	0.15
13	0.18	0.12	0.14	0.16	0.17	0.13	0.15
14	0.42	0.38	0.42	0.31	0.26	0.37	0.36
15	0.47	0.48	0.45	0.38	0.41	0.40	0.43
16	3.39	3.62	3.26	2.70	3.00	2.90	3.15
17	Not Applicable[(a)]						
18	1.31	1.16	1.25	0.99	1.07	1.03	1.13
19	0.00	0.00	0.00	0.15	0.00	0.00	0.03
20	0.34	0.37	0.34	0.26	0.28	0.27	0.31
21	Not Applicable[(a)]						
22	0.34	0.30	0.33	0.38	0.30	0.32	0.33

a) Current velocity is not applicable at this location as it is interior to the building.
b) Maximum current impact velocity is the velocity component normal to the structure.

Table 13. Maximum debris force (kips) at each location (FHRR Table 2.10-5, sheet 1)

Debris Type		Maximum Debris Force (kips) at each Location 1	2	3	4	5	6	7	8	9	10
1. Cryogenic Dewar	Tare	0.43	0.18	2.41	0.99	0.52	0.75	2.26	1.98	0.34	0.34
2. Refrigerant Recovery Tank	Tare	0.48	0.20	2.69	1.10	0.57	0.83	2.52	2.20	0.38	0.38
	Gross	1.93	0.82	10.73	4.41	2.30	3.32	10.06	8.79	1.51	1.53
3. IBC Tank	Tare	1.51	0.64	8.42	3.46	1.80	2.60	7.90	6.90	1.19	1.20
4. BRE (Elevated)	Gross	17.72	7.51	98.52	40.49	21.08	30.45	92.38	80.74	13.88	14.01
5. Shipping Container, 53'	Tare	14.92	6.32	82.92	34.08	17.74	25.62	77.75	67.95	11.68	11.79
	Gross	97.23	41.19	540.47	222.11	115.65	167.02	506.74	442.91	76.12	76.88
6. Shipping Container, 45'	Tare	15.78	6.69	87.75	36.06	18.78	27.12	82.27	71.91	12.36	12.48
	Gross	108.45	45.95	602.88	247.76	129.01	186.31	565.26	494.06	84.91	85.75
7. Office Trailer	Gross	5.79	2.45	32.17	13.22	6.88	9.94	30.16	26.36	4.53	4.58
8. Recreational Boat	Gross	36.17	15.32	201.07	82.63	43.03	62.14	188.52	164.77	28.32	28.60
9. Automobile	Gross	5.79	2.45	32.17	13.22	6.88	9.94	30.16	26.36	4.53	4.58
10. Telephone Pole	Gross	2.16	0.91	11.98	4.92	2.56	3.70	11.24	9.82	1.69	1.70

Table 14. Maximum debris force (kips) at each location (FHRR Table 2.10-5, sheet 2)

Debris Type		Maximum Debris Force (kips) at each Location											
		11	12	13	14	15	16	17	18	19	20	21	22
1. Cryogenic Dewar	Tare	0.14	0.14	0.14	0.33	0.39	2.87	0	1.04	0.02	0.28	0	0.30
2. Referigerant Recovery Tank	Tare	0.15	0.15	0.15	0.37	0.44	3.20	0	1.15	0.03	0.31	0	0.34
	Gross	0.61	0.61	0.61	1.46	1.75	12.78	0	4.61	0.10	1.26	0	1.34
3. IBC Tank	Tare	0.48	0.48	0.48	1.15	1.38	10.03	0	3.62	0.08	0.99	0	1.05
4. BRE (Elevated)	Gross	5.63	5.63	5.63	13.41	16.12	117.38	0	42.33	0.94	11.55	0	12.30
5. Shipping Container, 53'	Tare	4.74	4.74	4.74	11.29	13.56	98.79	0	35.63	0.79	9.72	0	10.35
	Gross	30.88	30.88	30.88	73.59	88.40	643.93	0	232.23	5.15	63.34	0	67.48
6. Shipping Container, 45'	Tare	5.01	5.01	5.01	11.95	14.35	104.54	0	37.70	0.84	10.28	0	10.95
	Gross	34.45	34.45	34.45	82.08	98.61	718.28	0	259.05	5.74	70.65	0	75.27
7. Office Trailer	Gross	1.84	1.84	1.84	4.38	5.26	38.33	0	13.82	0.31	3.77	0	4.02
8. Recreational Boat	Gross	11.49	11.49	11.49	27.38	32.89	239.56	0	86.40	1.91	23.56	0	25.10
9. Automobile	Gross	1.84	1.84	1.84	4.38	5.26	38.33	0	13.82	0.31	3.77	0	4.02
10. Telephone Pole	Gross	0.68	0.68	0.68	1.63	1.96	14.28	0	5.15	0.11	1.40	0	1.50

Table 15. Maximum debris pressure (kips/ft^2) at each location (FHRR Table 2.10-6, sheet 1)

Debris Type		Maximum Debris Pressure (kips/ft^2) at each Location									
		1	2	3	4	5	6	7	8	9	10
1. Cryogenic Dewar	Tare	0.12	0.05	0.65	0.27	0.14	0.20	0.61	0.53	0.09	0.09
2. Refrigerant Recovery Tank	Tare	0.10	0.04	0.55	0.23	0.12	0.17	0.51	0.45	0.08	0.08
	Gross	0.39	0.17	2.19	0.90	0.47	0.68	2.05	1.79	0.31	0.31
3. IBC Tank	Tare	0.07	0.03	0.41	0.17	0.09	0.13	0.39	0.34	0.06	0.06
4. BRE (Elevated)	Gross	0.40	0.17	2.24	0.92	0.48	0.69	2.10	1.84	0.32	0.32
5. Shipping Container, 53'	Tare	0.18	0.08	1.03	0.42	0.22	0.32	0.96	0.84	0.14	0.15
	Gross	1.20	0.51	6.69	2.75	1.43	2.07	6.27	5.48	0.94	0.95
6. Shipping Container, 45'	Tare	0.20	0.08	1.09	0.45	0.23	0.34	1.02	0.89	0.15	0.15
	Gross	1.34	0.57	7.46	3.07	1.60	2.31	7.00	6.11	1.05	1.06
7. Office Trailer	Gross	0.12	0.05	0.64	0.26	0.14	0.20	0.60	0.53	0.09	0.09
8. Recreational Boat	Gross	65.77	27.86	365.60	150.25	78.23	112.98	342.78	299.61	51.49	52.00
9. Automobile	Gross	0.29	0.12	1.61	0.66	0.34	0.50	1.51	1.32	0.23	0.23
10. Telephone Pole	Gross	2.18	0.92	12.11	4.97	2.59	3.74	11.35	9.92	1.70	1.72

Table 16. Maximum debris force (kips/ft^2) at each location (FHRR Table 2.10-6, sheet 2)

Debris Type		Maximum Debris Pressure (kips/ft^2) at each Location											
		11	12	13	14	15	16	17	18	19	20	21	22
1. Cryogenic Dewar	Tare	0.04	0.04	0.04	0.09	0.11	0.78	0	0.28	0.01	0.08	0	0.08
2. Refrigerant Recovery Tank	Tare	0.03	0.03	0.03	0.07	0.09	0.65	0	0.24	0.01	0.06	0	0.07
	Gross	0.13	0.13	0.13	0.30	0.36	2.61	0	0.94	0.02	0.26	0	0.27
3. IBC Tank	Tare	0.02	0.02	0.02	0.06	0.07	0.49	0	0.18	0.00	0.05	0	0.05
4. BRE (Elevated)	Gross	0.13	0.13	0.13	0.30	0.37	2.67	0	0.96	0.02	0.26	0	0.28
5. Shipping Container, 53'	Tare	0.06	0.06	0.06	0.14	0.17	1.22	0	0.44	0.01	0.12	0	0.13
	Gross	0.38	0.38	0.38	0.91	1.09	7.97	0	2.87	0.06	0.78	0	0.84
6. Shipping Container, 45'	Tare	0.06	0.06	0.06	0.15	0.18	1.29	0	0.47	0.01	0.13	0	0.14
	Gross	0.43	0.43	0.43	1.02	1.22	8.89	0	3.21	0.07	0.87	0	0.93
7. Office Trailer	Gross	0.04	0.04	0.04	0.09	0.11	0.77	0	0.28	0.01	0.08	0	0.08
8. Recreational Boat	Gross	20.89	20.89	20.89	49.78	59.80	435.58	0	157.09	3.48	42.84	0	45.64
9. Automobile	Gross	0.09	0.09	0.09	0.22	0.26	1.92	0	0.69	0.02	0.19	0	0.20
10. Telephone Pole	Gross	0.69	0.69	0.69	1.65	1.98	14.42	0	5.20	0.12	1.42	0	1.51

The licensee states the debris impact force and pressure values presented in FHRR Tables 2.10-5 and 2.10-6 (Tables 13 – 16) represent a conservative assessment of the effect of debris in a hurricane-induced storm surge event. The licensee opines conservative aspects include

that the analysis did not consider deformation of the debris, which will lessen forces, and the analysis assumes debris interact with critical structure during the time of maximum water velocity.

FHRR Section 3.10.2 states that the CLB for the SGS site does not include evaluation of the effects of waterborne debris impacts on safety related structures. The CLB analysis included evaluation of seismic effects, tornado loads, and hurricane wind and hydrodynamic loads. Since the effects of waterborne debris were not evaluated in the current licensing or design basis the FHRR states these effects are not considered bounded, and proposed interim actions based on the results of this assessment are described in FHRR Chapter 4. FHRR Section 4.2 states that PSEG will further analyze the structural capability of flood protection features to withstand the effects of debris impact during the storm surge event.

Staff Technical Evaluation
Staff reviewed the licensee's technical analysis and approach for the debris and waterborne projectiles at the SGS site. The staff review and analysis confirms the debris load calculations presented in the FHRR, given the FHRR forcing. The FHRR calculations apply an average velocity (from six representative storms) to develop the load. A more conservative approach applies the maximum velocity from individual storms — shown to triple the load from 250 kips to 750 kips for one specific case.

Staff technical evaluation conclusions require additional PSEG analyses that FHRR Section 4.2 states will further analyze the structural capability of flood protection features to withstand the effects of debris impact during the storm surge event.

13.0 Effects of Sediment Erosion and Deposition.

Information Submitted by the Licensee
FHRR Section 2.4.6 describes the analyses conducted to estimate storm surge-induced sediment erosion and deposition at the site. The licensee states that normal tidal currents range from 2–3 ft/s (0.6 – 0.9 m/s) at the PSEG SGS site. The licensee states that the ADCIRC+SWAN results for Storm 11 show maximum velocities near the service water intake structures have a similar magnitude to the normal tidal currents. For their analysis, the licensee considers that the calculated current velocities are capable of resuspension of natural sediment, which can cause erosion. The analysis focuses on post-event deposition in Delaware Bay and localized effects of sedimentation and erosion at the PSEG SGS site.

The licensee's sediment deposition analysis applies an assumed total suspended solids (TSS) concentration of 5,000 mg/L (0.31 lb/ft^3) that deposits after a hurricane passage. The assumed TSS concentration applies results from studies of concentrations after hurricane passage in other estuaries (Jones et al., 1992; Walker, 2001; Wilber et al., 2006) that indicate post-storm TSS levels approximately tenfold more than pre-storm levels. Cook et al. (2007) indicates that TSS levels near the bottom of the Delaware Bay normally range between 450 and 525 mg/L (0.028 and 0.033 pb/ft^3) during the flood and ebb periods in the tidal cycle. Therefore, FHRR Section 2.4.6 indicates, the TSS levels immediately after a hurricane could reach 5,000 mg/L (0.31 lb/ft^3), 10 times greater than the normal level. The licensee's calculations, based on the assumption that 5,000 mg/L of total suspended solids deposit shortly after the passage of the hurricane, indicate that deposition is not expected to exceed 2 in. (0.17 ft, 0.05 m) of sediment.

FHRR Section 2.4.6 states that areas surrounding the safety-related structures are highly compacted and covered with pavement, concrete, or gravel. Model results indicate a maximum average water velocity of approximately 4 ft/sec was recorded for locations around the PSEG SGS site (FHRR Figure 2.4-4). The licensee opines that the potential for erosion during a hurricane event is low, since the flow velocities are less than the maximum permissible velocities shown in Chow (1959).

Based on the FHRR sedimentation and deposition analyses, the licensee opines that the effect of the storm surge related sediment deposition and erosion is not expected to adversely affect operation of safety-related SSC.

Staff Technical Evaluation
The FHRR analyses reference the Cook et al. (2007) study with suspended sediment data within Delaware Bay. Studies from other estuaries (Jones et al., 1992; Walker, 2001; Wilber et al., 2006) indicate post-storm TSS levels can reach approximately ten times pre-storm levels. The licensee's analysis assumes a gross deposition with all total suspended solids in the water column deposited within a few days after a storm's passage. The licensee's calculates the gross deposition results in an even layer of 2 in. (0.17 ft, 0.05 m) of sediment throughout the entire estuary. However, deposition can feature localized areas of higher and lower levels depending on site-specific features that influence currents and waves. Even if deposition levels increase by an order of magnitude near the site, the deposition would remain below levels capable of adversely affecting safety-related SSC.

The FHRR analysis of erosion includes the licensee's observation that the areas surrounding the safety-related SSC are highly compacted and covered with pavement, concrete, or gravel. Thus, the licensee opines that the storm-induced erosion does not present a risk to safety-related SSC. While currents are not strong enough to induce significant erosion, the licensee's analysis indicates that currents are strong enough to limit sedimentation near the safety-related SSC. Given the on-site ground covering, on-site erosion near safety related SCC should be minimal.

Review of the FHRR's analysis indicates that erosion and deposition should not adversely affect operation of safety-related SSC.

14.0 Consideration of Other Site-Related Evaluation Criteria.

Information Submitted by the Licensee
The storm surge analysis did not focus on seismic hazards

Staff Technical Evaluation
The FHRR did not focus on seismic hazards. The FHRR applied a probabilistic approach to develop the storm surge stillwater and total water levels. NRC staff developed independent deterministic storm surge estimates (stillwater and total water level) based on state-of-the-art numerical models and probable maximum hurricane forcing. The NRC staff'sindependent storm surge levels agree well with the current design basis stillwater and total water levels for the site.

15.0 Conclusion

The staff confirmed the licensee's conclusion that the reevaluated hazard for flooding from storm surge is bounded by the current design basis flood hazard.

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