Nuclear Development
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PSEG
Power LLC

ND-2011-0066
December 9, 2011

U. S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555-0001

Subject: PSEG Early Site Permit Application
Docket No. 52-043
Supplemental Response to Request for Additional Information, RAI No. 39, Probable Maximum Surge and Seiche Flooding

References:
1) PSEG Power, LLC letter to USNRC, Application for Early Site Permit for the PSEG Site, dated May 25, 2010
2) RAI No. 39, SRP Section: 02.04.05 – Probable Maximum Surge and Seiche Flooding, dated October 27, 2011 (eRAI 6051)
3) PSEG Power, LLC letter to USNRC, ND-2011-0064, Response to Request for Additional Information, RAI No. 39, Probable Maximum Surge and Seiche Flooding, dated November 22, 2011

The purpose of this letter is to respond to the request for additional information (RAI) identified in Reference 2 above. This RAI addresses Probable Maximum Surge and Seiche Flooding, as described in Section 2.4.5 of the Site Safety Analysis Report (SSAR), as submitted in Part 2 of the PSEG Site Early Site Permit Application, Revision 0. A partial response to RAI No. 39 was provided in Reference 3.

Enclosure 1 provides our response for RAI No. 39, Questions No. 02.04.05-3, 02.04.05-4, 02.04.05-6, 02.04.05-8, 02.04.05-9, 02.04.05-10, and 02.04.05-11. Enclosure 2 includes the revisions to SSAR Subsection 2.4.5 resulting from our responses to Question Nos. 02.04.05-1, 02.04.05-2, and 02.04.05-5 provided in Reference 3. Enclosure 4 includes the new regulatory commitments established in this submittal.

If any additional information is needed, please contact David Robillard, PSEG Nuclear Development Licensing Engineer, at (856) 339-7914.

D079
NRO

I declare under penalty of perjury that the foregoing is true and correct. Executed on the 9th day of December, 2011.

Sincerely,

James Mallon
Nuclear Development
Early Site Permit Manager
PSEG Power, LLC

Enclosure 1: Response to NRC Request for Additional Information, RAI No. 39, Questions No. 02.04.05-3, 02.04.05-4, 02.04.05-6, 02.04.05-8, 02.04.05-9, 02.04.05-10 and 02.04.05-11; SRP Section: 2.4.5 – Probable Maximum Surge and Seiche Flooding
Enclosure 2: Proposed Revisions Part 2 – Site Safety Analysis Report (SSAR) Subsection 2.4.5 – Probable Maximum Surge and Seiche Flooding
Enclosure 3: CD-ROM Containing Digital Input Files
Enclosure 4: Summary of Regulatory Commitments

cc: USNRC Project Manager, Division of New Reactor Licensing, PSEG Site (w/enclosures)
USNRC, Environmental Project Manager, Division of Site and Environmental Reviews (w/enclosures)
USNRC Region I, Regional Administrator (w/enclosures)

PSEG Letter ND-2011-0066, dated December 9, 2011

ENCLOSURE 1

RESPONSE to RAI No. 39

QUESTION Nos.
02.04.05-3
02.04.05-4
02.04.05-6
02.04.05-8
02.04.05-9
02.04.05-10
02.04.05-11

Response to RAI No. 39, Question 02.04.05-3:

In Reference 2, the NRC staff asked PSEG for information regarding Probable Maximum Surge and Seiche Flooding, as described in Section 2.4.5 of the Site Safety Analysis Report. The specific request for Question 02.04.05-3 was:

> *To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge are needed. The storm surge induced by the PMH can be estimated as recommended by Regulatory Guide 1.59, supplemented by current best practices. SSAR Section 2.4.5.1 discusses modification of the default wind drag coefficient applied in the Bodine model to estimate storm surge at the proposed site. The NRC staff requests that PSEG provide results of sensitivity testing undertaken to evaluate the effect of modifying the default wind drag coefficient in the Bodine storm surge model, or justify why this is not necessary.*

PSEG Response to NRC RAI:

In response to this RAI, PSEG conducted sensitivity testing. The basis for the range of values used in sensitivity testing and results are presented in this response. Recent research discussed below shows that the wind stress coefficient does not increase monotonically at hurricane force winds, and that current research indicates *k* is "capped" at high wind speeds. This research indicates that *k* does not exceed 3.0×10^{-6} at high wind speeds that occur during the site-specific PMH. Based on these findings the functional relationship defining *k* was modified in accordance with ANS/ANSI 2.8 (SSAR Reference 2.4.5-1).

Authors cited in this response use several different phrases and symbols to express the relationship between wind stress and wind speed. These phrases and symbols are directly related. The terminology and any necessary conversion factors between those phrases and symbols in cited sources are defined in this paragraph. Wind stress is generally found to be proportional to the square of the wind speed. SSAR Reference 2.4.5-2 defines a dimensionless wind stress coefficient *k* relating the square of the wind speed to wind stress. Zedler, *et al.* (2009; Reference RAI-39-3-2) simply refer to the "drag coefficient" without using any symbol. SSAR References 2.4.5-7 and 2.4.5-26 express comparable results using a drag coefficient C_d and Amorocho and DeVries (1980; Reference RAI-39-3-1) define a related coefficient C_{10}, where. $C_{10} = C_d$ for wind speeds at 10 meters above water surface, the elevation at which wind speed is determined by SSAR Reference 2.4.5-18 and used in PMH wind stress calculations.

The wind stress coefficient *k* (SSAR Subsection 2.4.5.1) is defined as:

$$k = \frac{\rho_a}{\rho w} C_d$$ (SSAR Reference 2.4.5-23, pages 9 and 13)

Where ρ_a = density of air
ρ_w = density of water

$$\rho_a = 1.0\ x\ 10^{-3}\ \frac{g}{cm^3} \qquad \frac{\rho_a}{\rho_w} = 1.0\ x\ 10^{-3}$$

$$\rho_w = 1.0 \frac{g}{cm^3}$$

$$k = 1.0\ x\ 10^{-3}\ C_d = 1.0\ x\ 10^{-3}\ C_{10}$$

In the remainder of this response, values of related terms are expressed as *k* using the relationships defined in these equations.

Bodine (SSAR Reference 2.4.5-2) defined k as a function of wind speed given by:

$$k = K_1 \qquad \text{for } W \le W_c$$

$$k = K_1 + K_2\left(1 - \frac{W_c}{W}\right)^2 \qquad \text{for } W \ge W_c$$

Where K_1 = 1.1×10^{-6}
K_2 = 2.5×10^{-6}
W_c = critical wind speed = 14 kt
W = wind speed (kt)

Additional measurements of *k* have been published since 1971 (References RAI-39-3-1, RAI-39-3-2, RAI-39-3-3, RAI-39-3-4, and SSAR References 2.4.5-7 and 2.4.5-26). Several functional relationships have been presented between 1977 through 1981 (References RAI-39-3-1, RAI-39-3-3, and RAI-39-3-4). The functional relationships cited above, however, are not based on direct measurements of *k* at hurricane force winds. These functional relationships have been used by other researchers in estimating hurricane storm surges (e.g., References RAI-39-3-6, RAI-39-3-7, RAI-39-3-8, and RAI-3-9). Garratt's relationship (Reference RAI 39-3-3) was based on indirect methods at wind speeds greater than 41 kt, and errors associated with those indirect methods are large (SSAR Reference 2.4.5-26). Amorocho and DeVries (Reference RAI-39-3-1) found that *k* reaches a maximum value of 2.54×10^{-6} for wind speeds exceeding 42 kt. Large and Pond (Reference RAI-39-3-4) presents a relationship that is applicable up to a maximum wind speed of 49 kt.

Powell and others (2003, SSAR Reference 2.4.5-26) used advanced instrumentation (GPS dropwindsonde), not available in 1971, and measured *k* in a 1998 hurricane at wind speeds from 54 to 97 kt. They state that reliable measurements of *k* at wind speeds greater than 49 kt had not been made prior to their study. Their measurements showed that the mean of the observed *k* does not exceed 2.6×10^{-6} regardless of wind speed. These results, as well as the Amorocho and DeVries relationship, support an upper bound on the wind setup coefficient of 2.6×10^{-6}.

Donelan and others (2004, SSAR Reference 2.4.5-7) conducted laboratory experiments showing that k does not exceed 2.5 x 10^{-6} for wind speeds between 64 and 97 kt.

Observations made with more advanced measuring devices after 2000, have found that *k* does not exceed 3.0 x 10^{-6} at hurricane force winds, with maximum ("capped") values in the range of 2.5 x 10^{-6} to 3.0 x 10^{-6}.

Further support that *k* does not exceed 3.0 x 10^{-6} during hurricanes include SSAR References 2.4.5-3, 2.4.5-4, and RAI-39-3-10, in which Bretschneider asserts that 3.0 x 10^{-6} is appropriate for design hurricane calculations; NOAA (SSAR Reference 2.4.5-23) uses k = 3.0 x 10^{-6} in the SLOSH model; and Dietrich and others (Reference RAI-39-3-6), validated a storm surge model for Hurricane Gustav (2008), with *k* "capped" at 3.0 x 10^{-6} in the sector to the right of the storm track, which is the only sector simulated in the SSAR.

Sensitivity of the peak surge calculated at the mouth of the Delaware Bay for the PMH to three alternative relationships between *k* and wind speed is presented in Table RAI 39-3-1. The PMH is defined to possess a Radius to Maximum Winds, R = 28 nautical miles and Forward Speed, T = 26 kt. In each alternative relationship, the default Bodine relationship is used for relatively low wind speeds. In the first alternative relationship, *k* is defined by the default Bodine relationship for *W* less than 56 kt; and "capped" at 2.5 x 10^{-6} for $W \geq 56$ kt. In the second alternative, the relationship is used to determine the PMH surge, and *k* is defined by the default Bodine relationship for *W* less than 109 kt and is "capped" at 3.0 x 10^{-6} for $W \geq 109$ kt. The third alternative is the default Bodine wind drag relationship, i.e. "uncapped". Figure RAI-39-3-1 illustrates the three alternative relationships between wind stress and wind speed used in sensitivity testing.

Table RAI 39-3-1. Sensitivity of the Maximum Surge at the Mouth of the Delaware Bay to Alternative Relationships between *k* and *W*

Alternative	Description	Maximum Surge (ft,NAVD)
Alternative 1	*k* "capped" at 2.5 x 10^{-6}	18.8
Alternative 2	*k* "capped" at 3.0 x 10^{-6} as used to determine the PMH surge	20.9
Alternative 3	*k* "uncapped" default Bodine relationship (3.08 x 10^{-6} at max. wind speed = 128 kt)(SSAR Reference 2.4.5-2)	21.2

Alternative 3 exhibited a maximum value of *k*, at any time during the simulation, of 3.08 x 10^{-6} associated with the maximum wind speed of 128 kt. The maximum value of *k* in Alternative 3 is 3% greater than the maximum value of *k* in the relationship used to determine the PMH surge (Alternative 2); and the maximum surge at the mouth of the Delaware Bay in Alternative 3 is 1% greater than the maximum surge for Alternative 2. Alternative 1 has a maximum value of *k* 19% less than the maximum value of *k* in Alternative 3; and the maximum surge is smaller by 11%. These results show that the maximum surge is not as sensitive as the maximum value of *k* used in the surge computation.

Observations of the wind stress coefficient *k* made with more advanced measuring devices after 2000, have found that *k* does not exceed 3.0 x 10^{-6} at hurricane force winds. Sensitivity testing indicates that the maximum surge when *k* is capped at 3.0 x 10^{-6} produces a maximum surge 1% less at the mouth of the Delaware Bay for the site-specific PMH when compared with the "uncapped" Bodine relationship in SSAR Reference 2.4.5-2. The PMH surge at the mouth of the Delaware Bay determined with a wind stress coefficient "capped" at 3.0 x 10^{-6} is supported by recent research results and is conservative.

References:

RAI-39-3-1 Amorocho, J. and J.J. DeVries. 1980. A New Evaluation of the Wind Stress Coefficient Over Water Surfaces. *J. Geophy. Res.* 85:433-42.

RAI-39-3-2 Zedler, S.E., P.P. Niiler, D. Stammer, E. Terrill, and J. Morzel. 2009. Ocean's Response to Hurricane Frances and its Implications for Drag Coefficient Parameterization at High Wind Speeds. *J. Geophys. Res.* 114:C04016 (19 pp).

RAI-39-3-3 Garratt, J.R. 1977. Review of Drag Coefficients over Oceans and Continents. *Monthly Weather Rev. 105:915-29.*

RAI-39-3-4 Large, W.G. and S. Pond. 1981. Open Ocean Momentum Flux Measurements in Moderate to Strong Winds. *J. Phys. Oceanog.*11:324-336.

RAI-39-3-6 Dietrich, J.C., J.J. Westerink, A.B.Kennedy, J.M. Smith, R. E. Jensen, M. Zijlema, L.H. Holthuijsen, C. Dawson, R.A. Luettich,Jr,.M.D. Powell, V.J. Cardone, A.T. Cox, G.W. Stone, H. Pourtaheri, M.E. Hope, S. Tanaka, L.G. Westerink, H. J. Westerink, and Z. Cobell. 2011. Hurricane Gustav (2008) Waves and Storm Surge: Hindcast, Synoptic Analysis, and Validation in Southern Louisiana. *Monthly Weather Rev.* 139:2488-2522.

RAI-39-3-7 Hagen, S.C., D. Dietsche, and Y. Funakoshi. 2004. Storm Tide Hindcasts for Hurricane Hugo: Into an Estuarine and Riverine System. *Advances in Hydro-Science and Engineering*, Volume VI.

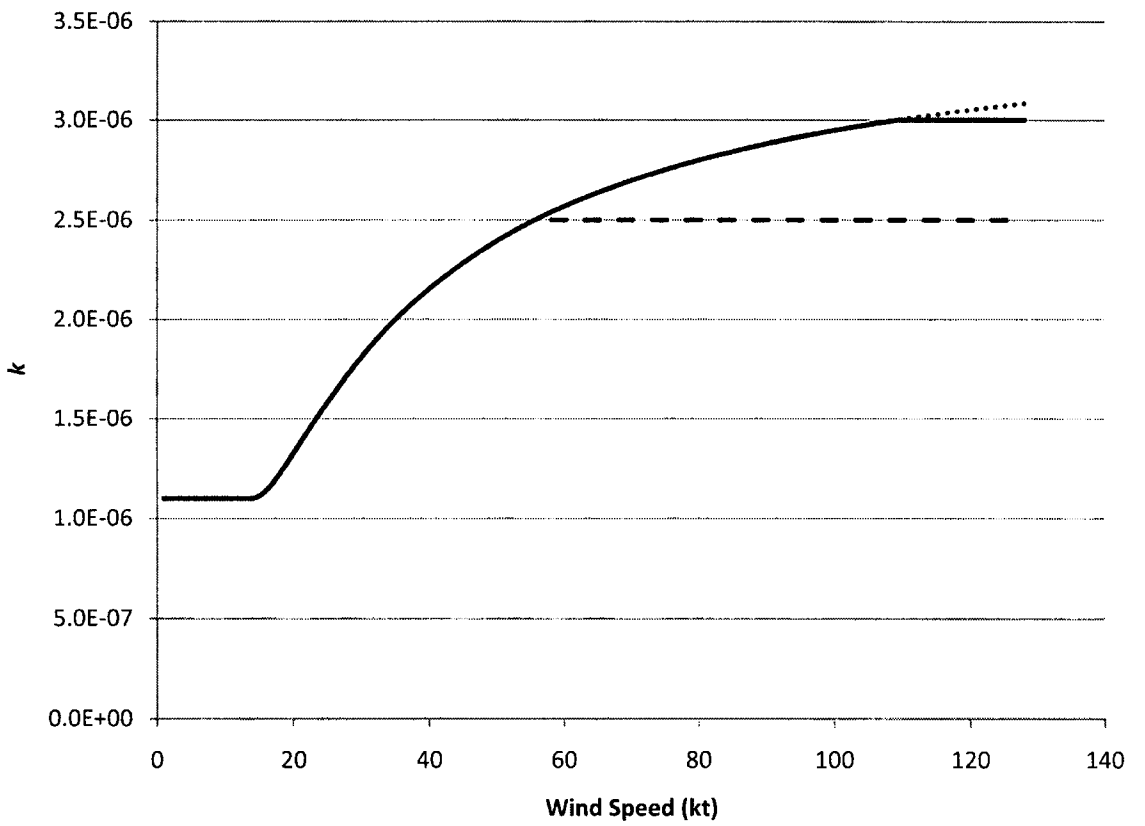
RAI-39-3-8 Dietsche, D., S.C. Hagen, and P. Bacopoulos. 2007. Storm Surge Simulations for Hurricane Hugo (1989) On the Significance of Inundation Areas. *J. of Waterway, Port, Coastal, and Ocean Engin.*, May/June 2007: 183-91.

RAI-39-3-9 Irish, J.L., D.T. Resio, and J.J. Ratcliff. The Influence of Storm Size on Hurricane Surge. 2008. *J. of Phys. Oceanog.*38:2003-13.

RAI-39-3-10 Dames & Moore, "Report Consultation, Storm Surge Re-Evaluation, Salem Nuclear Generating Station, Salem, New Jersey," Public Service Electric & Gas Company, January 24, 1973.

Associated PSEG Site ESP Application Revisions:

None.



Legend

- Default Bodine Relationship
- Capped at 3.0 x 10^{-6}
- Capped at 2.5 x 10^{-6}

PSEG Power, LLC
PSEG Site ESPA
Part 2, RAI No. 39

Alternative Relationships between Wind Speed and Wind Stress Coefficient

FIGURE RAI 39-3-1 Rev. 0

Response to RAI No. 39, Question 02.04.05-4:

In Reference 2, the specific request for Question 02.04.05-4 was:

> *To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge are needed. The storm surge induced by the PMH can be estimated as recommended by Regulatory Guide 1.59, supplemented by current best practices. SSAR Sections 2.4.5.2.1 and 2.4.5.2.2 discuss application of the Bodine storm surge model to develop the surge at the mouth of Delaware Bay. The NRC staff requests that PSEG provide the Bodine model input files and information on boundary conditions applied in the modeling, or justify why this is not necessary.*

PSEG Response to NRC RAI:

The Bodine model input files are provided in Enclosure 3. The initial condition is that water levels are at the specified tide elevation in feet, Mean Low Water (ft, MLW). A no-flow condition exists at the mouth of the bay, which is an intrinsic feature of the bathystrophic assumption used in the Bodine model (SSAR Reference 2.4.5-2). The assumption of no flow at the mouth of the Delaware Bay is conservative because surge flow through the mouth into the Bay would result in a reduction in the water surface elevation at the mouth of the Bay. HEC-RAS does not establish a no-flow boundary at the mouth of the Delaware Bay. The combination of Bodine and HEC-RAS as presented in the SSAR accurately reproduces the surge at Reedy Point, DE, near the project site (see SSAR Subsection 2.4.5.2.2.

Associated PSEG Site ESP Application Revisions:

None.

Response to RAI No. 39, Question 02.04.05-6:

In Reference 2, the specific request for Question 02.04.05-6 was:

> *To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge are needed. The storm surge induced by the PMH can be estimated as recommended by Regulatory Guide 1.59, supplemented by current best practices. SSAR Sections 2.4.5.2.2.2 and 2.4.5.2.2.3 discuss application of the SLOSH storm surge model to develop the surge at the mouth of Delaware Bay and at the proposed project site. In addition, SSAR Sections 2.4.5.2.2.2 and 2.4.5.2.2.3 discuss and compare the model results (e.g., SLOSH versus Bodine); however, the storm characteristics for each method are not completely explained. The NRC staff requests that PSEG provide additional information on the storm parameters for the SLOSH model that developed the SLOSH Display Program V. 1.61g data applied in the study. This data will allow a more direct comparison of the storm parameters applied to develop the SLOSH (visualization program) and Bodine model storm surge estimates at the mouth of Delaware Bay and at the proposed project site.*
>
> *Discussions with the applicant during the site audit suggested that the applicant may obtain the SLOSH executable files and conduct SLOSH model simulations using site specific (e.g., PMH) storm characteristics. The NRC staff requests that PSEG provide results from any SLOSH simulations conducted by the applicant for storms with the PMH parameters.*

PSEG Response to NRC RAI:

Additional information on the storm parameters for the SLOSH model using the SLOSH Display Program applied in the study is provided in the following response.

For reference, the PMH that causes the maximum surge at the project site has the following meteorological parameters as defined in SSAR Subsection 2.4.5.1:

- Central Pressure, p_o = 26.65 in. of Hg
- Pressure Drop, Δp = 3.5 in. of Hg
- Radius of maximum winds, R = 28 nautical miles (NM)
- Forward speed, T = 26 kt.
- Coefficient related to density of air, K = 68 (when parameters are in units of in. of Hg. and kt.)
- Track direction, from 138 degrees (moving northwest)

The SLOSH Display Program v1.61g does not readily present all of the meteorological parameters of each storm provided in the Delaware Bay Basin v3 catalog. The SLOSH Display Program provides a listing of Maximum of Maximums (MOM) for each grid cell in the basin. From the listing of MOMs, the hurricane category, forward speed and track

direction can be determined for the MOMs as well as other storms presented in the SLOSH Display Program.

The highest surge elevation at the mouth of the Delaware Bay reported by the SLOSH Display Program is 17.6 ft, NAVD. This peak surge resulted from the following meteorological parameters provided as characterization by the SLOSH Display Program:

- Maximum winds, between 114 and 135 kt, based on Category 4 on Saffir Simpson Hurricane Wind Scale
- Forward speed, T = 52 kt.
- Track direction, from 112.5 degrees (moving towards west-northwest)

The storm evaluated by the SLOSH Display Program with the most similarity to the PMH that causes the maximum surge at the project site is a Category 4 storm traveling with a track of 135 (moving towards the northwest) with a static high tide at 2 ft, NAVD. The SLOSH Display Program reported a peak surge at the mouth of the Delaware Bay of 15.9 ft, NAVD for this storm.

The highest surge elevation at the project site reported by the SLOSH Display Program v1.61g is 22.8 ft, NAVD. This peak surge resulted from the following meteorological parameters provided in SLOSH Display Program outputs:

- Maximum winds, between 114 and 135 kt, based on Category 4 on Saffir Simpson Hurricane Wind Scale
- Forward speed, T ranging between 26 and 35 kt.
- Track direction, from 135 degrees (moving towards northwest)

At the time of the staff's hydrology audit in February 2011, PSEG intended to develop formal SLOSH model simulations for the proposed site using the PMH storm characteristics specified in NOAA Technical Report NWS 23. PSEG pursued developing this alternative flood analysis to compare the conservative results derived from the PMH flood level methodology described in the SSAR to results from SLOSH simulations in predicting PMH flood level for the site. This effort was based solely on project economics in that a potential reduction in overall PMH flood level via the use of SLOSH would have a direct effect on reducing overall excavation and backfill costs for the project.

PSEG has since decided not to pursue the investigation of developing SLOSH model simulations for PMH flood level at the site. This decision is based in part on the potential for future regulatory driven re-analysis of flood hazards resulting from the Fukushima incident, as well as the uncertainty in scope and schedule of the staff's initiative to revise Regulatory Guide 1.59, "Design Basis Floods for Nuclear Power Plants." PSEG has a high degree of confidence in the conservative PMH flood level results obtained using the methodology described in SSAR Section 2.4.5.

Associated PSEG Site ESP Application Revisions:

None.

Response to RAI No. 39, Question 02.04.05-8:

In Reference 2, the specific request for Question 02.04.05-8 was:

> *To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge are needed. The storm surge induced by the PMH can be estimated as recommended by Regulatory Guide 1.59, supplemented by current best practices. SSAR Section 2.4.5.2.2.3 discusses application of the Kamphuis wind setup model to estimate wind-induced water level changes from the mouth of Delaware Bay (developed by the Bodine model) to the project site approximately 80 km (50 miles) inland. The NRC staff requests that PSEG provide the model setup and input conditions applied to develop the wind-induced water level changes from the mouth of Delaware Bay to the project site. The NRC staff requests that PSEG provide information related to any additional analysis completed to understand how the shape of Delaware Bay would influence wind-induced water level changes in the bay.*

PSEG Response to NRC RAI:

Shear stress from wind blowing over the top of an enclosed water body, such as Delaware Bay, can pile water on the downwind shore, resulting in water level rise. This is a component of storm surge known as wind setup. Estimation of increases in water level due to wind setup has been presented by Bretschneider (SSAR Reference 2.4.5-4) and Kamphuis (SSAR Reference 2.4.5-10). Calculations are performed using the one-dimensional form to determine the steady state response to steady winds. This formula, as shown in Equation (RAI 39-8-1) (SSAR Reference 2.4.5-10), computes the maximum setup.

Equation (RAI 39-8-1)
$$\frac{dS}{dx} = \frac{k(U\cos\phi)^2}{gD}$$

Where:

S = Water level rise due to wind setup (m)
U = Wind speed over the fetch (m/sec.)
Φ = Angle between the wind direction and the fetch (degrees)
X = Fetch length (m)
D = Depth of water (still water depth plus surge and wind setup from previous step, m)
g = Gravitational constant ($9.81 m/s^2$)
κ = $k(1+\tau_b/\tau_s)$, $\tau_b/\tau_s=0.1$

The wind stress coefficient, k, is defined as discussed in SSAR Subsection 2.4.5.1 and discussed in additional detail in Response to RAI No. 39, Question 02.04.05-3.

Calculation of the wind setup is performed by numerically integrating the one-dimensional solution presented in Equation (RAI 39-8-1). A straight line fetch is generated and the depth is recorded at 10 meter intervals. Therefore, the numerical solution steps along the fetch, calculating the surge due to wind setup at every 10 meters, based on the depth at that point plus the surge of the previous step. Figure RAI 39-8-1 conceptually illustrates the wind setup geometry and procedure.

Wind Setup Fetch

Wind setup is estimated over a single fetch. The water level rise due to wind setup increases with fetch length, so the longest possible fetch within the Delaware Bay to the new plant is chosen as the length of water over which wind setup calculations is performed. The corresponding fetch is in the south-southeast direction from the new plant, at 153 degrees, as shown in Figure RAI 39-8-2.

Therefore, winds blowing from 153 degrees, from south-southeast create the maximum water level rise due to wind setup. This fetch direction is generally aligned with the maximum winds associated with the PMH, which blow from the southeast. This fetch is 53.1 Miles (85.4 kilometers) in length.

Wind Setup Inputs

Wind setup is a function of the depth of water under the fetch. The elevation of the bottom of the bay along the fetch line is determined using the Triangular Irregular Network described in SSAR Subsection 2.4.5.2.2.3. Water surface elevation is calculated using HEC-RAS at the midpoint of the fetch line, at approximately River Mile 25, and is used with the bathymetry along the fetch line to calculate the depth of water along the fetch line. Inputs from HEC-RAS are in feet NAVD and converted to meters (NAVD) for use by the wind setup spreadsheet model. Wind speed and direction are calculated at the midpoint of the fetch line during the PMH simulation in accordance with NOAA NWS 23 (SSAR Reference 2.4.5-19). Wind speeds are initially calculated in miles per hour, and the direction is referenced to the traverse line, as presented in SSAR Figure 2.4.5-1. These results are converted to wind speed in meters per second, and the direction is referenced to North as part of the wind setup calculations. As presented in the SSAR, these calculations are performed over a 4 hour period from 1 hour before the maximum still water level to 3 hours after the maximum still water level, to ensure that the maximum still water level is determined. In response to RAI No. 39, Question 02.04.05-9, the calculation period is extended an additional half hour.

Wind setup model input files are provided in Enclosure 3.

The shape of the Delaware Bay influences the propagation of waves through the bay. The amplitude of a long wave, such as a tidal wave, tends to increase as the cross-section converges, while frictional losses tend to reduce the amplitude (Reference RAI 39-8-1). The funnel shape of the Delaware River estuary results in an increase in mean tidal range from 4.08 ft. at River Mile (RM) 0 at Lewes, DE to 7.86 ft. at RM 126 at Newbold, PA (SSAR Table 2.4.1-3). The calibrated HEC-RAS model of the Delaware

River estuary accurately reproduced the amplification of the tidal range throughout the estuary (SSAR Subsection 2.4.3.1.1.3).

The shape of the Delaware Bay also results in amplification of storm surge through the bay as observed during historical hurricanes that made landfall to the west of the mouth of the bay, as summarized in SSAR Subsection 2.4.5.2.1. Storm surge at Reedy Point, DE (RM 58) near the project site and Philadelphia, PA (RM 100) exceeded the surge observed at the mouth of the bay during the Chesapeake-Potomac hurricane (1933), Floyd (1999), and Isabel (2003). The storm surge model used to determine the Probable Maximum Hurricane (PMH) surge in the SSAR, which consists of the Bodine model combined with the Kamphuis wind setup model, reproduces the amplification of the storm surge from the mouth of the bay to Reedy Point, DE, observed during the 1933 hurricane. The surge model calculates a peak surge at the mouth of the bay for the Chesapeake-Potomac hurricane of 1933 of 3.8 ft (observed 3.8 ft); and calculates a peak surge at Reedy Point, DE of 7.9 ft, compared with the observed peak surge of 7.7 ft (SSAR Reference 2.4.5-3 for observed surge).

The Kamphuis wind setup model determines the steady state slope of the water surface under steady winds. Hydrostatic pressure gradient balances the countering forces imposed by the steady wind stress. As shown by Equation (RAI 39-8-1), the slope of the water surface is greater in shallow water bodies. Variation in depth, a feature of the shape of the water body, affects the wind setup calculation.

Successful reproduction of the storm surge at both the mouth of the Delaware Bay and at Reedy Point, DE, demonstrates that the processes that affect storm surge within the bay, including the effects of the shape of the bay on wind-induced water level changes within the bay, are accurately represented using the combination of HEC-RAS and Kamphuis wind setup within the Delaware Bay. The resulting storm surge determined for the PMH amplifies through the Bay, with a calculated peak surge of 20.9 ft, NAVD at the mouth of the bay and a maximum still water level of 26.9 ft, NAVD at the project site at RM 52. The calculated amplification of 6 ft from the mouth to the project site compares conservatively with the amplification observed in SLOSH Display Program, where the surge amplifies from 19.8 ft, NAVD at the mouth to 25.3 ft, NAVD (after accounting for the 10% exceedance high tide) at the project site, an amplification of 5.5 ft (SSAR Subsections 2.4.5.2.2.2 and 2.4.5.2.2.3).

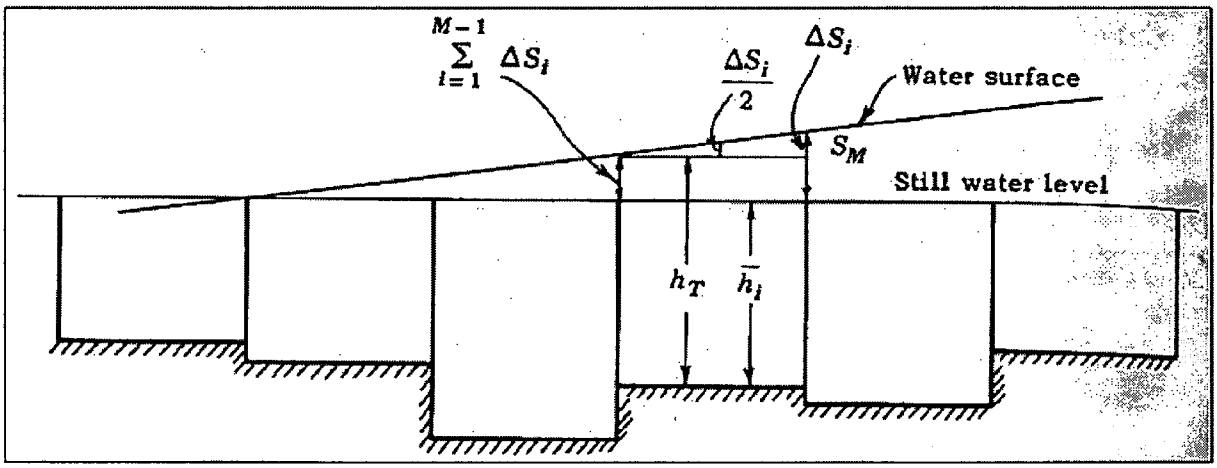
The analysis indicates that the combination of HEC-RAS and Kamphuis wind setup accurately accounts for the effect of water body shape on wind-induced water level changes in the Delaware Bay.

References:

RAI 39-8-1 Harleman, D.R.F. 1966. Tidal Dynamics in Estuaries, Part II: Real Estuaries. In *Estuary and Coastline Dynamics*. Ippen, A.T., Ed. McGraw-Hill: New York. p. 522-545.

Associated PSEG Site ESP Application Revisions:

None.



Source: SSAR Reference 2.4.5-5

PSEG Power, LLC PSEG Site ESPA Part 2, RAI No. 39
Theoretical Wind Setup Diagram FIGURE RAI 39-8-1 Rev. 0

LEGEND

★ PSEG Site

Wind Setup Fetch

N W E S

0 2 4 8 Nautical Miles

PSEG Power, LLC
PSEG Site ESPA

Part 2, RAI No. 39

Longest Possible Fetch Length
Across Delaware Bay
Figure RAI 39-8-2

Rev. 0

Response to RAI No. 39, Question 02.04.05-9:

In Reference 2, the specific request for Question 02.04.05-9 was:

> *To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge are needed. The storm surge induced by the PMH can be estimated as recommended by Regulatory Guide 1.59, supplemented by current best practices. SSAR Section 2.4.5.3.1 discusses the development of the wave runup at the project site. The NRC staff requests that PSEG provide plots that illustrate the wind vector directions and magnitudes at the time of, and at several times before and after, maximum PMH surge. NRC staff also requests that PSEG provide wave runup estimates at the proposed project site for these times.*

PSEG Response to NRC RAI:

Wind vectors throughout Delaware Bay, including some areas that may be inundated during the PMH surge, are provided in Figures RAI 39-9-1 through RAI 39-9-6 for simulation times 19.0, 20.0, 21.0, 22.0, 23.0, and 24.0 (hr). Wind vectors were calculated in accordance with NOAA NWS 23 (SSAR Reference 2.4.5-18). The ends of the lines nearest the arrows are the points at which the winds were determined. The lengths of the vectors are proportional to the wind speed.

Wave runup is calculated in accordance with ANSI/ANS 2.8, the USACE *Coastal Engineering Manual* and d'Angremond and Roode (SSAR References 2.4.5-1, 2.4.5-18, and 2.4.5-6); and additional details are provided in our response to RAI No. 39, Question 02.04.05-10. SSAR Subsection 2.4.5.3.2 presented wave runup results at the time of maximum still water level (Simulation Time = 21.0 hours), and at Simulation Time = 21.5 hours, the time of the design flooding condition when still water level plus wave runup reaches its maximum elevation. Additional results are provided in Table RAI 39-9-2 between Simulation Times of 19.0 to 24.0 hours. Table RAI 39-9-1 provides the wind speed and direction at the plant site at half hour increments between Simulation Times of 17.5 to 24.0 hours. Winds are averaged over a sufficient period to ensure that the wave field is fetch-limited and not duration-limited. The averaging times and the wind speed and direction used in the calculations of wave runup are detailed in Table RAI 39-9-1. The results of still water level, wave runup, and their sum are provided in Table RAI 39-9-2. Results of still water level throughout the simulation are provided in SSAR Figure 2.4.5-6.

Table RAI 39-9-1. Wind Speed and Direction Information Used to Calculate Wave Runup

Simulation Time (hr)	Speed (mph)	Direction (From)	Resultant Speed for Runup (mph)	Resultant Direction for Runup	Averaging Period (hrs)
17.5	73.2	NE			
18.0	83.6	NE			
18.5	91.0	NE			
19.0	100.6	NE	86.66	NE	17.5-19.0
19.5	112.4	ENE	95.69	ENE	18.0-19.5
20.0	123.2	E	109.74	ENE	19.0-20.0
20.5	132.4	ESE	117.95	E	19.5-20.5
21.0	134.4	SE	123.46	ESE	20.0-21.0
21.5	128.9	SE	119.37	ESE	20.0-21.5
22.0	118.6	SSE	114.83	ESE	20.0-22.0
22.5	108.2	SSE	102.85	ESE	19.5-22.5
23.0	97.0	SSE	91.88	ESE	19.0-23.0
23.5	87.8	S	82.65	ESE	18.5-23.5
24.0	78.9	S	92.64	S	22.5-24.0

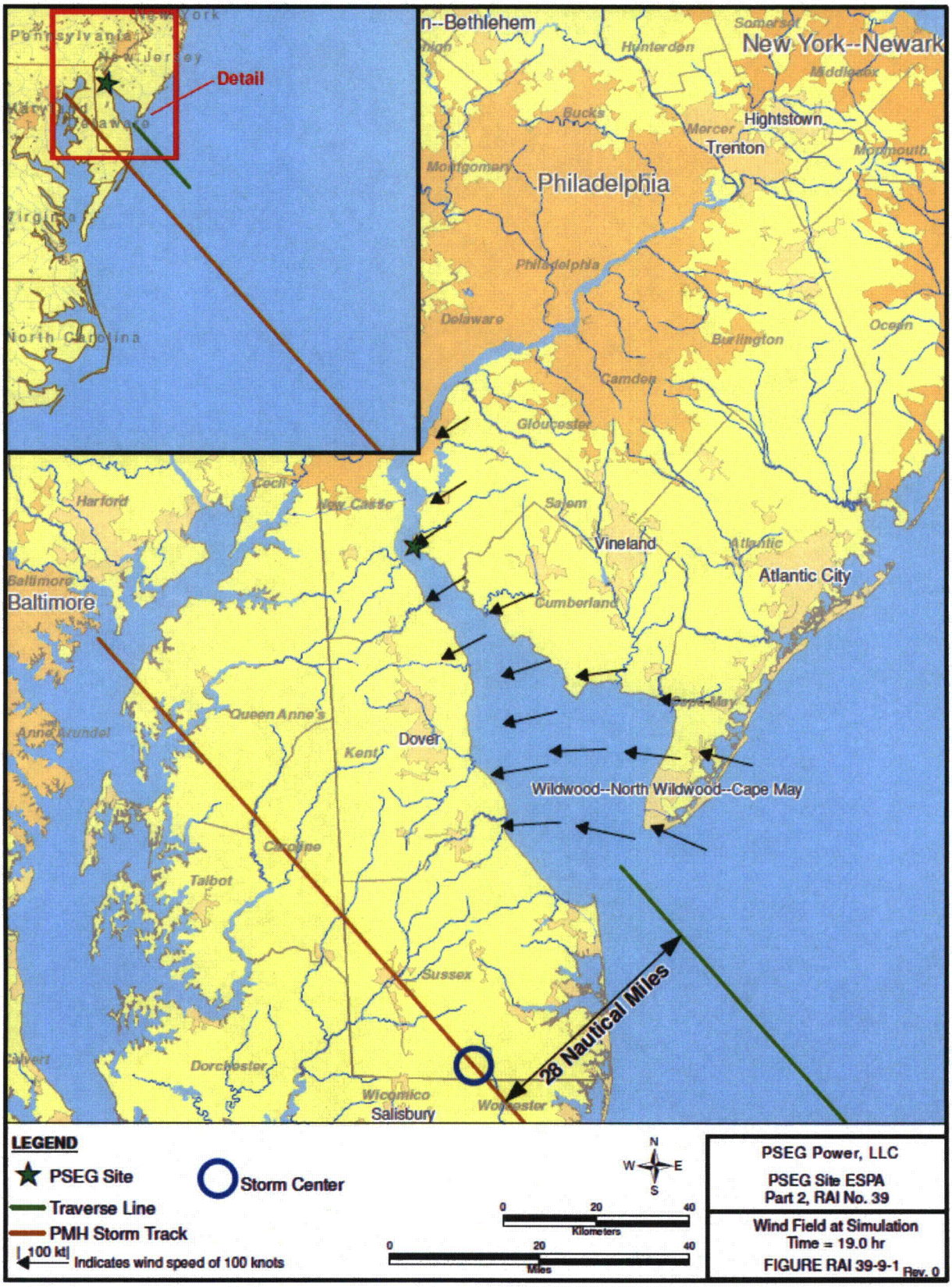
Table RAI 39-9-2. Still Water Levels and Wave Runup Results

Simulation Time (hr)	Stillwater Level (ft, NAVD)	Wave Runup (ft)	Hurricane Surge with Wave Runup (ft, NAVD)
19.0	9.3	3.1	12.4
19.5	15.4	4.9	20.3
20.0	23.3	6.5	29.7
20.5	26.9	7.0	33.8
21.0	26.7	7.9	34.6
21.5	25.7	7.6	33.3
22.0	24.4	7.3	31.7
22.5	22.9	6.6	29.5
23.0	21.5	6.0	27.5
23.5	19.5	5.4	25.0
24.0	17.0	5.9	22.9

Note: Flood Elevation does not include sea level rise.

Associated PSEG Site ESP Application Revisions:

None.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PMH Storm Track
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Indicates wind speed of 100 knots
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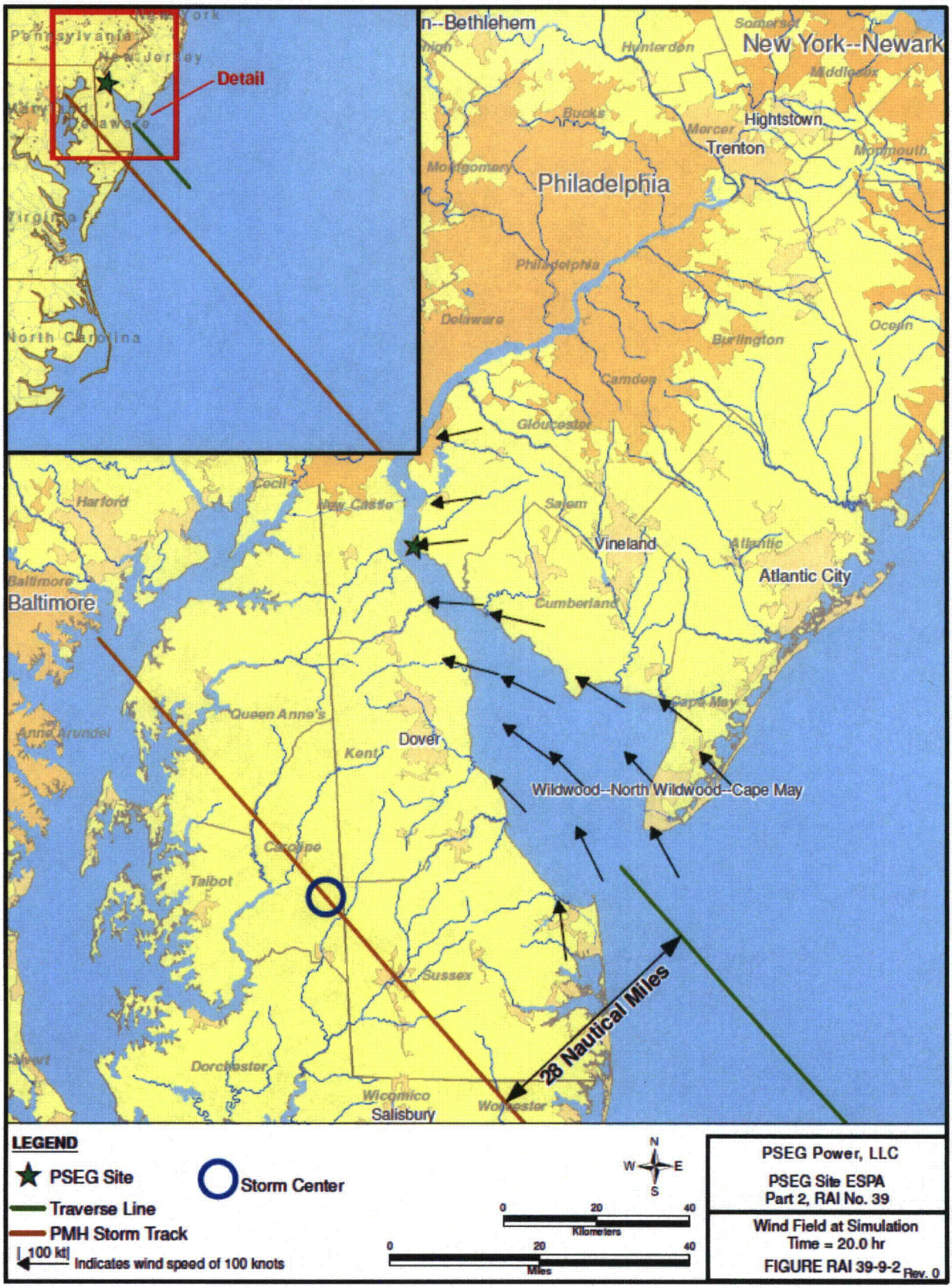
PSEG Power, LLC
PSEG Site ESPA
Part 2, RAI No. 39
Wind Field at Simulation
Time = 19.0 hr
FIGURE RAI 39-9-1
Rev. 0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Indicates wind speed of 100 knots
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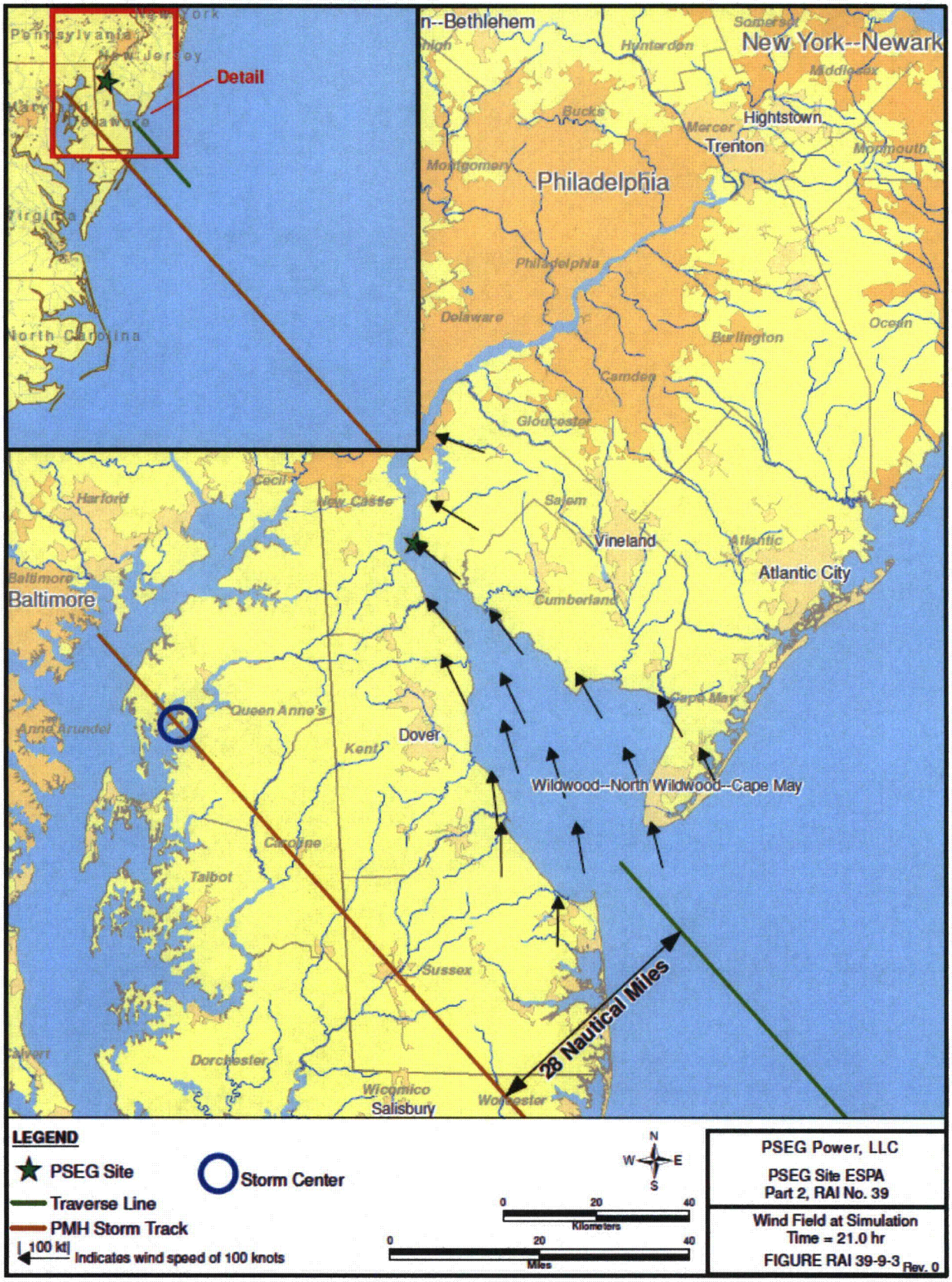
PSEG Power, LLC
PSEG Site ESPA
Part 2, RAI No. 39
Wind Field at Simulation
Time = 20.0 hr
FIGURE RAI 39-9-2 Rev. 0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PMH Storm Track
100 kt
Indicates wind speed of 100 knots
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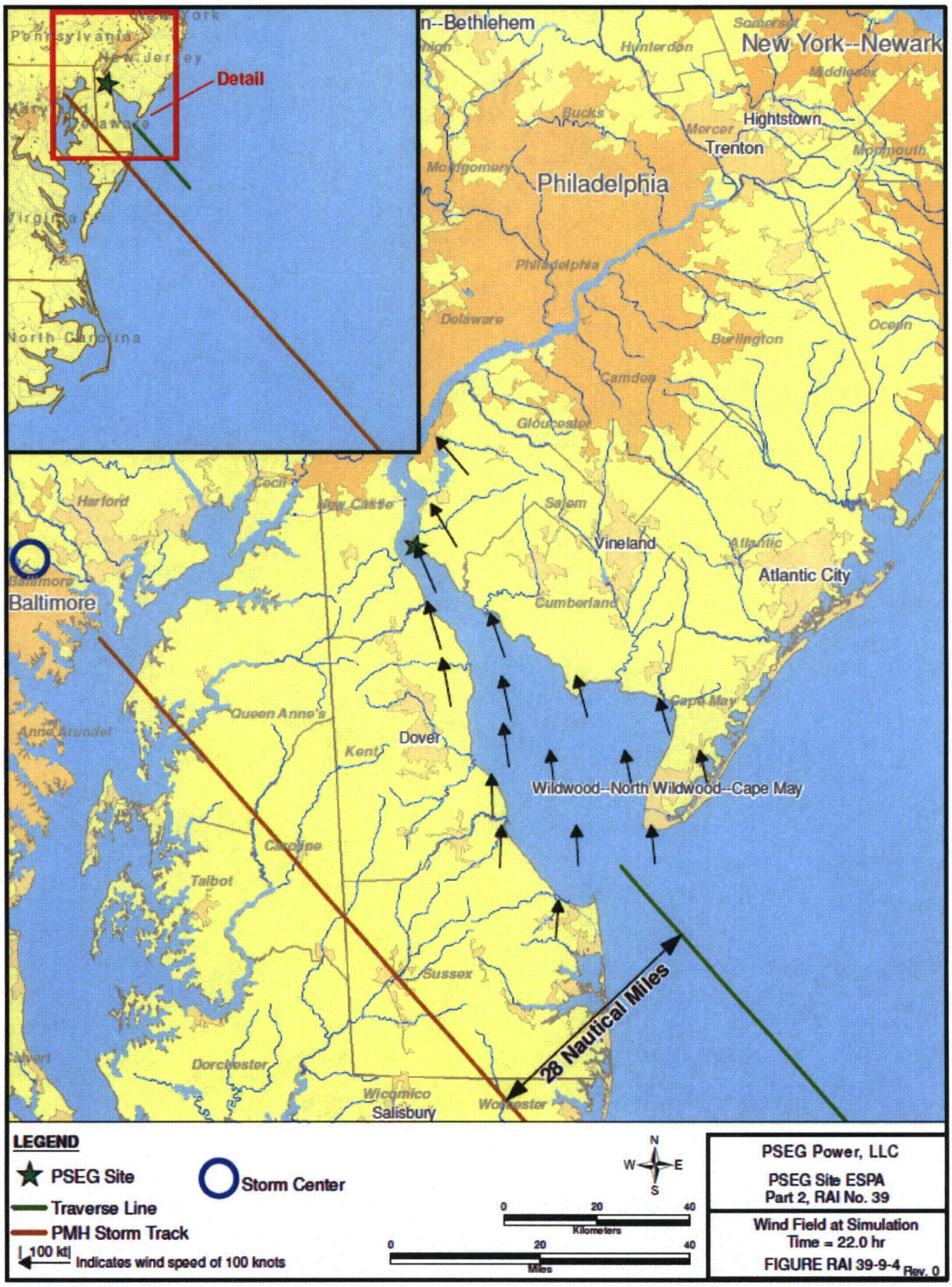
PSEG Power, LLC
PSEG Site ESPA
Part 2, RAI No. 39
Wind Field at Simulation
Time = 21.0 hr
FIGURE RAI 39-9-3
Rev. 0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PMH Storm Track
100 kt
Indicates wind speed of 100 knots
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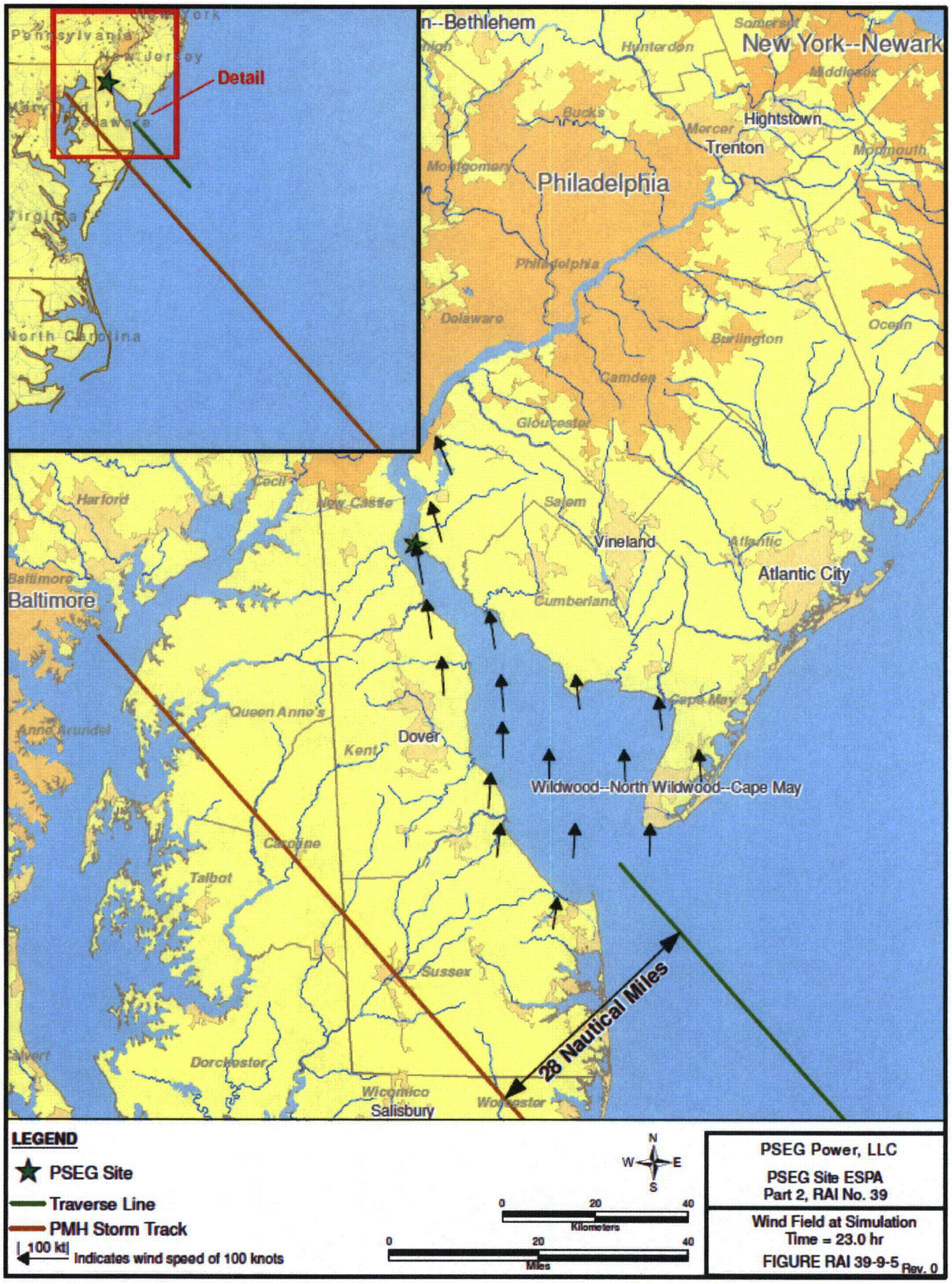
PSEG Power, LLC
PSEG Site ESPA
Part 2, RAI No. 39
Wind Field at Simulation
Time = 22.0 hr
FIGURE RAI 39-9-4
Rev. 0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Indicates wind speed of 100 knots
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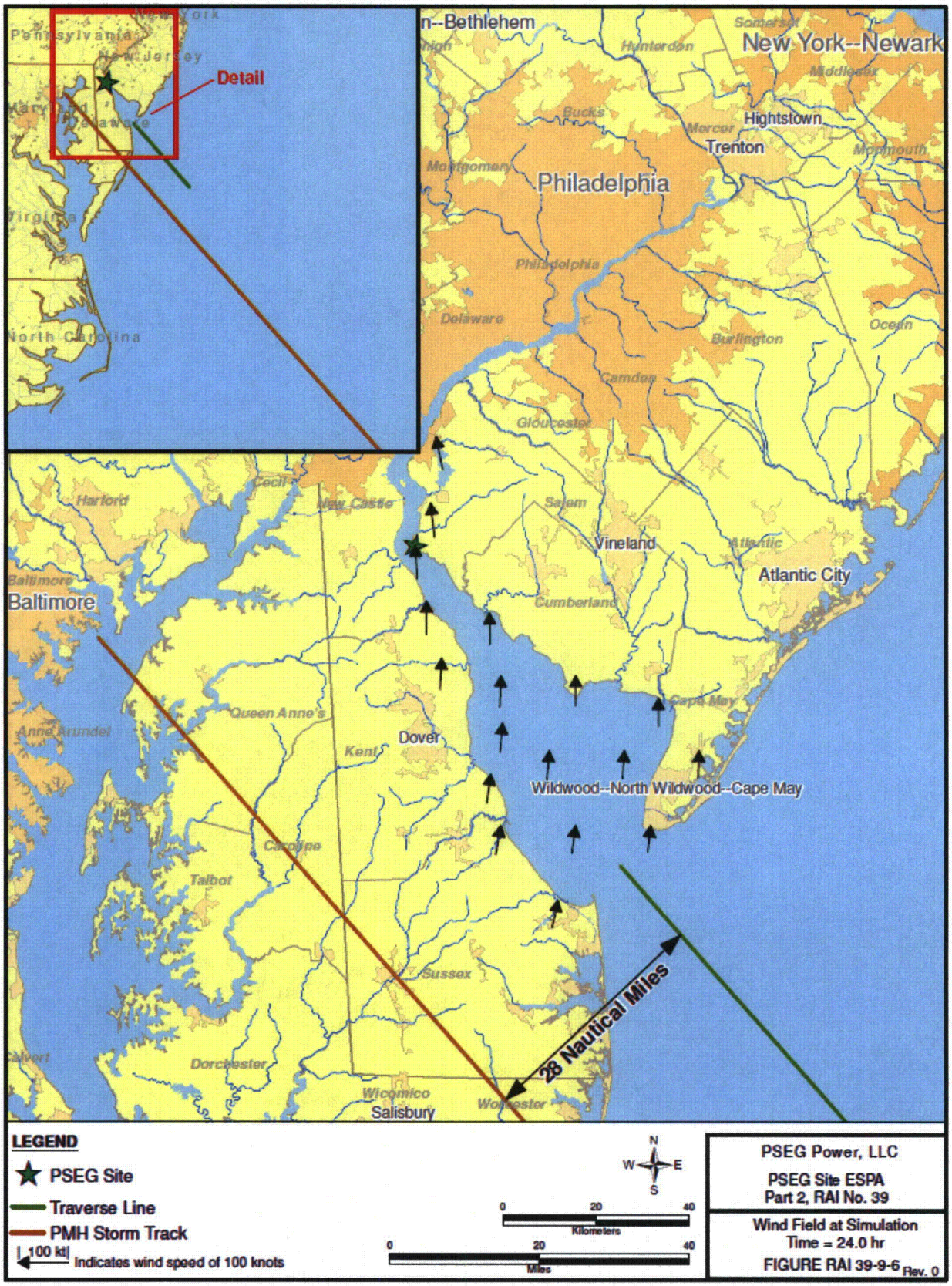
PSEG Power, LLC
PSEG Site ESPA
Part 2, RAI No. 39
Wind Field at Simulation
Time = 23.0 hr
FIGURE RAI 39-9-5 Rev. 0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PSEG Power, LLC
PSEG Site ESPA
Part 2, RAI No. 39
Wind Field at Simulation
Time = 24.0 hr
FIGURE RAI 39-9-6 Rev. 0

Response to RAI No. 39, Question 02.04.05-10:

In Reference 2, the specific request for Question 02.04.05-10 was:

> *To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge are needed. The storm surge induced by the PMH can be estimated as recommended by Regulatory Guide 1.59, supplemented by current best practices. SSAR Section 2.4.5.3.2 discusses the development of the wave runup estimate for the project site. The NRC staff requests that PSEG provide details of the equations and parameters applied to estimate the wind-induced wave runup at the project site. Specifically, the NRC staff requests that PSEG provide information on the equations applied, the wind speed averaging calculations, and the breaking ratio applied. In addition, the NRC staff requests that PSEG clearly define the wave heights (maximum versus significant) applied in the equations.*

PSEG Response to NRC RAI:

Estimation of Wave Runup

<u>Significant Wave Height and Spectral Peak Period</u>

The US Army Corps of Engineers Coastal Engineering Manual (CEM), Section II-2-2-b(1)(b) (SSAR Reference 2.4.5-27), presents the deepwater wave equations used to determine the significant wave height as a function of wind speed and fetch length. These calculations are performed on the assumption that the wind blows, with essentially constant direction, over a fetch for sufficient time to achieve steady-state, fetch limited values. Drag coefficient (C_D) and friction velocity (u_*), Equations (RAI 39-10-1) and (RAI 39-10-2), respectively, are functions of wind speed required to calculate the significant wave height.

Equation (RAI 39-10-1) $$C_D = 0.001(1.1 + 0.035U_{10m})$$

Equation (RAI 39-10-2) $$u_* = \sqrt{C_D U_{10m}^2}$$

Note: Equations (RAI-39-10-1) through (RAI-39-10-14) are from SSAR Reference 2.4.5-27.
Where U_{10m} = wind speed at 10 meters above water surface (m/sec.) and
u_* = friction velocity (m/sec.).

Using the two above factors, the significant wave height and the spectral peak period of the significant wave can be calculated for each straight line fetch distance (X) using several different durations and their associated wind speeds. Significant wave height (H_{m0}, m) and spectral peak period (T_P, sec.) can be calculated using Equations (RAI 39-10-3) and (RAI 39-10-4), respectively.

Equation (RAI 39-10-3) $$\frac{gH_{m0}}{u_*^2} = 4.13\left(10^{-2}\right)\left(\frac{gX}{u_*^2}\right)^{1/2}$$

Where g = acceleration of gravity (9.81 m/s^2) and
X = fetch distance (m)

Equation (RAI 39-10-4) $$\frac{gT_p}{u_*} = 0.751\left(\frac{gX}{u_*^2}\right)^{1/3}$$

Section II-2-2-b(1)(a) of the CEM discusses both fetch-limited and duration-limited wave development. Because the above equations are applied to scenarios that are assumed to be fetch-limited, a check for each duration is required to determine if the significant wave height is duration-limited. The limiting duration ($t_{x,u}$, s), as shown in Equation (RAI 39-10-5), is a function of fetch distance and wind speed. Therefore, the limiting duration is different for each separate wind speed duration, even if the fetch distance does not change.

Equation (RAI 39-10-5) $$t_{x,u} = 77.23\frac{X^{0.67}}{u^{0.34}g^{0.33}}$$

Where u = wind speed (m/s)

To be duration limited, the limiting duration is greater than the duration associated with the respective wind speed used in the above equation. However, the CEM suggests that conditions for meeting duration-limited wave growth are rarely met in nature. If the wave development calculations are duration limited for a given duration and wind speed, the wind vectors are averaged over sequentially increasing half-hour increments until the wind field is no longer duration limited. Equation (RAI 39-10-6) is used to determine the limiting duration along the given fetch length, as described in CEM Section II-2-2-b(1)(b).

Equation (RAI 39-10-6) $$\frac{gX_e}{u_*^2} = 5.23\left(10^{-3}\right)\left(\frac{gt}{u_*}\right)^{3/2}$$

Where t = wind duration (s)

The equations above, used to predict wave growth, are fetch-limited but are also deep water equations. Therefore, a check is made to ensure that the wave growth is not limited by depth across the fetch. This check is made by calculating the limiting peak wave period ($T_{P,L}$, s) and comparing it to the predicted wave period of the significant wave, as described in Section II-2-2-b(1)(d) of the CEM. Equation (RAI 39-10-7) shows that the limiting peak wave period is a function of the average depth of water across the fetch, d (m).

Equation (RAI 39-10-7) $$T_{P,L} \approx 9.78\left(\frac{d}{g}\right)^{1/2}$$

If the calculated peak period is greater than the limiting peak period, then the deepwater equations are not valid. In this case the maximum breaker height relationship defined in CEM Equation II-4-18 (based on the Miche criterion) is used and show in Equation (RAI 39-10-8).

Equation (RAI 39-10-8) $$H_{\max} = 0.14\, L \tanh\left(2\pi d / L\right)$$

Where H_{max} is the maximum breaker height (m);
d is the depth of water (m);
and L is the limiting peak wavelength (m).

When calculating wave runup, the smaller of the maximum breaker height or the maximum wave height, a function of the significant wave height, (1.67 H_{mo}) is used (Reference 2.4.5-1).

Significant Wave Height Fetch

Significant wave height is estimated along eight different fetch lines radiating out at selected compass directions from the new plant. Several fetches were chosen to ensure that the calculated wave height represents an accurate estimate of the highest possible level occurring during the PMH, due to the dramatic shift in wind direction as the storm center passes by the new plant. During the time of highest surge the winds are primarily originating out of the southeastern quadrant. Therefore, the eight fetches are between the East, Northeast and Southwest directions at common 22.5 degree differences. Fetches are generated in GIS to determine distance and depth/elevation profiles along the fetch using NOAA Bathymetry of Delaware Bay (SSAR Reference 2.4.5-19) and elevation data of the land surrounding Delaware Bay from USGS topography data (SSAR Reference 2.4.5-30). Each fetch is identified in Figure RAI 39-10-1.

Fetch lengths vary based on the overwater distance of the respective fetches. The still water level bathymetry data and overland elevation data are combined to create a fetch profile. The still water level is raised to theoretically submerge portions of adjacent land and increase depth over Delaware Bay in order to predict fetch lengths and the average depths along the fetches that occur during the PMH.

Adjusted Wind Speed

The overwater maximum wind speeds of the PMH are calculated in accordance with NOAA NWS 23 (SSAR Reference 2.4.5-18). Overwater maximum wind values calculated in accordance with SSAR Reference 2.4.5-18 correspond to the 10-meter, 10-minute average wind speed. However, the maximum significant wave height may not occur with the 10-minute duration wind speed. Wind speeds with durations lasting less than 10 minutes will be faster than the 10-minute wind speed, while wind speeds

with durations lasting longer will have slower magnitudes. Therefore, the estimated overwater maximum wind speed is adjusted for several different durations to use in significant wave height calculations. The CEM, Section II-2-1-a(3)(e), describes the method for wind speed adjustments as a function of duration. For durations below 3,600 sec. Equation (RAI 39-10-9) is used, while Equation (RAI 39-10-10) is used for duration adjustments above 3,600 sec.

Equation (RAI 39-10-9) $\frac{U_t}{U_{3600}} = 1.277 + 0.296\tanh\left(0.9\log_{10}\left(\frac{45}{t}\right)\right)$ If t < 3,600 sec.

Equation (RAI 39-10-10) $\frac{U_t}{U_{3600}} = 1.5334 - 0.15\log_{10}(t)$

If 3,600 sec < t < 36,000 sec.

Where: U_t = wind speed (m/s) with duration of "t" s.

Wave Runup

The CEM presents calculations for determining wave runup in Section II-4-4-a(1). The wave runup for regular, breaking waves on a smooth, impermeable slope is calculated by applying a factor to the significant wave height. This factor, the surf similarity parameter (ξ_0), is multiplied by significant wave height as shown in Equation (RAI 39-10-11), known as Hunt's formula.

Equation (RAI 39-10-11) $R = \xi_0 H_{max}$ for $0.1 < \xi_0 < 2.3$

Where R = wave runup (m)

The surf similarity parameter is a function of the significant wave height, in addition to the near-shore bottom slope and the deepwater wavelength, L_0 (m), as shown in Equation (RAI 39-10-12). This equation can be modified to extend the application to steep slopes by replacing tan β with sin β. The modified formula is valid for slopes from 1/10 to vertical. For near-shore bottom slopes greater than 1/10, becomes

Equation (RAI 39-10-12) $\xi_0 = \sin\beta\left(\frac{H_{max}}{L_0}\right)^{-1/2}$

Inputs for the surf similarity equation are known, except for the wavelength (L_0). The maximum wave height (H_{max}) has been calculated from Equation (RAI 39-10-8) and the near-shore bottom slope is obtained from site topography. The wavelength value used to calculate the surf similarity parameter is the deepwater wavelength. Equations presented in the CEM for deepwater wavelength are valid for relative depths (d/L) greater than 1/2. To determine the relative depth, the wavelength, L (m), must first be determined. The general case solution for determining wavelength, Equation (RAI 39-10-13), is shown in Section II-1-2-c(2)(e) of the CEM.

Equation (RAI 39-10-13) $$L = \frac{gT_P^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2 d}{gT_{P,C}^2}\right)}$$

Calculation of the relative depth (*d/L*) using the average depth across the fetch as the value *d* and dividing by the general case wavelength value shown in Equation (RAI 39-10-13), shows that the relative depth across all the fetches is not greater than 1/2. A simplified solution for calculation of the deepwater wavelength is shown in CEM Section II-1-2-c(2)(k). However, the CEM provides a useful equation for converting between the general case wavelength and the deepwater wavelength, as shown in Equation (RAI 39-10-14), in Section II-1-2-c(4)(b).

Equation (RAI 39-10-14) $$\frac{d}{L_0} = \frac{d}{L}\tanh\left(\frac{2\pi d}{L}\right)$$

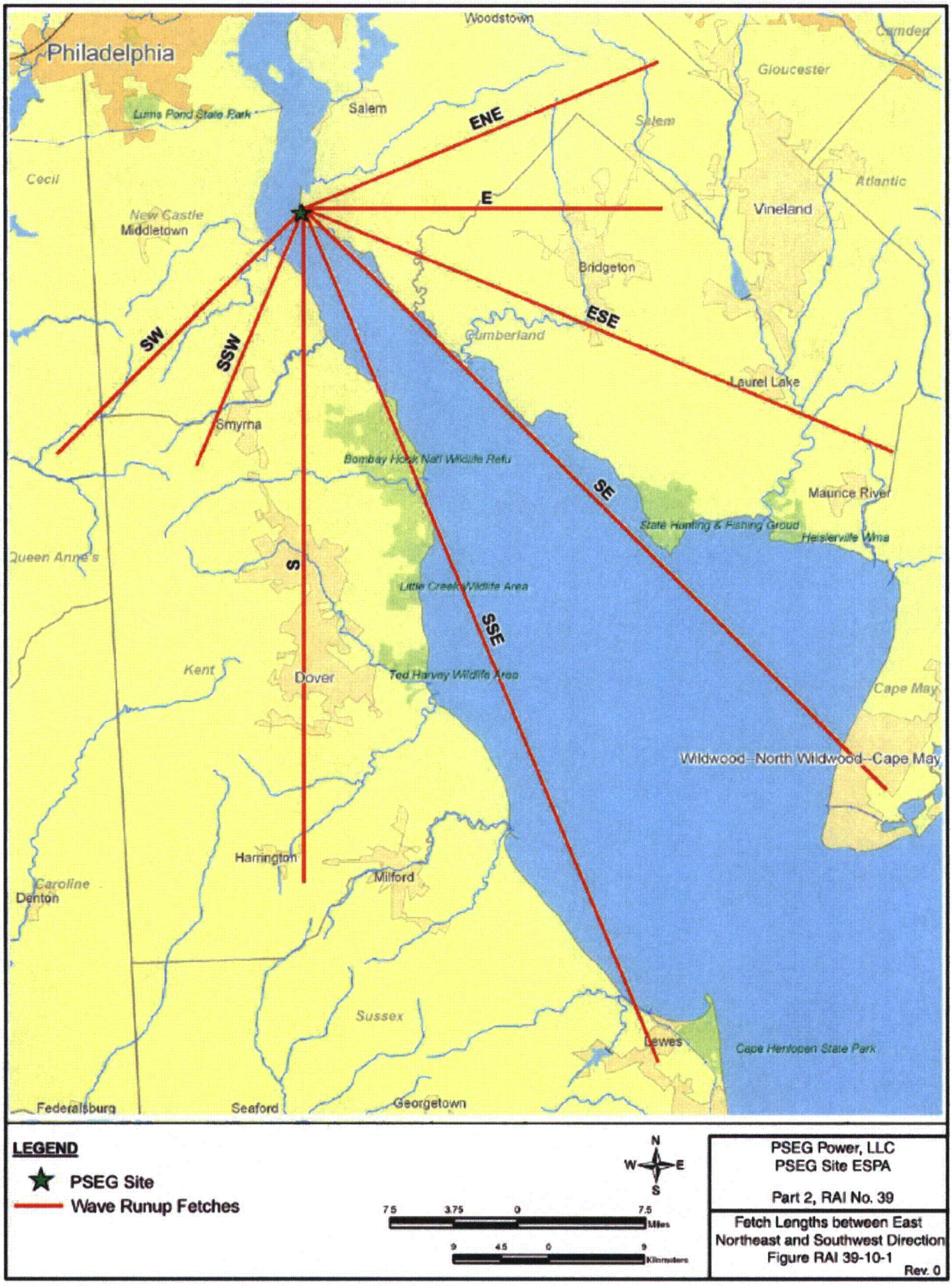
Once the deepwater wavelength is calculated, it can be used in Equation (RAI 39-10-12) to determine the surf similarity parameter (ξ), which is then applied to the maximum wave height or maximum breaker height, whichever is smaller, to estimate wave runup.

Runup calculated with the surf similarity parameter does not include a correction for dampening of runup due to the roughness of the ground. The height of run-up can be reduced by increasing the roughness or the permeability of the area being struck by incoming waves. Angremond and van Roode (SSAR Reference 2.4.5-6) present correction factors for roughness and permeability for application with the general run-up formula. These calculations assume that the site will include a rip-rap seawall, corresponding to a roughness correction factor (γ) of 0.5. This coefficient is multiplied by the wave runup value to account for actual conditions at the new plant. By augmenting Equation (RAI 39-10-11) with the roughness correction factor, a simple Equation (RAI-39-10-15) is generated to calculate the final wave runup value.

Equation (RAI 39-10-15) $$R = \xi_0 \gamma\, H_{\max}$$

Associated PSEG Site ESP Application Revisions:

None.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PSEG Power, LLC
PSEG Site ESPA
Part 2, RAI No. 39
Fetch Lengths between East Northeast and Southwest Direction
Figure RAI 39-10-1
Rev. 0

Response to RAI No. 39, Question 02.04.05-11:

In Reference 2, the specific request for Question 02.04.05-11 was:

> *To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge are needed. The storm surge induced by the PMH can be estimated as recommended by Regulatory Guide 1.59, supplemented by current best practices. SSAR Section 2.4.5.6 discusses the effects of sediment deposition and erosion at the project site. The NRC staff requests that PSEG provide additional information concerning the sediment dynamics near the proposed project site under hurricane-induced current velocities, including additional information to support the assumption of uniform deposition. Analysis of the two-dimensional (horizontal) distribution of sediment erosion and deposition may require estimation of the two-dimensional current velocity field (application of a two-dimensional hydrodynamic model).*

PSEG Response to NRC RAI:

Over the past 20 years numerous hydrodynamic modeling efforts involving the Delaware River Estuary have been conducted. The purposes have included hydrodynamics research; estimation of salinity, temperature and density distributions; sediment transport; and effects of sea level rise.

Based on review of the prior modeling efforts, the following general observations can be made:

- Most hydrodynamic modeling efforts for Delaware Estuary have been directed at characterizing salinity in the tidal Delaware River due to the critical effects of salinity on aquatic biota and the potential for use as a water supply source (Table RAI 39-11-1). Only recently has sediment transport become a topic of significant research interest, primarily as a result of ecological concerns (e.g., contaminated sediment and marsh sedimentation) (References RAI-39-11-3 and RAI-39-11-4).

- Two-dimensional (2D) depth-integrated models do not adequately incorporate the three-dimensional nature of the physics of transport, particularly sediment transport, required for prediction of sediment transport dynamics with sufficient geographic resolution (References RAI-39-11-3 and RAI-39-11-5).

- A limited number of three-dimensional (3D) models with sediment transport capabilities exist, but those models have not been widely evaluated or developed to be available for application to scenarios similar to that required for the Delaware River Estuary. With sufficient calibration and verification, 3D models have been found to be generally capable of reproducing

hydrodynamics; however, models for transport, typically salinity, have had limited success in reliably reproducing observed conditions (References RAI-39-11-2, RAI-39-11-3 and RAI-39-11-4).

- Estuarine circulation drives sediment transport. Identification and understanding of these forces under normal conditions, usually for limited geographic areas, is on-going through research efforts. Currently, a better understanding of basic physics of sediment transport is needed, and predictions of sediment transport for normal conditions are not reliable (References RAI-39-11-1, RAI-39-11-3, RAI-39-11-2, RAI-39-11-6, RAI-39-11-7, RAI-39-11-8 and SSAR Reference 2.4.3-3).

Çelebioğlu's research/academic modeling effort (Reference RAI-39-11-3) produced results most nearly relevant to the NRC's request. Çelebioğlu developed a hydrodynamic and sediment transport model for the entire estuary and simulated a hypothetical 100-year riverine flood event. He identified many of the considerations listed in the four basic conclusions above as reasons for need to develop a sediment transport model for the Delaware River Estuary.

Çelebioğlu's sediment transport model included suspended sediment, but no bed transport. He noted that for estuary conditions, bed erosion typically does not exceed a few sediment grain diameters. His model defined a 10 millimeter thick layer uniformly across the estuary as being an active layer subject to erosion. He concluded that the model tendency to overestimate suspended sediment concentrations may have been due to the thickness of that active layer being too large. Model predictions generally agreed with the estuary turbidity maximum observations in terms of longitudinal location and concentrations.

Çelebioğlu presented figures showing areas of net sediment erosion and deposition during a historic period of near normal conditions and during the 100-year flood simulation. While the figures encompass the entire estuary, the spatial resolution is sufficient to observe conditions in the general vicinity of the PSEG Site. The figures indicate model predictions of areas with tendencies for net sediment deposition and net erosion for the prescribed forcing conditions. There was no apparent tendency for large sedimentation depths in the vicinity of the proposed water intake. Deposition areas and erosion areas appear to be small relative to the scale of the estuary as a whole and relatively uniform. Although deposition and erosion amounts vary in the vicinity of the project site, the depth of erosion or deposition is not more than 2 millimeters during the 100-year storm event.

The PMH would create larger hydraulic gradients, higher maximum water levels, larger tidal flows, and increased erosion of sediments than those modeled by Çelebioğlu. The accompanying winds and other driving forces and circulation may also be different, and more dynamic, than the normal conditions or the 100-year flood conditions. However, the general patterns of erosion and deposition, while likely larger in magnitude, may ultimately be relatively similar to Çelebioğlu's results if simulated with the same model.

Çelebioğlu's results, indicating deposition amounts of not more than 2 millimeters (0.08 inches) associated with the 100-year riverine flood, support the finding of SSAR Subsection 2.4.5.6 that deposition amounts from the PMH are not likely to exceed 2 inches.

The river intake structure is the only safety-related system, structure or component (SSC) that will be affected due to sedimentation in the river as a result of the PMH. The forebay of the intake structure will be designed to accommodate any sedimentation, and maintenance dredging will be implemented as required. The design of the intake structure and the adjacent shoreline will also consider appropriate erosion control measures as discussed in SSAR Subsection 2.4.8.

Available models have not demonstrated predictive skill sufficient to accurately predict sediment deposition amounts at specific locations, such as the safety-related intake structure, from extreme events such as the PMH. Erosion and deposition associated with the PMH may be expected to vary within the Delaware River estuary, however the estimate provided in SSAR Subsection 2.4.5.6 is conservative and appropriate.

References:

RAI-39-11-1 Duzinski, P.J. Cross-Channel Transport in the Upper Delaware Estuary: Numerical Experiments for Contamination Vulnerability Assessment. Thesis. Drexel University. December 12, 2010. http://idea.library.drexel.edu/bitstream/1860/3410/1/Duzinski,%20Philip%20J..pdf

RAI-39-11-2 Stammerman, R. and M. Piasecki. Skill Assessment of the MARINA Hydrodynamic Code Using NOAA's Delaware Bay Estuary Modeling Evaluation Environment (MEE). ASCE Proceedings, Conference of the Eleventh International Conference on Estuarine and Coastal Modeling, November 6-9, 2009. http://ascelibrary.org/proceedings/resource/2/ascecp/388/41121/3_1?isAuthorized=no

RAI-39-11-3 Çelebioğlu, T.K. Simulation of Hydrodynamics and Sediment Transport Patterns in Delaware Bay. A Thesis submitted to the Faculty of Drexel University for degree of Doctor of Philosophy. November 2006.

RAI-39-11-4 Sherwood, C.R., R.P. Signell and C.K. Harris. Report of the Community Sediment Transport Modeling Workshop, June 22-23, 2000. Open File Report 01-32. USGS. http://walrus.wr.usgs.gov/transport/

RAI-39-11-5 Luettich, R.A.Jr., J.J. Westerink and N.W. Scheffner, November 1992. ADCIRC: An Advanced Three-Dimensional Circulation Model for Shelves, Coasts, and Estuaries. Report 1, Theory and Methodology of ADCIRC-

2DDI and ADCIRC-3DL. Technical Report DRP-92-6. U.S. Army Corps of Engineers.

RAI-39-11-6 Sommerfield, C.K., T.L. Cook, and K. Wong. 2006. Observations of tidal and springtime sediment transport in the upper Delaware Estuary. Estuarine, Coastal and Shelf Science 72 (207) 235-246

RAI-39-11-7 Sommerfield, C.K. and K. Wong, January 2011. Mechanisms of sediment flux and turbidity maintenance in the Delaware Estuary. Journal of Geophysical Research. Vol 116, C01005, doi: 10.1029/2010JC006462

RAI-39-11-8 Aubrey Consulting, Inc., 1995. Numerical Circulation Model Implementation: Salem and Hope Creek Nuclear Generating Stations, Field and Data Report. Cataumet, Massachusetts, Prepared for Public Service Enterprise Group (PSEG).

Associated PSEG Site ESP Application Revisions:

None.

PSEG Letter ND-2011-0066, dated December 9, 2011

ENCLOSURE 2

Proposed Revisions

Part 2 – Site Safety Analysis Report (SSAR)
Section 2.4.5 - Probable Maximum Surge and Seiche Flooding

additional effects of wind blowing over Delaware Bay, not simulated by HEC-RAS, are calculated using a formula for wind setup in semi-enclosed bodies of water as presented by Kamphuis (Reference 2.4.5-10).

Wave heights and wave runup coincident with the maximum still water level are determined in Subsection 2.4.5.3, using the hurricane wind field specified by NWS 23. Wave runup is calculated in accordance with USACE's *Coastal Engineering Manual* (Reference 2.4.5-27). Wave runup is added to the maximum still water level of the PMH surge.

In Subsection 2.4.5.4, the potential future effects of sea level rise are evaluated and added to the maximum water level from the PMH surge, which includes coincident wind wave activity, to determine the maximum future water level through the projected life of the new plant in Subsection 2.4.5.5.

Subsection 2.4.5.6 addresses sediment erosion and deposition associated with the PMH and their potential effects on the safety-related intake structure. Subsection 2.4.5.7 demonstrates that Delaware Bay does not resonate with meteorological or seismic forcing, providing further confirmation that the PMH surge as calculated in this section represents the most severe flooding that could occur at the new plant.

2.4.5.1 Probable Maximum Winds (PMW) and Associated Meteorological Parameters

This subsection identifies the meteorological characteristics of the PMH that causes the PMH surge and demonstrates that the PMH wind field represents the PMWS at the new plant location. The basic meteorological parameters that define the PMH are varied within limits given by NOAA (Reference 2.4.5-18) to determine the most severe combination that results. The detailed analysis of surge (in Subsection 2.4.5.2) is based on the most severe combination of these parameters.

The meteorological parameters associated with the PMH at the mouth of Delaware Bay are based on NWS 23. The mouth of Delaware Bay is defined as a point bisecting the line from Cape May, New Jersey (NJ) to Cape Henlopen, Delaware (DE), at latitude 38°51'30"N, longitude 75°01'30"W. At this location, NOAA provides the following meteorological parameters for the PMH:

- Central pressure, p_0 = 26.65 inches of mercury [in. of Mercury (Hg)].
- Pressure drop, Δp = 3.5 in. of Hg.
- Radius of maximum winds, R = from 11 to 28 nautical miles (NM).
- Forward speed, T = from 26 to 42 knots (kt).
- Coefficient related to density of air, K = 68 (when parameters are in units of in. of Hg and kt)
- Track direction, from 138 degrees (moving northwest).

The northwest track direction is perpendicular to bathymetric contours of the continental shelf offshore of the mouth of Delaware Bay (Reference 2.4.5-20). The track of this storm is illustrated in Figure 2.4.5-1. This track direction is within the range of directions that NOAA specifies for the PMH at the mouth of Delaware Bay. The inflow angle, which varies with distance from the storm center, is as specified by NOAA (Reference 2.4.5-18). From these parameters, the maximum winds range from 128 to 135 kt. Thus, the PMH is a relatively strong

ADD: ", as shown in Table 2.4.5-2."

ADD: (presented in Table 2.4.5-3)

ADD: This tide condition differs from the dynamic tidal input used in Subsection 2.4.5.2.2.

Category 4 hurricane by the Saffir-Simpson hurricane scale. Category 4 maximum sustained winds ranging from 114 to 135 kt.

NOAA specifies that the PMH may occur within a range of radius of maximum winds (R) and forward speed (T) (Reference 2.4.5-18). The method described in Subsection 2.4.5.2.2.2 is used to calculate the maximum storm surge at the open coast for nine possible combinations of R (11, 20, and 28 NM) with T (26, 34, and 42 kt) spanning the ranges of these parameters specified by NOAA. This analysis follows methodology described in ANSI/ANS-2.8-1992 Section 7.2.1.4. In these preliminary simulations, designed to identify the PMH producing the maximum storm surge, a static high tide condition is specified. These preliminary screening level analyses show that the surge at the mouth of Delaware Bay increases with R and T, with a maximum surge at the mouth of the bay when both R and T are high, specifically for the PMH with R = 28 NM, and T = 42 kt. This result is consistent with modeling performed in support of RG 1.59, which determined that the maximum surge at the coast consistently resulted from the PMH with high R and T.

The hurricane producing the maximum surge at the open coast may not produce maximum water levels in bays and estuaries. A storm that progresses at approximately the same speed as the tide propagates is expected to produce maximum surges within Delaware Bay (Reference 2.4.5-3). The speed of propagation of the tide in Delaware Bay is approximately 14 kt (References 2.4.5-3 and 2.4.5-8). Therefore, it may be expected that a PMH with a high forward speed (42 kt) may not produce the highest storm surge at the new plant location, even though it produces the highest surge at the mouth of Delaware Bay. A fast-moving storm moves ahead of the storm surge wave, while a slower moving storm tends to reinforce the surge. Since the surge at the mouth of Delaware Bay is strongly dependent on the radius of maximum winds, R, but weakly dependent on the forward speed, (T), the three storms with R = 28 are further investigated using HEC-RAS and the Kamphuis wind setup method to determine the potential effects of these storms on still water levels at the new plant location. This analysis shows that the PMH with R = 28 NM, and T = 26 kt produces the maximum surge at the new plant location consistent with Bretschneider's evaluation.

A PMH with R = 28 NM, and T = 26 kt produces the PMH surge at the new plant location. The PMH with R = 28 NM and T = 26 kt, is simulated in more detail in Subsections 2.4.5.2 and 2.4.5.3. Specifically, the storm is simulated with a fluctuating tide at the mouth of Delaware Bay, which produces the 10 percent exceedance high tide at the new plant location. The phase of the tide is established in relation to the development of the storm surge such that the 10 percent exceedance high tide coincides with the peak storm surge at the new plant location.

ADD: (presented in Table 2.4.5-3)

The pressure distribution and wind field associated with this storm are determined by NOAA (Reference 2.4.5-18) for a PMH. Wind speed and direction at any point T and R; the distance and angular orientation of the specified point relative to the center of the storm and the direction of storm movement. Wind speed varies with time at a point as the storm moves along its track relative to that point. Latitude and the density of air also affect the wind speed calculations. The maximum sustained winds over the ocean are calculated to be 128 kt; while the maximum winds over Delaware Bay are 126 kt, and maximum winds at the new plant location are 116 kt.

The HEC-RAS hydraulic simulation does not account for wind stress acting on water within Delaware Bay, therefore, the effect of wind stress within the bay is determined using the steady

Delaware River estuary 30 miles northeast of the PSEG Site) include the Chesapeake-Potomac hurricane (1933), Hazel (1954), Connie (1955), Floyd (1999), and Isabel (2003). Tracks of these storms are shown in Figure 2.4.5-2, based on data accessed from NOAA's Coastal Services Center (Reference 2.4.5-12). This list of storms is assembled from published descriptions of hurricanes producing significant surges in Delaware Bay and from review of hurricane tracks passing within 100 NM of the new plant location, while making landfall to the west of the mouth of Delaware Bay.

7.7

The Chesapeake-Potomac hurricane made landfall as a Category 1 hurricane near Currituck, North Carolina (NC). Traveling northwest, its track paralleled the western shore of Chesapeake Bay. It then turned northeasterly, bringing the storm center within 80 NM of the new plant location (Reference 2.4.5-12). It produced a maximum storm surge of 3.8 ft. near the mouth of Delaware Bay; ~~7.6~~ ft. at Reedy Point, DE (nearest tidal gage to the new plant location), and 7.1 ft. at Philadelphia (Reference 2.4.5-3).

Hazel made landfall as a Category 4 hurricane near the border of NC and South Carolina (SC). It moved north, with Category 1 status at its nearest approach to the new plant location, when the storm center was 98 NM west of the new plant location (Reference 2.4.5-12). Hazel produced a maximum storm surge at Philadelphia of 9.4 ft. (Reference 2.4.5-31).

Connie made landfall near Cape Charles, Virginia, as a tropical storm, and its inland track generally followed the eastern shore of Chesapeake Bay. At its nearest point, the storm center was within 43 NM of the new plant location (Reference 2.4.5-12). It produced a maximum surge at Philadelphia of 5.0 ft. (Reference 2.4.5-31).

The storm center of Floyd bypassed Delaware Bay to the south and east, 70 NM from the new plant location, moving northeast as a Category 1 hurricane (Reference 2.4.5-12). It produced a storm surge (after correcting for astronomical tide) of 3.0 ft. at Cape May, NJ (mouth of Delaware Bay); 2.9 ft. at Reedy Point; and 4.0 ft. at Philadelphia (Reference 2.4.5-13).

Traveling northwest, Isabel made landfall as a Category 1 hurricane near Beaufort, NC. The storm center was closest to the new plant location at 163 NM to the southwest. At this point, it was a tropical storm (Reference 2.4.5-12). Isabel produced a storm surge of 3.1 ft. at Lewes, DE; 5.0 ft. at Reedy Point; and 5.4 ft. at Philadelphia (Reference 2.4.5-14).

Hurricane Hazel and the Chesapeake-Potomac hurricane produced the maximum historical storm surges recorded in Delaware Bay. Of these, the Chesapeake-Potomac hurricane storm center passed closer to the new plant location, exhibiting a northwesterly track most similar to the hypothetical storm track of the PMH (References 2.4.5-18 and 2.4.5-17). Based on the storm track and adequate available data related to this storm, the Chesapeake-Potomac hurricane of August 1933 is selected to validate the storm surge model used to determine the PMH surge.

2.4.5.2.2 Estimation of Probable Maximum Storm Surge

In order to satisfy the combined events criteria specified in Section 9.2.2 of ANSI/ANS-2.8-1992, (Reference 2.4.5-1) storm surge at the new plant is evaluated combining probable maximum surge and seiche with wind wave activity concurrent with the 10 percent exceedance high tide, and effects of hurricane-associated precipitation. This subsection outlines the sequence of steps

virtually negligible (less than 3 percent) upstream of the head of Delaware Bay (upstream of RM 48), and reduces surge on the east side of the estuary at the new plant location (Reference 2.4.5-3), therefore neglecting cross-wind effects is conservative at the new plant location. The wind setup algorithm of Kamphuis is a steady-state analytical solution of the fundamental equations governing hydrodynamics, which can be found in reference texts (References 2.4.5-27 and 2.4.5-4). Its primary assumption, that water levels exhibit a steady state response to varying winds, is conservative because the bay does not respond to the winds instantaneously. The combination of these methods is demonstrated to be valid by reproducing the storm surge of an actual historical hurricane as described in the remaining paragraphs of this subsection.

These methods are validated by reproducing the surge observed during the Chesapeake-Potomac hurricane of 1933. The pressure distribution and winds associated with this storm are specified as described by Bretschneider (Reference 2.4.5-3) and NOAA (Reference 2.4.5-18). Bretschneider reports a pressure drop of 0.85 in. of Hg. This value is used with NOAA (Reference 2.4.5-18) formulas for the Standard Project Hurricane to determine the pressure distribution and wind field throughout the storm. Bretschneider reports maximum sustained winds over the ocean of 58 mph (50 kt), and maximum sustained winds over Delaware Bay of 50 mph (43 kt). The simulated storm exhibits maximum winds of 64 mph (56 kt) over the ocean, and 47 mph (41 kt) over Delaware Bay, similar to the wind speeds reported for the Chesapeake-Potomac hurricane.

7.7

Coincident astronomical tides are specified at the mouth of Delaware Bay. Comparison of model results with the actual response to the Chesapeake-Potomac hurricane is expressed as storm surge, the difference between actual water levels and the predicted astronomical tide level. The storm surge calculated at the mouth of Delaware Bay, using the Bodine method, reproduces the observed surge as described by Bretschneider (Reference 2.4.5-3). Comparison of observed and simulated surge at the mouth of the bay is illustrated in Figure 2.4.5-3. The peak storm surge at Reedy Point, DE, is calculated to be 7.9 ft., while the observed surge at Reedy Point was 7.6 ft. Water surface elevations (surge plus tide) at Reedy Point are illustrated in Figure 2.4.5-4. The Delaware Bay storm surge model described here is demonstrated to be conservative. The margin of error is consistent with comparable models, such as NOAA's SLOSH model which has a stated margin of error of +/- 20 percent (Reference 2.4.5-23).

2.4.5.2.2.1 Estimation of 10 Percent Exceedance High Tide

Maximum monthly high tide values from 1987 through 2008 are analyzed at NOAA tidal gage stations upstream and downstream from the new plant location to determine the 10 percent exceedance high tide at the site (Reference 2.4.5-16). This analysis calculates a 10 percent exceedance high tide of 4.2 ft. NAVD at the Lewes, DE, NOAA tidal gage (8557380) at river mile (RM) 0, and 4.6 ft. NAVD for the Reedy Point, DE, NOAA tidal gage (8551910) at RM 59 as illustrated in Figure 2.4.5-5. Based on these values, the 10 percent exceedance high tide at the new plant location at RM 52 is determined by linear interpolation to be 4.5 ft. NAVD.

This approach for estimating 10 percent exceedance high tide, based on recorded tides, intrinsically includes the effects of sea level anomaly (also known as initial rise). ANSI/ANS-2.8-1992, Section 7.3.1.1.2, concludes sea level anomaly need not be included when 10-percent exceedance high tide is based on recorded tides. Sea level anomaly is not included in this analysis because recorded tide data is used to calculate the 10-percent exceedance high tide.

This table is to be added at the end of SSAR Subsection 2.4.5.

Table 2.4.5-2
Maximum Sustained Wind Speed (kt) for Multiple PMH Scenarios

		Radius of Maximum Winds, R (NM)		
		11	**20**	**28**
Forward Speed, T (kt)	**42**	135	133	132
	34	133	131	130
	26	131	129	128

Note: Each PMH evaluated in the above table exhibited a central pressure, p_0, = 26.65 inches of mercury; pressure drop, Δp = 3.5 inches of mercury; and track direction from 138 degrees (moving northwest). These parameters, and the ranges considered, represent the PMH that can affect the project site according to NOAA (Reference 2.4.5-18).

This table is to be added at the end of SSAR Subsection 2.4.5.

Table 2.4.5-3
Maximum Surge (ft. NAVD) for Multiple PMH Scenarios from Screening Simulations

		Radius of Maximum Winds, R (NM)			
		11	20	28	28
		At Mouth of Delaware Bay (RM 0)			At the Site (RM 52)
	42	18.5	21.7	22.7	23.4
Forward Speed, T (kt)	34	18.4	21.2	22.1	25.3
	26	18.1	21.2	22.1	27.8

Note: Each PMH evaluated in the above table exhibited a central pressure, p_0, = 26.65 inches of mercury; pressure drop, Δp = 3.5 inches of mercury; and track direction from 138 degrees (moving northwest). The tide is specified as static at the 10% exceedance high tide at the mouth of the Delaware Bay. Consequently these results cannot be compared with results presented in table 2.4.5-1 where a dynamic tide input is specified.

PSEG Letter ND-2011-0066, dated December 9, 2011

ENCLOSURE 3

CD-ROM containing Input Files


PSEG Site
Early Site Permit Application
RAI No. 39
Enclosure 3
PSEG
Power LLC
Sargent & Lundy LLC

PSEG Letter ND-2011-0066, dated December 9, 2011

ENCLOSURE 4

Summary of Regulatory Commitments

ENCLOSURE 4

SUMMARY OF REGULATORY COMMITMENTS

The following table identifies commitments made in this document. (Any other actions discussed in the submittal represent intended or planned actions. They are described to the NRC for the NRC's information and are not regulatory commitments.)

COMMITMENT	COMMITTED DATE	COMMITMENT TYPE	
		ONE-TIME ACTION (YES/NO)	PROGRAMMATIC (YES/NO)
PSEG will revise SSAR Subsection 2.4.5 to incorporate the changes described in the responses to RAI 39, Questions 02.04.05-1, 02.04.05-2, and 02.04.05-5 and in Enclosure 2.	This revision will be included in the next update of the PSEG Site ESP Application SSAR.	Yes	No