Figure 2.2-1
Location of the PSEG Site within the Delaware river Basin

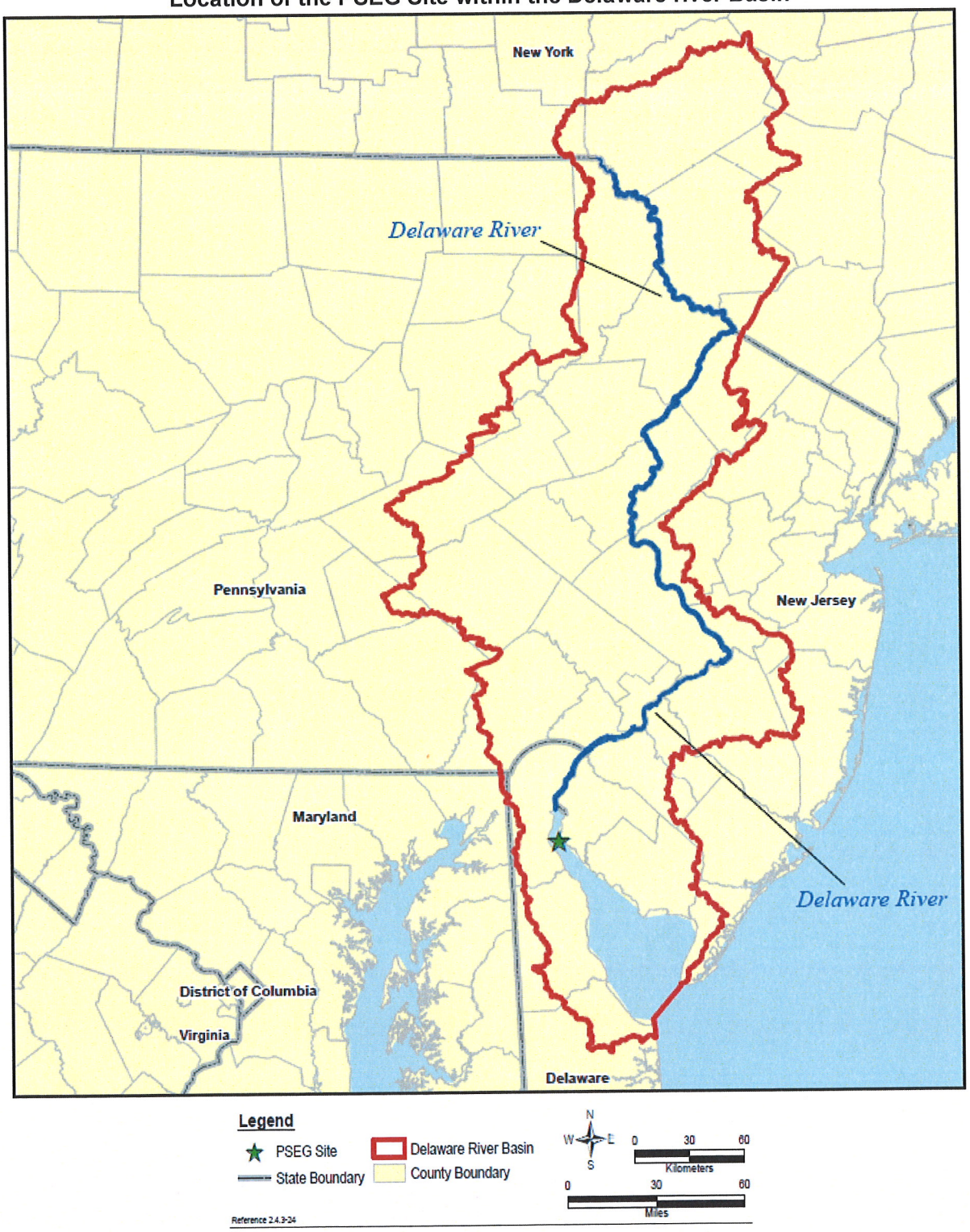


Figure 2.2-2
Storm surge time series results from Hurricane Hazel at the PSEG Site

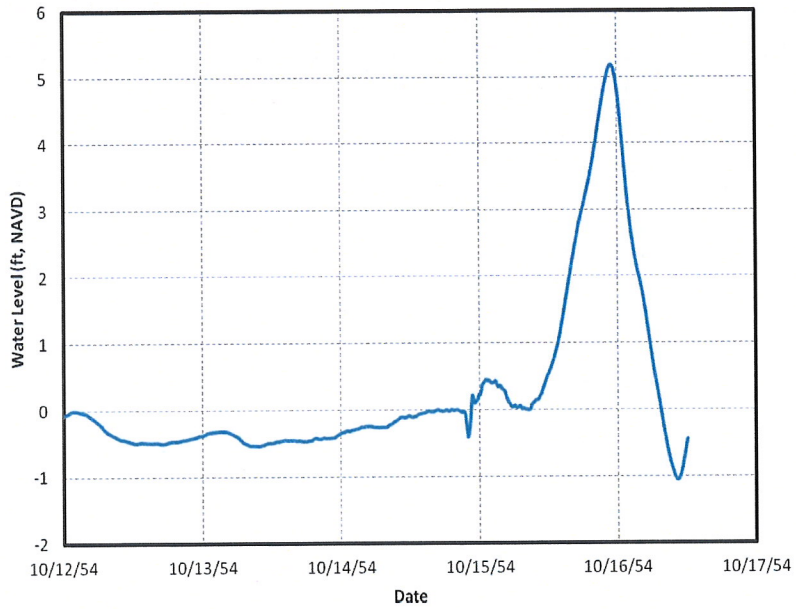
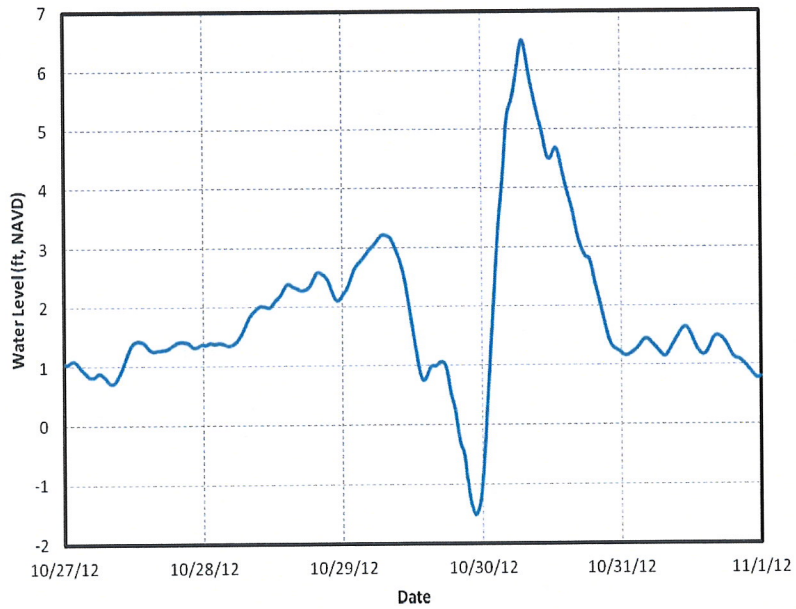


Figure 2.2-3
Storm surge time series results from Hurricane Sandy at the PSEG Site



2.3 DAM BREACHES AND FAILURES

Pursuant to the United States Nuclear Regulatory Commission (USNRC) Interim Staff Guidance (ISG) document JLD-ISG-2013-01,*Guidance for Assessment of Flooding Hazards Due to Dam Failure*, simplified modeling approaches are used in this section to determine the critical and non-critical dams in the watershed. In accordance with the ISG, non-critical dams are those that do not have the potential to adversely affect the safety-related systems, structures or components (SSCs) at a site. The findings of the simplified modeling approach are applicable to all modes of failure (Hydrologic, Seismic, and Sunny Day). (Reference 2.3-4)

The following stepwise procedure is used to determine the effect of dam failure on the PSEG Site in accordance with JLD-ISG-2013-01:

- Compile the Upstream Dam Database
- Assess the peak outflow water surface elevation without considering attenuation
- Assess the peak outflow water surface elevation with consideration of attenuation

There are no dams on the Delaware River. No safety-related water control or storage structures (e.g., reservoirs) are located on the PSEG Site. Therefore, only upstream dams on tributaries to the Delaware River are evaluated to determine the effects of flooding due to dam failures at the PSEG Site.

2.3.1 Compilation of the Upstream Dam Database

The National Inventory of Dams (NID) managed by the United States Army Corps of Engineers (USACE) is utilized to characterize the dams located upstream of the PSEG Site (Reference 2.3-2). The NID is queried by county, and the pertinent information for all of the dams in the counties located within the watershed is tabulated. The dam name, latitude, longitude, county, state, height, storage, primary purpose, and other attributes are obtained for each reservoir. The dams are then mapped using ArcGIS by latitude and longitude. The United States Geological Survey (USGS) 8 digit Hydrologic Unit Code (HUC) basin features are used to determine the extent of the upstream watershed, and are utilized as the basis of the spatial selection of the dam features.

A total of 1,024 dams are identified as being located upstream of the PSEG Site based on the information available in the NID (Reference 2.3-2). Of the identified dams, twelve do not have either height or storage information. In order to complete the dataset, the missing information is referenced from a nearby dam of similar dimensions. The maximum reported dam height is 280 ft. for the Merrill Creek Main Dam located in Warren County, New Jersey, and the reported maximum dam storage is 609,740 acre-feet for the Downsville Dam (Pepacton Reservoir) located in Delaware County, New York. The average dam height and storage volume is 22 ft. and 3,500 acre-feet, respectively (Reference 2.3-2). The locations of the upstream dams are depicted in Figure 2.3-1.

2.3.2 Peak Outflow without Attenuation

The volume method, Method 1 of ISG Section 3.2 (Reference 2.3-4), is not applicable to the site, since the site is located within a coastal floodplain, and the surrounding extent of the storage capacity for the Hope Creek and Salem Sites is not defined. Pursuant to the ISG

Section 3.2, Method 2, peak outflow without attenuation method, the peak discharge for each dam was determined utilizing the United States Bureau of Reclamation (USBR) regression equation (Reference 2.3-3).

The USBR regression equation describes the relationship between peak discharge and dam height as:

$$Q_p = 19.1(h_w)^{1.85}$$ Equation 2.3-1

where:

$Q_p =$ Peak breach outflow, (m^3/s)

$h_w =$ Height of water above breach invert at time of failure, (m)

For the purpose of this analysis, it is assumed the height of water would be equal to the dam height. Based on the USBR regression, it is estimated that a failure of all of the dams in the upstream portion of the watershed would result in a total breach discharge of approximately 4.18×10^7 cubic feet per second (cfs).

The potential effect of the estimated breach discharge on the PSEG Site was determined utilizing a stage discharge curve for river mile 50.8. The stage discharge curve was developed from the output of the Cannonsville-Pepacton breach scenario that was completed for the PSEG Site ESPA (Reference 2.3-1). The selected unsteady flow simulation has a static tide elevation of 0 ft. NAVD. Therefore, tidal fluctuations do not affect the stage-discharge relationship. An evaluation of the stage-discharge relationship for the Cannonsville-Pepacton breach scenario indicates that the most conservative stage-discharge relationship is observed when the flow at river mile 50.8 is greater than 600,000 cfs. Therefore, flows greater than 600,000 cfs are utilized for the development of the stage-discharge relationship. The linear regression of the stage discharge relationship indicates that the stage at river mile 50.8 is equal to 1.38×10^{-6} times discharge minus 0.3488 ($y = 1.38E\text{-}6 * x - 0.3488$). The linear regression exhibits a coefficient of determination (R^2) of 0.99. The stage-discharge hydrograph for the Cannonsville-Pepacton breach scenario at river mile 50.8 is presented in Figure 2.3-2, and the linear regression of the stage-discharge relationship is presented in Figure 2.3-3. It should be noted that most stage-discharge relationships are described with a logarithmic regression. However, a linear regression was utilized in this instance to incorporate a degree of conservatism, since a linear regression conservatively assumes that no conveyance occurs in the floodplain.

The stage-discharge relationship is used to estimate the potential water surface elevation if the sum of the peak discharges for the breach of all upstream dams were to be observed at the PSEG Site. Reference 2.3-4 specifies using the 500-year flood elevation as the initial stage for the analysis. A water surface elevation of 6.5 ft. NAVD that was estimated for the 500-year flood combined with the 10 percent exceedance high tide simulation that was completed for the PSEG Site ESPA (Reference 2.3-1) is considered representative of the base hydrologic conditions coincident with the upstream dam failures; it is estimated that 2.5 ft. of inundation could occur above the base hydrologic conditions prescribed in the ISG Section 3.2 Method 2 without exceeding site grade, which is conservatively taken as 9 ft. NAVD. Based on the stage-discharge relationship at river mile 50.8, it is estimated that the sum of the peak discharges from

a breach of all of the upstream dams (4.18 x 10^7 cfs) could potentially create an inundation above base hydrologic conditions of 57.4 ft. (63.9 ft. NAVD).

The utilization of a stage-discharge relationship developed with a maximum stage of approximately 0.8 ft. that is used to extrapolate inundation levels up to approximately 60 ft. is an extremely conservative approach, since the stage-discharge relationship with a maximum stage within the normal tidal range does not incorporate the significant increase in available cross-sectional area for flow once the water surface elevation overtops the river channel bank and floodwaters enter the floodplain. Similarly, adding the 10 percent exceedance high tide to the estimated level of inundation rather than developing a stage-discharge relationship where the initial water surface elevation is equal to the 10 percent exceedance high tide is conservative since the conveyance capacity of the floodplain is ignored. Therefore, a more refined analysis, as described in the ISG is developed in the below subsection.

2.3.3 Peak Outflow with Attenuation

Since the simplified Method 2 of ISG Section 3.2 indicates potential for flooding to exceed site grade, screening pursuant to Method 3 of ISG Section 3.2 is completed. Method 3 recommends utilizing the peak discharges developed in Method 2 and attenuating the discharges based on a regression developed using storage volume, downstream distance, and attenuation of peak flow (Reference 2.3-4). The four breach scenarios evaluated for the PSEG Site ESPA (Reference 2.3-1) are used to develop a watershed-specific attenuation regression (Reference 2.3-3). The ratio of maximum combined discharge at river mile 50.8 (Qx) to the average peak discharge for the failure scenario (Qp) is used to estimate the ratio of peak discharge to attenuated discharge (Qx/Qp) to develop the attenuation relationship for the failure of dams within the watershed. The ratio of peak discharge to attenuated discharge is plotted by average storage area for the failure scenario, and minimum river mile distance upstream for the failure scenario. The use of minimum river mile distance and average storage area is a conservative approach in developing the regression for the attenuation of the peak discharge, since it assumes that the average reservoir volume for the breach scenario produces the maximum discharge of the combined failure; effectively underestimating the attenuation. This conservative approach is utilized because the maximum discharge for the combined failure cannot easily be apportioned to each reservoir without using a series of assumptions. The reservoir data, results of the combined failure evaluation, and the Qx/Qp ratios at river mile 50.8 for each failure scenario evaluated for the PSEG Site ESPA are presented in Table 2.3-1, and the attenuation curves are depicted in Figure 2.3-4.

The attenuation rate for each attenuation curve is estimated using an exponential regression. The estimated attenuation rates are then plotted against storage volume, and two linear regressions are utilized to describe the relationship between storage volume of a reservoir and the expected attenuation rate of the peak discharge. The method of regression is implemented to characterize the rapid increase in attenuation rate for dams with storage volumes less than approximately 100,000 ac-ft. Therefore, the intersection point of the two regressions at 102,979 ac-ft is utilized to differentiate the appropriate regression for each dam; where the attenuation rate of dams with storage volumes greater than 102,979 ac-ft is calculated as y = 6.796E-9 * x - 0.01529, and the attenuation rate of dams with storage volumes less than 102,979 ac-ft is calculated as y = 1.816E-6 * x - 0.2016, with x equal to storage volume in ac-ft. The regression of the attenuation rates is presented in Figure 2.3-5. The nearest river mile to each dam was estimated by using the point distance feature in ArcGIS. The point distance feature was used to

determine the nearest river mile referenced to the nearest dam cross section locations that were developed for the PSEG Site ESPA (Reference 2.3-1). The point distance feature also calculates the radial distance between the dam feature and the cross section feature, but this distance is ignored as a manner of conservatism.

The distance of each dam from the PSEG Site (River mile of dam – River mile of PSEG Site) based on nearest river mile is multiplied by the attenuation rate that is estimated based on the storage volume of the dam, and the product is then used as the power of the exponential function to estimate the Qx/Qp ratio for the dam. The Qx/Qp ratio is then multiplied by the peak discharge calculated for Method 2 to estimate the attenuated discharge for each dam. A summation of the attenuated discharges indicates that the total peak discharge of 4.18×10^7 cfs would attenuate to 9.08×10^5 cfs at river mile 50.8 based on the conveyance characteristics of the watershed. The uncertainty associated with the approximation of the dam height and storage for the twelve uncharacterized dams discussed previously is insignificant since doubling the estimated values for the twelve dams did not affect the attenuated discharge to three significant figures.

Based on the stage-discharge relationship that was developed in Subsection 2.3.2, it is estimated that a discharge of 9.08×10^5 cfs would result in an inundation level above base hydrologic conditions of approximately 0.9 ft.; where the inundation is calculated using the linear regression $y = 1.38E\text{-}6 * x - 0.3488$, with an x of 9.08×10^5 cfs. An inundation level above base hydrologic conditions of 0.9 corresponds to a water surface elevation of 7.4 ft. NAVD on top of 6.5 ft. NAVD for 10 percent exceedance high tide and the 500-year flood. This indicates that the estimated attenuated discharge from the breach of all upstream dams would produce a still water surface elevation that is approximately 1.6 ft. below site grade, which is conservatively taken as 9 ft. NAVD.

2.3.4 Conclusions

Based on the screening procedure described above, all upstream dams and reservoirs in the watershed can be classified as non-critical, in accordance with Method 3 of ISG Section 3.2. The breach of combinations of reservoirs in the watershed is unlikely to inundate the PSEG Site. Since all dams and reservoirs in the watershed are classified as non-critical, further analysis of Hydrologic, Seismic, or Sunny Day dam failure is not warranted per Section 3.2 of the ISG (Reference 2.3-4). Therefore, dam failure is not a viable flood causing mechanism at the PSEG Site.

2.3.5 References

2.3-1 PSEG Power, LLC and PSEG Nuclear, LLC. PSEG Site Early Site Permit Application, Part 2, Rev.2, Subsection 2.4.4, 2013.

2.3-2 U.S. Army Corps of Engineers, “National Inventory of Dams,” Website, https://nid.usace.army.mil/, accessed October 4, 2013.

2.3-3 U.S. Bureau of Reclamation, 1982, Guidelines for Defining Inundation Areas Downstream from Bureau of Reclamation Dams, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado.

2.3-4 U.S. Nuclear Regulatory Commission, “Guidance for Assessment of Flooding Hazards Due to Dam Failure, JLD-ISG-2013-01” Interim Staff Guidance, Revision 0, July 29, 2013.

Table 2.3-1
PSEG Site ESPA SSAR Dam Failure Scenario Summary Data and Qx/Qp Ratios at RM 50.8

Name of Dam or Reservoir	Failure Scenario	Storage* (ac-ft)	River Mile Location of Dam	Maximum Breach Discharge (cfs)	Average Storage for Combined Failure Scenario (acre-feet)	Average Breach Discharge Qp (cfs)	Minimum Distance Upstream of RM 50.8 (miles)	Maximum Combined Discharge at RM 50.8 Qx (cfs)	Peak Discharge Attenuation Ratio (Qx/Qp)
Pepacton Reservoir	1	460,000	331	7,590,000	381,500	7,060,000	280.2	198,785	0.0282
Cannonsville Reservoir		303,000	331	6,530,000					
LakeWallenpaupack	2	209,000	278	1,080,000	175,500	1,435,000	203.2	81,193	0.0566
Neversink Reservoir		142,000	254	1,790,000					
Francis E. Walter Reservoir	3	111,000	184	2,210,000	93,833	1,261,667	106.2	46,102	0.0365
Beltzville Reservoir		104,000	184	1,120,000					
Nockamixon Reservoir		66,500	157	455,000					
Blue Marsh Reservoir	4	50,000	92	1,070,000	23,475	362,150	20.2	14,450	0.0399
Marsh Creek Reservoir		22,200	71	214,000					
Springton Reservoir (Geist Dam)		10,700	85	113,000					
Edgar Hoopes Reservoir		11,000	71	51,600					

Figure 2.3-1
Locations of Upstream Dams

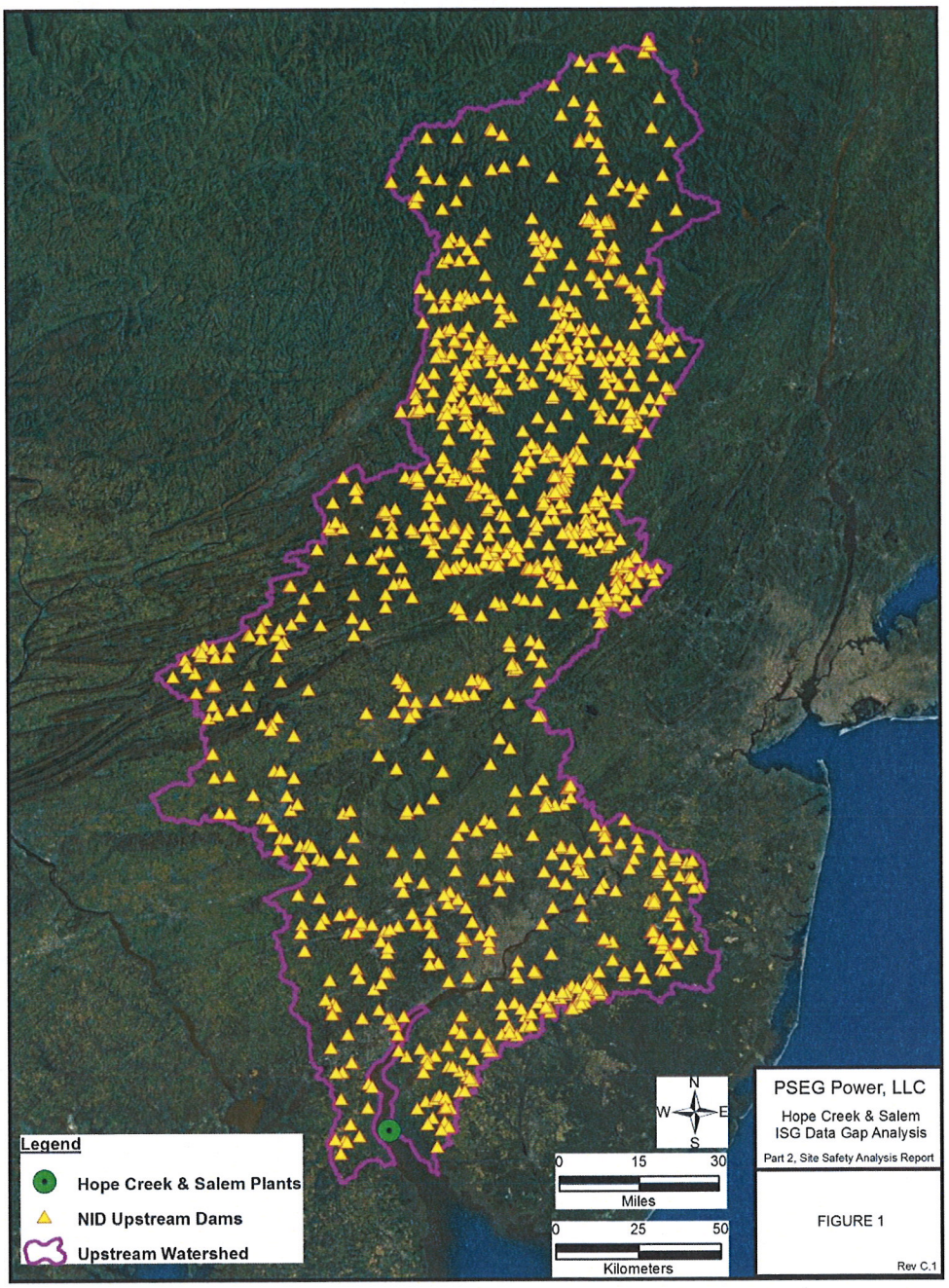


Figure 2.3-2
Stage-Discharge Hydrograph for Cannonsville-Pepacton Breach Scenario

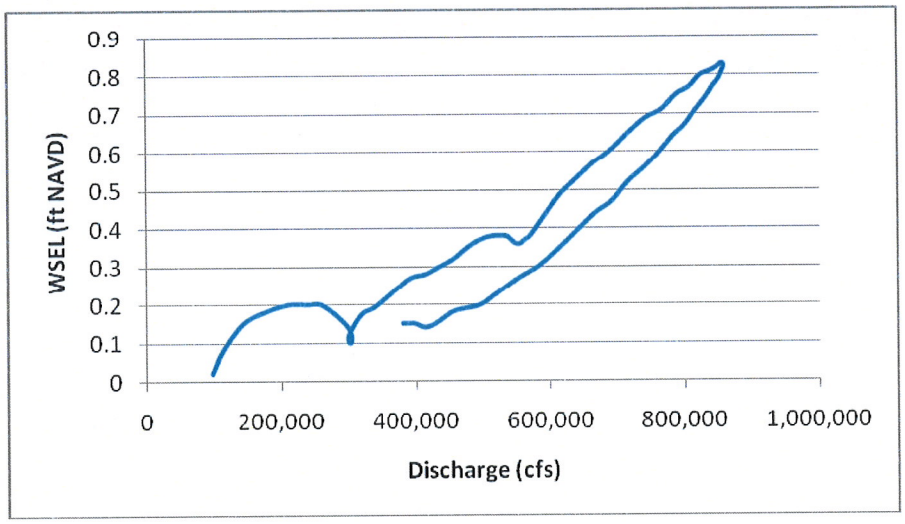


Figure 2.3-3
Linear Regression for Cannonsville-Pepacton Breach Scenario

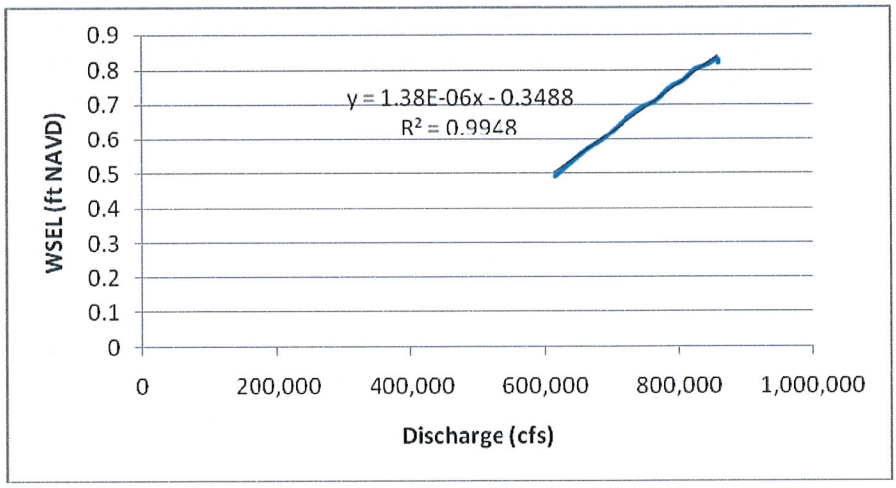


Figure 2.3-4
Attenuation Curves Based On Storage Volume

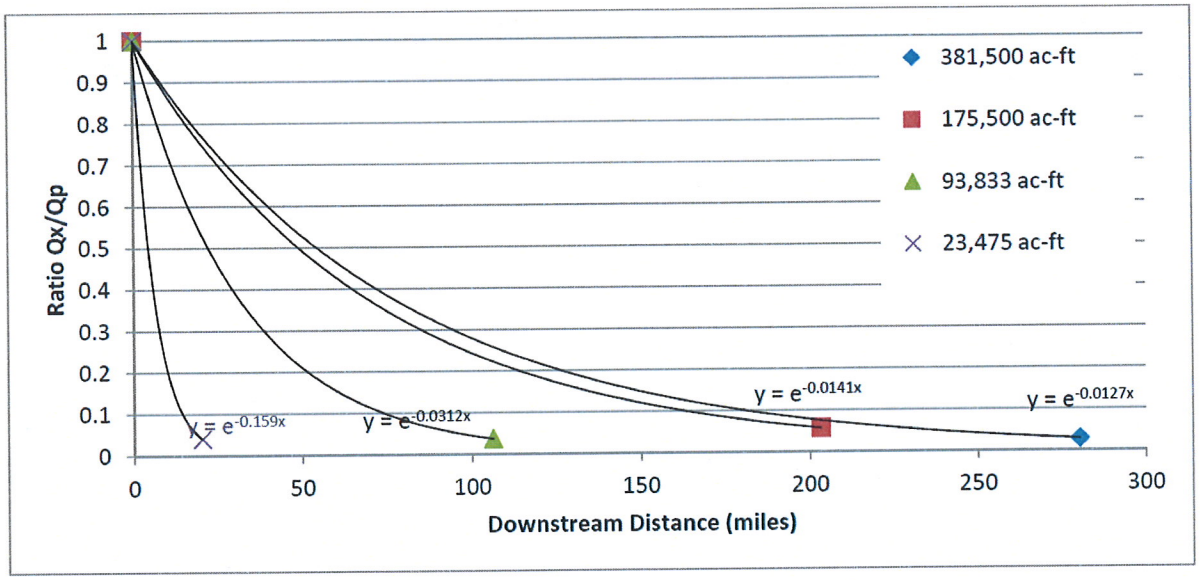
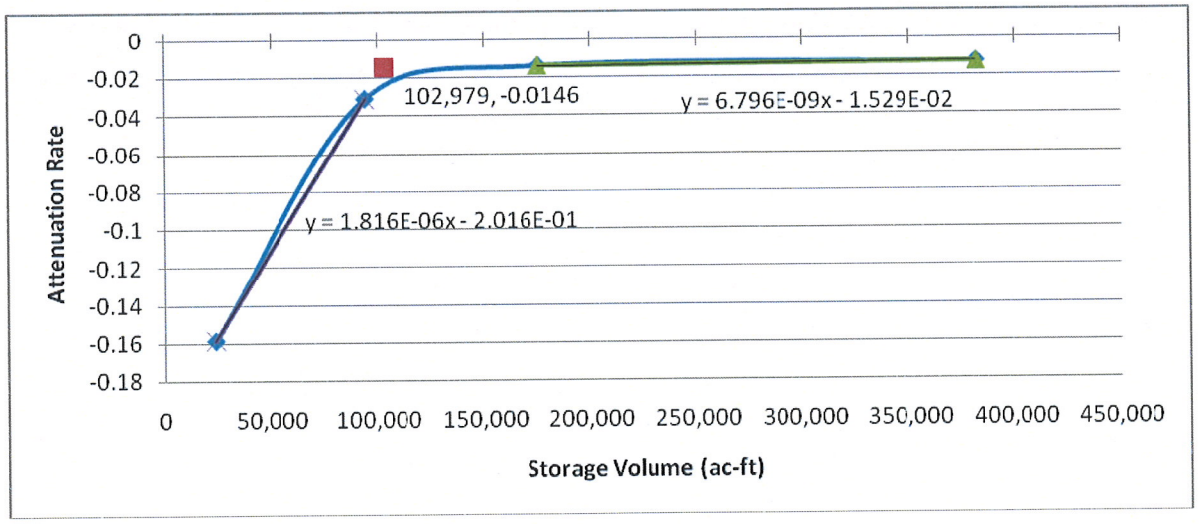


Figure 2.3-5
Storage Volume Attenuation Rate Relationship



2.4 STORM SURGE

In this subsection, the hydrometeorological basis is developed to define the potential flooding hazard to safety-related structures, systems, and components (SSC) at Salem Generating Station, due to the effects of storm surge. SGS is located on the eastern shore of the Delaware River estuary. The existing topography at the PSEG Site ranges from 5 to 15 ft. NAVD (Reference 2.4-17). Consequently, the SGS may be affected by hurricane induced storm surge. The analysis presented in this subsection is essentially identical to the analysis performed for the PSEG Site ESPA in response to Request for Additional Information (RAI) Number 67 (Reference 2.4-22). The primary difference is in the approach to determining wave run-up at critical structures around SGS.

The methodologies used to determine the effects of storm surge are in accordance with Regulatory Guide 1.59, as supplemented by current regulatory guidance, including NUREG/CR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, NUREG/CR-7134, The Estimation of Very-Low Probability Hurricane Storm Surges for Design and Licensing of Nuclear Power Plants in Coastal Areas, and JLD-ISG-2012-06, Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment. (References 2.4-33, 2.4-34, 2.4-35 and 2.4-36)

Subsections 2.4.1 through 2.4.3 present the methodology used to determine the storm surge at the PSEG Site. The analysis combines a state-of-the-art high resolution numerical modeling platform with the Joint Probability Method – Optimal Sampling (JPM-OS) method developed by U.S. Army Corps of Engineers (USACE) to evaluate storm surge.

Subsection 2.4.4 addresses the potential future effects of sea level rise. Subsection 2.4.5 adds the potential sea level rise value to the 10^{-6} annual exceedance probability (AEP) total water surface elevation (WSEL) from the storm surge, to determine the future 10^{-6} AEP WSEL through the remaining life of the plant.

Subsection 2.4.6 addresses sediment erosion and deposition associated with the storm surge. Subsection 2.4.7 demonstrates that the winds associated with the storm surge bound those associated with a severe wind storm.

2.4.1 Estimation of Storm Surge

The PSEG Site is located adjacent to the Delaware River estuary and is potentially susceptible to inundation from storm surge. Storm surge can result from several different types of storms (e.g., tropical cyclones, extra-tropical cyclones and squall lines). The storm surge estimation for the PSEG Site considers tropical cyclones (i.e., hurricanes) and extra-tropical cyclones (e.g., nor'easters). Winds from squall lines are discussed in Subsection 2.4.7.

The methodology selected to develop the storm surge analysis for the PSEG Site follows the current state of practice for Federal Emergency Management Agency (FEMA) and USACE for storm surge inundation analyses (References 2.4-6 and 2.4-23). In recent years, it has been recognized that of the available methods, Joint Probability Method (JPM) pioneered by Myers (References 2.4-9 and 2.4-18) for coastal flood estimation is preferred for the tropical storm environment. The JPM approach has the conceptual advantage of considering all possible

storms consistent with the local climatology, each weighted by its appropriate rate of occurrence. In brief, the most basic JPM approach adopts a parametric storm description involving key hurricane parameters such as central pressure, radius to maximum winds (R_{max}), and forward speed. By analyzing the local climatology, probability distributions (not necessarily mutually independent) are developed for each of the several parameters. These distributions are each discretized into a small number of representative values, and all possible parameter combinations are simulated using a hydrodynamic model constructed to accurately represent the bathymetry, topography, and ground cover of the study site. (Reference 2.4-6)

Efforts by USACE and FEMA independently developed new and highly efficient methods of implementing the JPM approach in such a way as to minimize the number of storms requiring simulation. It was found that the simulation effort could be reduced by about an order of magnitude while still maintaining good accuracy. The two approaches are known as Joint Probability Method - *Optimal Sampling* (JPM-OS), quadrature method and response surface method. The procedure followed for the PSEG Site aligns with the USACE's approach for Louisiana, called the *Response Surface Method*. This 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. (Reference 2.4-6)

The PSEG Site is located adjacent to the Delaware Estuary, which from a flooding perspective, is covered by both FEMA Region II and Region III. FEMA Region II and Region III initiated coastal analysis and mapping studies to produce updated Digital Flood Insurance Rate Maps for coastal counties within their respective regions (Reference 2.4-7 and 2.4-8). The ADvanced CIRCulation (ADCIRC) model mesh used in the Region III study and the statistical storm surge analyses developed in Region II are used in the analysis developed for the PSEG Site.

Given the recommended storm surge analysis methodologies of FEMA and USACE, and the methodologies presented in Reference 2.4-35, PSEG established the following storm surge estimation procedure for developing the 10^{-6} AEP storm surge elevation at the PSEG 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 run-up analysis to determine the total water surface elevation 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 run-up) 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 run-up.

References 2.4-35 and 2.4-36 provide guidance on other combined effects for consideration in determining the effects of storm surge. As described in subsequent subsections, analysis of the storm surge at the PSEG Site considers the effects of waves and wave run-up, tides, and river discharge.

2.4.2 Modeling System

The modeling system for the determination of storm surge at the PSEG Site uses a suite of state-of-the-art numerical wind, wave, and surge models and methods to compute surge still and total WSELs at the points of interest. The model suite consists of the TC96 Planetary Boundary Layer (PBL) wind model for tropical storms, the wave-field model Simulating Waves Nearshore (SWAN), and the storm surge and tidal model ADCIRC. This wind, wave and surge modeling approach is very similar to the recent FEMA-sponsored Region III floodplain-mapping project (Reference 2.4-32). In addition to the numerical models, estimation of wave run-up at the points of interest to establish a peak total WSEL is determined using approaches described by USACE (Reference 2.4-29). The input to the modeling system is a series of parameters that represent the synthetic storm (i.e., storm track, which consists of time, position, central pressure, Holland B parameter [which controls the shape of the pressure and wind fields], radius to maximum winds, and peripheral pressure). The output from the modeling system is the peak still WSEL and total WSEL for the PSEG Site associated with each individual storm modeled.

2.4.2.1 Wind Model

The TC96 Planetary Boundary Layer (PBL) model is used to develop wind and pressure fields for the synthetic storms (Reference 2.4-25). For each storm, defined by a track and time-varying wind field parameters, the TC96 PBL model is applied to construct wind and atmospheric pressure fields every 60 minutes for driving surge and wave models. TC96 generates wind and pressure fields with a highly refined meso-scale moving vortex formulation developed originally by Chow (Reference 2.4-4) and modified by Cardone et al. (Reference 2.4-3). The model is based on the equation of horizontal motion, vertically averaged through the depth of the planetary boundary layer.

2.4.2.2 ADCIRC+SWAN Model

Storm surge simulations are performed using the tightly coupled ADCIRC+SWAN state-of-the-art coastal circulation and wave model. ADCIRC is based on the two-dimensional, vertically-integrated shallow water equations that are solved in Generalized Wave Continuity Equation form (Reference 2.4-28). The equations are solved over complicated bathymetry encompassed by irregular seashore boundaries using an unstructured finite-element method. This algorithm allows for flexible spatial discretizations over the entire computational domain. The advantage of this flexibility in developing a computational mesh is that larger elements can be used in open-ocean regions where coarser resolution is needed, whereas smaller elements can be applied in the nearshore and estuary areas where finer resolution is required to resolve hydrodynamic details and more accurately simulate storm surge propagation onto a complex coastal landscape. (Reference 2.4-35)

The recent FEMA Region III storm surge study developed a high-resolution ADCIRC mesh that covers the entire Delaware Bay and PSEG Site region (see Figure 2.4-1). The ADCIRC mesh is comprised of a high-resolution grid covering FEMA Region III that was appended to a previously developed grid of the western North Atlantic, the Gulf of Mexico and the Caribbean Sea. Specifically, the grid covers the area from the 60 degrees west meridian to the US mainland. Within FEMA Region III, the grid extends inland to the 49.2 ft. NAVD (15 m) contour to allow for inland storm surge flooding. In this region, 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 National Oceanic and Atmospheric Administration (NOAA) charts. (References 2.4-30 and 2.4-31)

After confirming the FEMA developed ADCIRC mesh was operating correctly on the project computing platform, the mesh is refined in the vicinity of the PSEG Site to more accurately represent the topographic features of the site. To properly describe the topographic features important to the hydrodynamic and wave characteristics at the PSEG 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 refined mesh for the PSEG Site area is shown on Figure 2.4-2. The refined PSEG Site mesh is inserted into the overall FEMA Region III mesh and the model is re-validated using the same Hurricane Isabel and Nor'easter Ida test storm input files as the FEMA Model validation report prepared by USACE (Reference 2.4-32). A graphical comparison of water levels from the Hurricane Isabel storm simulation on the refined PSEG Site mesh and unmodified FEMA Region III at locations around the PSEG Site is shown in Figure 2.4-3. This process confirms that the refined mesh produces results that are essentially the same as the unmodified FEMA Region III mesh in the vicinity of the PSEG Site.

2.4.2.3 Nested SWAN Model

To develop a detailed analysis of wave conditions on the PSEG Site, surge and wave parameters from the larger-scale ADCIRC+SWAN model are used to drive a high-resolution 'nested' SWAN model covering the upland portion of the site and the immediate offshore area. The high resolution (approximately 3 m node spacing) allowed for the inclusion of fine details on the site such as narrow seawalls and concrete barriers. Figure 2.4-12 provides a comparison of the nested SWAN grid resolution to the ADCIRC+SWAN mesh resolution. The result is a SWAN grid that produces highly-detailed wave characteristics across the entire PSEG Site. The boundary between the ADCIRC+SWAN grid and the nested SWAN grid is shown on Figure 2.4-13.

The nested SWAN model is validated by comparing significant wave height results in the main ADCRIC+SWAN grid to significant wave height results in the nested SWAN grid at four locations south of the PSEG Site during simulation of Storm 11, as shown in Figure 2.4-13.The significant wave height values are nearly identical with differences that are less than those presented by Catini et al. (Reference 2.4-39) and Muraleedharam et al. (Reference 2.4-40) for similar SWAN nesting studies. Allowing for slight changes from the much finer grid, it is clear from the results (see Figure 2.4-13) between the grids are consistent and that the nesting functions properly.

2.4.2.4 Wave Run-up Estimation Methodology

The ADCIRC+SWAN and nested SWAN simulations of each storm produce still WSEL and wave data (significant wave height, period and direction) over the course of each storm surge event. The nested SWAN wave field data at critical points around the plant (see Figure 2.4-4) are provided at 15-minute intervals. The data is analyzed and captured for the subsequent wave run-up calculations. The wave run-up calculations described below are performed 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 run-up, at each point is captured as the maximum value for that storm event at that point. Those values are subsequently used to establish the 10^{-6} AEP total WSEL for each point, as described in Subsection 2.4.3.7.

Wave run-up calculations for the plant are based upon the latest design guidance found in the USACE Coastal Engineering Manual (CEM), Chapter VI-5 (Reference 2.4-29). The waves are considered to impact vertical surfaces on the plant's critical surfaces; therefore, the Goda equations are used to determine the wave run-up height (Table VI-5-53, Equation VI-5-147 of Reference 2.4-29).

The CEM prescribes the use of 1.8 times the significant wave height for use as the design wave with the Goda equations. Typically for nuclear design, one alteration from the methodology presented in the CEM is the use of the lesser of (a) the maximum wave height, or (b) the "breaker height" (0.78 times depth of water) for computation of wave run-up as required by References 2.4-1 and 2.4-36. Per Reference 2.4-1, the maximum wave height, H_{max}, is defined as the 1 percent wave, $H_{1\%}$. CEM Equation II-1-132 defines $H_{1\%}$ as 1.67 times H_s (Reference 2.4-29). However, since the Goda equations (Table VI-5-53 of Reference 2.4-29) specifically define a design wave height of 1.8 times the significant wave height, this value is used as defined in the CEM equations since it is considered to be the intent of the author and more conservative than the typical 1 percent wave value required in References 2.4-1 and 2.4-36.

The 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 (H_s) and wave period (T) are acquired from the grid node closest to the location of interest at 15 minute increments.

- The significant wave height (H_s) is checked for both steepness and depth-limited conditions in accordance with Reference 2.4-41, and, if necessary, trimmed using the below equation:

  $$H_{s(applied)} = \text{minimum}\ [0.1*L*\tanh((2\pi/L)h),\ H_s] \qquad \text{(Equation 2.4-1)}$$

  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 (Table VI-5-53 of Reference 2.4-29).

  $$H_{design} = 1.8*H_{s(applied)} \qquad \text{(Equation 2.4-2)}$$

- The wave runup elevation is established using Equation VI-5-147 of Reference 2.4-29, as presented below:

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

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

The above procedure is executed for each timestep at each point of interest at the plant. The maximum value is then captured for each storm event analyzed and a corresponding total WSEL established, as further discussed in Subsection 2.4.3.7.

2.4.3 Joint Probability – Optimal Sampling

The JPM approach is a simulation methodology that relies on the development of statistical distributions of key hurricane input variables (e.g., central pressure, radius to maximum winds, forward speed, and storm heading) and sampling from these distributions to develop model hurricanes. The simulation results in a family of modeled storms that still preserve the relationships between the various input model components, yet provide a means to model the effects and probabilities of storms that have not yet occurred. For this study, the JPM-OS *Response Surface Method* is used which reduces the number of required simulated storms.

The following subsections describe the JPM-OS process followed in determining the 10^{-6}AEP surge still WSEL and total WSEL for the PSEG Site.

2.4.3.1 Develop Synthetic Suite of Storms

Two different storm populations generate potentially significant storm surges in coastal areas along the northeastern United States: storms of extra-tropical origin and storms of tropical origin. These two populations are considered mutually exclusive and are evaluated separately to determine their impact on the overall annual exceedance probability of 10^{-6} to define the probabilistic storm surge flood hazard level.

2.4.3.1.1 Contribution from Extra-Tropical Storms

The location of the PSEG Site within the context of the Delaware Bay is shown on Figure 2.4-5. The general geographic trend of the Delaware Bay runs approximately from northwest to southeast, starting at the PSEG Site moving toward the mouth of the Bay. This trend makes it difficult for storms with centers located off the coast in the Atlantic Ocean to generate winds which are moving up the Delaware Bay toward the PSEG Site, due to the counterclockwise rotation of such storms. Extra-tropical storms with the lowest central pressures and associated high wind speeds occur in oceanic areas rather than over land areas (Reference 2.4-27). Furthermore, the strongest of these storms are imbedded in the generally westerly flows in the mid-latitudes (which moves them in a general southwest to northeast direction) or tend to become somewhat stationary; consequently, storms of an extra-tropical origin rarely move in a direction that would produce persistent winds along the axis of the Delaware Bay toward the

PSEG Site (Reference 2.4-27). A byproduct of this characteristic of intense extra-tropical storms is that they contribute predominantly to surges in the recurrence interval range of 1 to 50 years in the Delaware Bay. For surges associated with events in the 100- to 500-year return-period range, the FEMA Region II flood analysis determined tropical systems dominate the surge estimates at these return periods. Therefore, given the 10^{-6} AEP range of the storm surge estimates for the PSEG Site, storms of extra-tropical origin are not considered in the analysis discussed below.

2.4.3.1.2 Contribution from Tropical Storms

Past studies have shown that the maximum surge generated by tropical storms and hurricanes can be estimated as a function of a set of several storm parameters (References 2.4-11, 2.4-12, 2.4-35). This function is provided below:

$$\eta_{max} = \eta_{max} (\Delta p, R_{max}, v_f, \theta_f, B, x_o)+\varepsilon \qquad \text{(Equation 2.4-4)}$$

where:

η_{max}	estimated maximum surge
Δp	pressure differential (peripheral pressure minus central pressure)
R_{max}	distance from the eye of the storm to maximum winds
v_f	forward velocity of the storm
θ_f	angle of storm heading
B	Holland B parameter
x_o	along coast location of landfall
ε	deviation in storm surge due to potential errors in the estimate

Typically, these past studies have focused on areas where tidal impact was minimal and river flows could be neglected. The Delaware River and Bay in the PSEG Site region is significantly influenced by the effects of tides, therefore tidal effects must be considered. Additionally, the Delaware River transitions into the Delaware Bay at river mile 48, in the vicinity of the PSEG Site; therefore, the effects of tides and river discharge must also be considered. Thus, the storm surge function considered for the PSEG Site is defined as follows:

$$\eta_{max} = \eta_{max} (\Delta p, R_{max}, v_f, Q_f, B, x_o, D, \delta\eta_{tide})+\varepsilon \qquad \text{(Equation 2.4-5)}$$

where:

η_{max}	estimated maximum surge
Δp	peripheral pressure minus central pressure
R_{max}	distance from the eye of the storm to maximum winds
v_f	forward velocity of the storm
θ_f	angle of storm heading
B	Holland B parameter
x_o	along coast location of landfall
D	river discharge
$\delta\eta_{tide}$	water level deviations due to tidal effects
ε	deviation in storm surge due to potential errors in the estimate

The last term (ε) in Equation 2.4-5 represents the sum of a wide range of omissions and potential errors in the storm surge estimate and is referred to as the epistemic uncertainty.