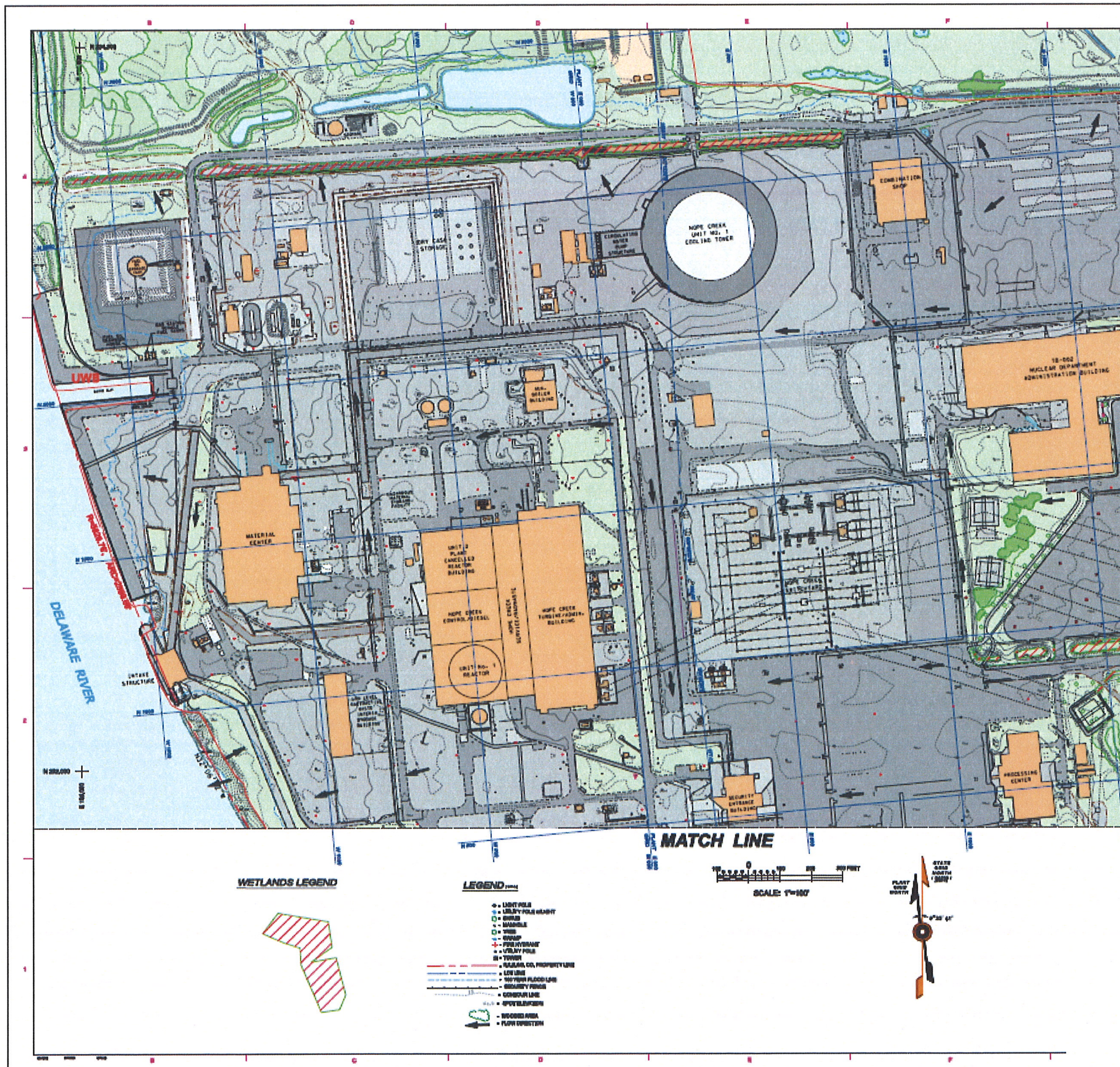
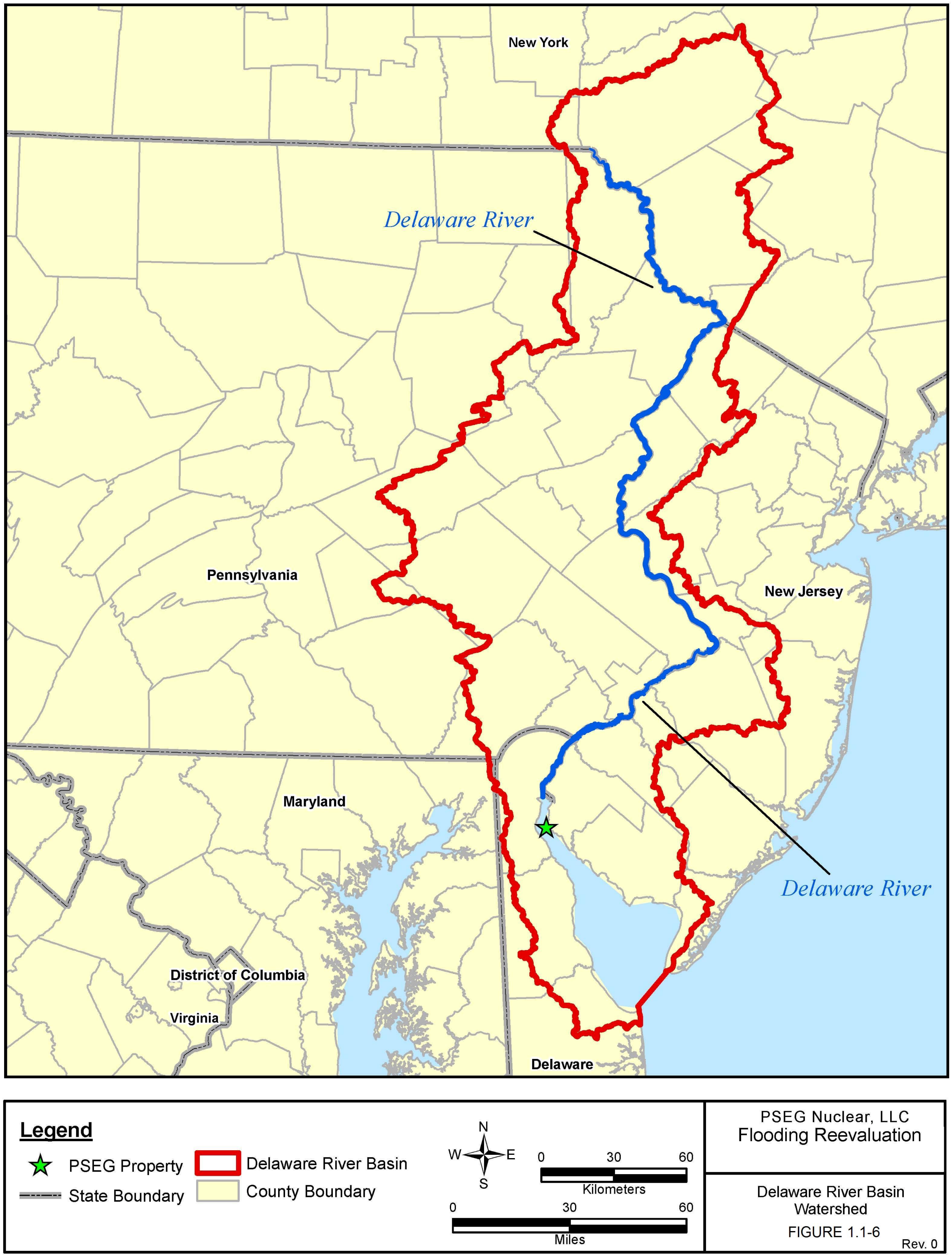


PSEG Nuclear, LLC
Flooding Reevaluation

HCGS Topography
FIGURE 1.1-5

Rev. 0

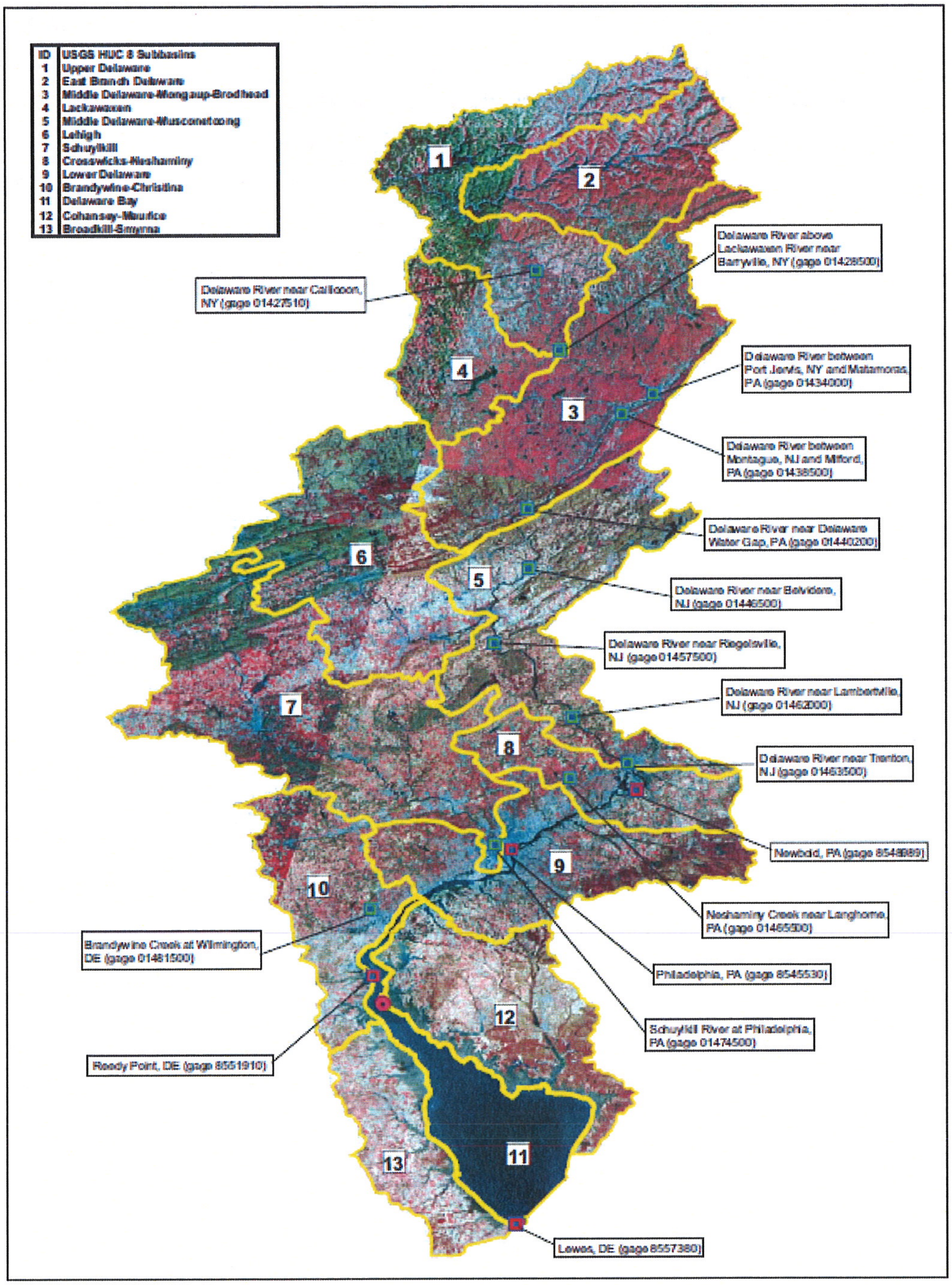


PSEG Nuclear, LLC
Flooding Reevaluation

Delaware River Basin Watershed

FIGURE 1.1-6

Rev. 0



LEGEND

- USGS Gage Stations
- NOAA Tidal Gage Stations
- Subbasins
- PSEG Property

N, W, E, S

0 20 40 Kilometers

0 20 40 Miles

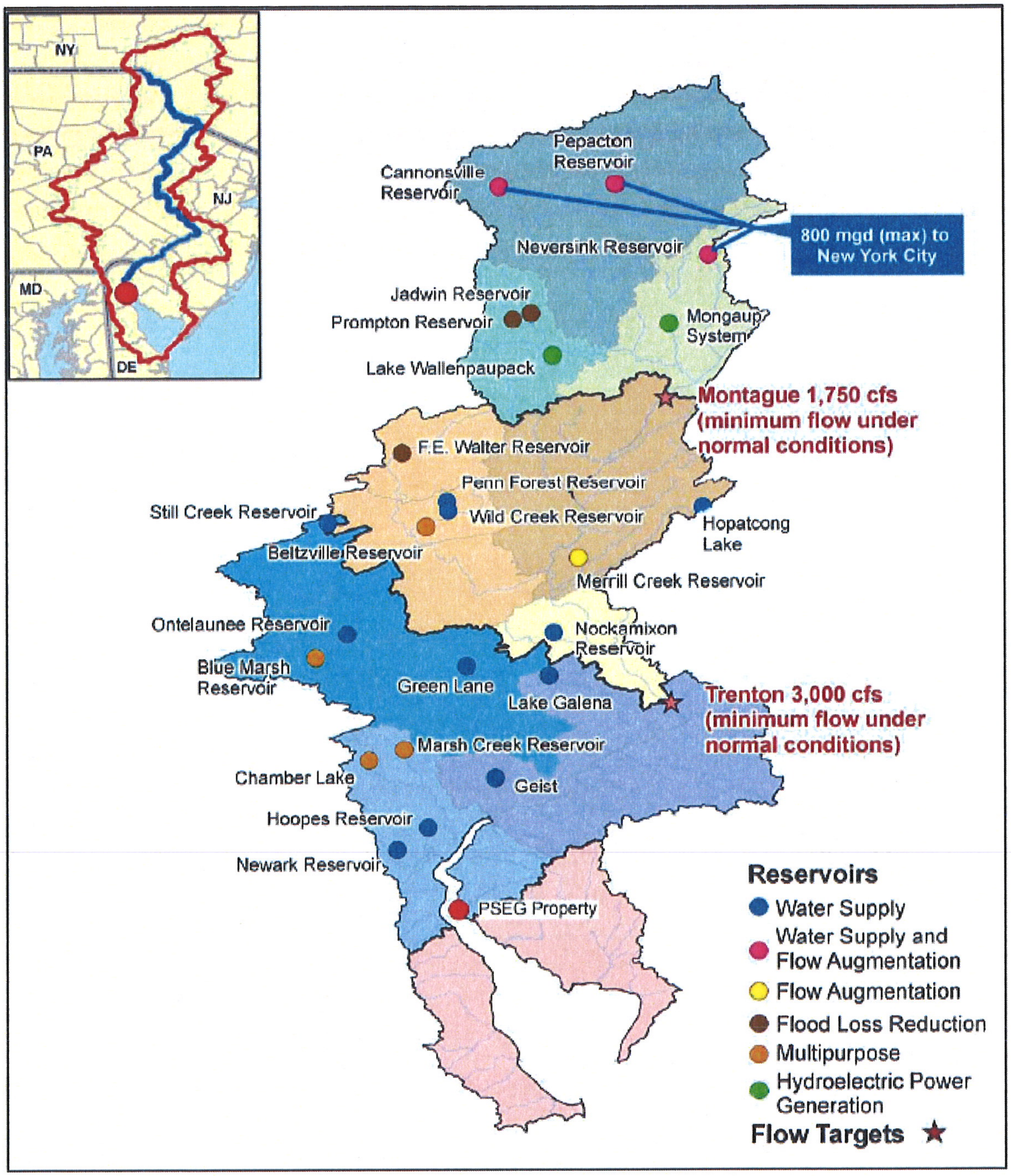
PSEG Nuclear, LLC
Flooding Reevaluation

Delaware River Watersheds and Stations

FIGURE 1.1-7

Rev 0

Sargent & Lundy LLC



PSEG Nuclear, LLC
Flooding Reevaluation

Reservoirs of the Delaware River Basin
FIGURE 1.1-8
Rev.0

NAVD	PSEG	
11.7	101.5	HCGS Nominal Site Grade
9.7	99.5	SGS Nominal Site Grade
7.7	97.5	Historical High Water (November 1950)[a]
4.5	94.3	10% Exceedance High Tide[a]
2.9	92.7	Mean Higher High Water (Reedy Point, DE)[a]
0	**89.8**	**North American Vertical Datum of 1988**
-0.8	89.0	National Geodetic Vertical Datum of 1929 (0 ft. msl)
-3.0	86.8	Mean Lower Low Water (Reedy Point, DE)[a]
-8.8	81.0	Historical Low Water (December 31, 1962)[a]

Not to Scale

a) Values developed as part of the PSEG Site Early Site Permit Application.

PSEG Nuclear, LLC
Flooding Reevaluation

Datum and Water Level Relationship
FIGURE 1.1-9

1.2 CURRENT DESIGN BASIS FLOOD ELEVATIONS

The design basis flood is the result of the probable maximum hurricane (PMH) surge with wave runup coincident with the 10% exceedance high tide. The design basis flooding event is applicable to all Operational Conditions (e.g., Power Operation, Startup, Hot Shutdown, Cold Shutdown, Refueling, and conditions when recently irradiated fuel is being handled in the secondary containment).

The HCGS UFSAR (Reference 1-6) characterizes tide in the Delaware Estuary as having a mean range at the estuary mouth at about 4.3 ft. and generally increasing through the estuary to about 6.7 ft. at Trenton. The Reedy Point Station is the tide gauge station nearest the site. The CLB describes the tides there as having a mean tide range of 5.5 ft., spring tide range of 6.0 ft., MSL of 2.8 ft. above mean low water (MLW) and a 10 percent exceedance high tide of 6.6 ft. The HCGS UFSAR also characterizes the tides at HCGS as having a mean tide range of 5.8 ft. and elevation 0 ft. NGVD (MSL), equivalent to 89 ft. PSD or 2.6 ft. above MLW (86.4 ft. PSD).

Table 1.2-1 lists the CLB flooding mechanisms at HCGS (see also HCGS UFSAR Table 2.4-6). The maximum design still-water height is as a result of the PMH surge and is 113.8 ft. PSD. The maximum run-up elevation on the site is 124.4 ft. PSD (HCGS UFSAR Table 2.4-10) for the power block structures and 134.4 ft. PSD (HCGS UFSAR Table 2.4-10a) for the intake structure.

1.2.1 CLB Local Intense Precipitation

The design rainfall rate for the yard drainage system is 4 inches per hour for a period of 20 minutes, based on 90 percent runoff from paved areas and 50 percent runoff from graded areas. However, the storm drainage system is evaluated for the probable maximum precipitation (PMP) provided in Table 1.2-2. The drainage system accommodates most rainfall intensities. The PMP was as found in *Probable Maximum Precipitation Estimates, United States East of the 105th Meridian*, Hydrometeorological Report No. 51, June 1978 (Reference 1-8).

The effect of local intense precipitation of the magnitude of a PMP concentrated on the plant site, assuming complete blockage of the underground storm drainage system, was evaluated and concluded that the site would flood during an occurrence of the PMP. However, because all openings into safety-related structures are protected from flooding to a minimum elevation of 32 ft. MSL or 19.5 ft. above plant grade by watertight doors and hatches, it was determined that site drainage does not pose a flooding problem.

As part of the Safety Evaluation Report (SER) for the HCGS FSAR, the NRC staff made an independent analysis using the PMP values cited in Table 1.2-2 and determined that the ponding level on plant grade during the PMP event would reach a maximum level of 12.9 ft MSL. This level would occur during the most intense 5 min of the event and is 0.1 ft below the door sill elevation of 13.0 ft MSL. Therefore, the NRC staff concluded that the plant met the requirements of General Design Criteria 2 with respect to the effects of local intense precipitation runoff on site drainage relative to flooding of safety-related structures.

1.2.2 CLB Flooding in Streams and Rivers

The CLB probable maximum flood (PMF) event for HCGS was considered over the entire Delaware River Basin. HCGS utilized analysis results from a previous study for the Summit Generating Station Preliminary Safety Analysis Review (PSAR) to estimate PMF discharge on the Delaware at the Chesapeake and Delaware Canal (8.3 miles above HCGS). Given the results from the previous study, the PMF was determined to be a relatively minor flooding event in comparison to other postulated events evaluated. Therefore, there was justification for adopting a simplified but conservative estimating approach for the PMF levels at the HCGS. With a PMF discharge at the HCGS of 1,250,000 cfs the PMF stillwater level elevation at HCGS was calculated to be 8.3 ft. MSL (97.3 ft. PSD).

The coincident wind wave activity superimposed on the maximum stillwater level elevation at the plant site provided the conditions to evaluate the maximum wind wave effects on the plant structures. The method used was the shallow water wave generation with limited fetch length technique recommended in the Shore Protection Manual, 1977 (Reference 1-3). Results gave the significant wave heights and significant wave periods at the end of the fetch directions, i.e., Artificial Island. The maximum wave height estimated for the most critical fetch direction was 9.9 ft. and the wave period of 5.0 seconds. The maximum wave runup height estimated was 109.8 ft. PSD.

1.2.3 CLB Dam Breaches and Failures

The HCGS dam breach flood hazard analysis was performed using guidance provided in Appendix A of Regulatory Guide 1.59 and ANSI, ANS-2.8-1981 (Reference 1-1) with respect to the selection of seismic failure models for the dams and coincident flow.

Single dam failures for the proposed Tock’s Island Dam and proposed modified Francis E. Walter Dam were postulated to determine the most critical single dam failure condition. The postulated failure of the dam was equated to the instantaneous disappearance of the dam. Analysis provided that a failure at Tock’s Island Dam resulted in a stillwater level of 9.0 ft. MSL (98 ft. PSD) and coincident wind/wave activity produces a wave height of 6.0 ft. and wave runup height of 21.6 ft. MSL (110.6 ft. PSD).

Based on the most critical single dam failure determined above, postulated multiple failures of Cannonsville, Pepacton, and Tocks Island Dams were also analyzed. The calculations of the dam break discharges and the routing of the flood wave followed the procedure outlined for single dam failures with modifications. The analysis resulted in a stillwater level elevation of 13.5 ft. MSL (102.5 ft. PSD) at the HCGS location. The maximum wave runup elevation was estimated to be 26.3 ft. MSL (115.3 ft. PSD).

1.2.4 CLB Storm Surge

As discussed above, the most critical combination of flood producing phenomena results from the postulated occurrence of the probable maximum hurricane (PMH) surge with wave runup coincident with the 10 percent exceedance high tide. The CLB contained in the HCGS UFSAR for the effects of a probable maximum hurricane (PMH) storm surge at the site was based on previous analyses performed for the Hope Creek PSAR (1968) and Salem FSAR (1972).

The PMH as defined in U.S. Weather Bureau Memorandum HUR 7-97, *Interim Report, Meteorological Characteristics of the Probable Maximum Hurricane, Atlantic and Gulf Coast of the United States*, 1968 (Reference 1-12) was used for the CLB. A hurricane of large radius and moderate forward speed produced the critical high water conditions on the open coast and in Delaware Bay. The critical path of the PMH was shoreward in a direction generally normal to the bottom contours of the continental shelf. The hurricane center makes a landfall approximately 39 nautical miles to the south of the Delaware Bay mouth.

The stillwater level was defined as the water level at the HCGS location as a result of PMH surge, which was the surge level at the site plus the cross-wind setup. It is also the level to which the rise in water levels due to wave action was referenced. This stillwater elevation was postulated to result from the PMH open coast surge at the mouth of Delaware Bay routed up the bay to the plant site. The CLB analysis used the bathystrophic storm tide theory as described by Marinos and Woodward in Reference 1-5, to estimate surge elevations at the mouth of Delaware Bay. The computed maximum surge elevation at the mouth of Delaware Bay was 21.9 ft. MLW and accounted for the 10 percent exceedance high tide of 6.6 ft.

The surge hydrograph at the mouth of Delaware Bay was routed to the site using the procedures developed by Bretschneider, as discussed in Reference 1-2. The process involved routing of the open coast surge through the entrance to Delaware Bay, allowing for convergence as the bay narrows, modifying of the surge due to friction, and additional wind stress on the surface of the surge. The routing of the surge produced a peak surge level of 24.5 ft. MLW. In reviewing the flood design considerations for the adjacent SGS, the Atomic Energy Commission (AEC) suggested that the computed surge levels be increased by 2.9 ft. The peak storm surge stillwater level after adjustment was 27.4 ft. MLW.

For the purpose of maximizing the effects of hurricane surge and coincident wave activity at the site, the PMH was postulated to have a track along the west side of and generally parallel to Delaware Bay and River. Wind speed and direction at the site changed as the PMH moves along this path because of the effects of friction and filling overland and also because of the position of the hurricane center with respect to the site. The wind speed along each fetch was based on the position of the storm center relative to the site and the time elapsed since landfall was provided from the procedures described in HUR 7-97, found in Reference 1-12. The maximum wind-generated wave activity coincident with surge induced stillwater levels had a maximum offshore wave height of 16.4 ft. with a period of 6.6 seconds. The corresponding significant wave height was 10.9 ft.

The average water depth used for this analysis was the sum of the water depth at the center of the fetch, including the surge level above mean low water and the increase of computed surge level of 2.9 ft. suggested by the AEC. The estimation of significant wave conditions in shallow water was determined using the procedures described in Reference 1-3. The ratio of maximum wave height to significant wave height was conservatively chosen to be 1.5 for design purposes. The effect of viscous damping was also considered for incident waves towards the shoreline for only waves approaching from fetches that would be subjected to interference by the land mass near the headland of the Artificial Island. Methods described in Reference 1-4 were used for the analysis.

The attenuated maximum wave heights were then used for wave run-up and wave loading analyses for the west-facing vertical wall of the Service Water Intake Structure. The effects of a range of wave periods associated with the attenuated maximum wave height for each fetch direction was investigated. The resulting maximum run-up elevation on the Service Water Intake Structure was 45.4 ft. MSL (134.4 ft. PSD). Non-breaking wave conditions were assumed for wave run-up and loading analyses on the vertical wall.

For impact on the remaining plant structures, the analysis accounted for wave transformation as the incident waves encountered the earth dikes and fill areas in the vicinity of the plant. The dikes extend to Elevation 106.5 ft. PSD. Large waves break before they reach the plant buildings. The maximum wave height incident on safety-related plant facilities was determined to be depth limited for all fetches except two. The breaking or maximum wave height for the depth limited fetches was equated to 0.78 of the water depth. The adjacent SGS effectively prevents waves from directly reaching the HCGS safety-related buildings along the remaining two fetches. The maximum wave heights were determined to be 1.31 and 1.29 times as large as the corresponding significant waves for these two fetches. A range of wave periods and corresponding wave lengths associated with the maximum wave height was examined. The wave run-up height was estimated by the Sainflou method (Reference 1-3). Results indicated the controlling wave runup height to Elevation 35.4 ft. MSL (124.4 ft. PSD) along the southeast face of the Reactor Building and a small corner face of the Auxiliary Building.

1.2.5 CLB Seiche

The possible forces and expected periods that could cause seiche or resonance flooding in the Delaware Estuary were listed in the HCGS UFSAR and are discussed below:

- The periods of wind generated waves in the Delaware Estuary could range between one and seven seconds. Since these periods are very much shorter than the fundamental period of free oscillation for the Delaware Estuary, no wave resonance would occur.
- The astronomical tide has a period on the order of 12 hours, which is approximately one half to one third of the maximum oscillation period of the Delaware Estuary. Thus, the astronomical tide would not provide the forcing mechanism to generate resonance.

Based on the analyses made, it was concluded that seiche is not a problem at the HCGS. Large amplitude oscillations are not possible, because the most probable forcing mechanisms identified lack either a period of oscillation close enough to the fundamental period of the Delaware Estuary to be of concern, or a magnitude and duration great enough to supply a significant amount of energy into the basin. In addition, energy dissipation of any water level oscillation occurs by frictional damping and reflection along the banks of the estuary.

1.2.6 CLB Tsunami

Records of Atlantic tsunamis are relatively rare. At the time of the HCGS analysis a total of about 30 large tsunamis within the recorded history had occurred in the Atlantic Ocean. The HCGS probable maximum Tsunami (PMT) was developed using input previously provided in NUREG/CR-1106 (Reference 1-7). The input was used for the computation of the resulting wave history anywhere within the ocean basin. The procedure was repeated for a number of potential source locations, chosen according to degree and type of seismic activity. Hypothetical coastal histories of great tsunamis emanating from any potential source area were simulated. The model predicted tsunami wave heights at offshore stations where the water depths are approximately 600 ft. The wave height predictions at these offshore stations included both the incident and reflected wave components. The wave characteristics at the site were the result of the transformation of the waves by their interaction with nearshore features as they propagate shoreward.

The peak tsunami wave heights and time histories were estimated for two stations bracketing the Delaware Bay entrance. These stations are located offshore of Atlantic City, New Jersey and offshore of Assateague Island, Maryland. The peak wave heights and arrival times were 1.5 ft. at 9511 seconds, and 2.8 ft. at 9370 seconds, respectively. A hypothetical distant earthquake located near Haiti produced the maximum wave displacement at the offshore stations.

The maximum water level at HCGS resulting from tsunami waves considers the effect of coincident wind-wave activity, as discussed in the PMF analysis. The coincident wave height with a 2-year extreme wind associated with the tsunami wave height and 10 percent exceedance high tide was 9.6 ft. above the maximum stillwater level, which is at 6.0 feet MSL. The resulting wave runup height on the SWIS was estimated at 18.1 ft. MSL (107.1 ft. PSD).

1.2.7 CLB Ice Induced Flooding

It was concluded that effects of an ice jam flood at HCGS were negligible because even though there is usually sufficient ice in the Delaware Bay and River to be of some concern to navigation, tidal currents keep the ice in motion and ice breakers keep packs of ice in the narrower parts open. Additionally, HCGS is credited with a deicing system that protects the service water intake against clogging by ice.

1.2.8 CLB Channel Migration or Diversion

It was concluded that there is no evidence of channel diversions of significance in the Delaware River Basin. Since the HCGS is located in a tidally affected portion of the basin, sources of cooling water are located both upstream and downstream of the site. In the highly unlikely event that either the river flow or the tidal flow is temporarily interrupted by a channel diversion event, the other source would continue to supply water to the site area.

1.2.9 CLB Combined Effects

Combined effects of different flood causing mechanisms is discussed in Subsection 1.2.1 through 1.2.8, where applicable.

1.2.10 CLB Associated Effects

Reference 1-14 defines “Flood height and associated effects” as:

> *“The maximum stillwater surface elevation plus the following factors:*
> - *wind waves and run-up effects;*
> - *hydrodynamic loading, including debris;*
> - *effects caused by sediment deposition and erosion;*
> - *concurrent site conditions, including adverse weather conditions;*
> - *groundwater ingress; and*
> - *other pertinent factors.”*

Inclusion of wind waves and run-up effects is discussed in Subsection 1.2.9; discussion of the remaining items, as addressed in the CLB, is provided below.

1.2.10.1 Hydrostatic and Hydrodynamic Loads

The current licensing basis for HCGS (Reference 1-6) includes evaluation of the effects of hydrostatic and hydrodynamic loads on all safety related structures. The loading combinations evaluated in the current design basis are presented in Table 1.2-3. Per HCGS UFSAR Section 3.4.1.1, "All exterior doors in Seismic Category I structures are designed to withstand the static and dynamic effects from postulated floods and the associated wave action." Therefore, it is considered that external watertight doors can withstand the same wave loading combinations the respective walls experience.

1.2.10.2 Debris Loads

The current licensing basis for HCGS (Reference 1-6) references probabilistic evaluation of floating missiles impacting plant operation. A.D. Little (Reference 1-15) prepared an analysis of the likelihood of waterborne traffic and other floating objects on the Delaware River impacting HCGS in severe storms. Reference 1-15 includes the following primary objectives:

- Developed a profile of floating objects and assessed their impacts on the overall integrity of the plant.
- Profiled the marine traffic on the Delaware River and estimate the likely probability of impacting HCGS.

With assistance from USACE and input from the Bechtel Design Guidelines for tornado missiles, Reference 1-15 developed a floating debris spectrum (see Table 1.2-4) of potentially damaging missiles during floods and severe storms. At that time, regulatory guidance for environmentally damaging missiles had not been established by the NRC or any other agency. The floating objects were considered to be moving at 20 mph when impacting plant structures, which were considered to be a highly conservative value. This analysis is used in the HCGS design basis to qualify the doors as capable of such impacts.

Reference 1-15 also develops a conservative analysis of the probability of marine vessel traffic in the Delaware River impacting the powerblock or intake structures. Based on a conservative review of marine traffic, the potential for a runaway vessel, and the probability of the initiating event (PMH storm occurring), the report concludes the probability of the service water intake structure probability being impacted by any vessel under the postulated storm conditions is 6.1×10^{-8}. The probability of the HCGS powerblock being impacted by any vessel under the postulated storm conditions is 4.7×10^{-8}.

The HCGS Safety Evaluation Report (Reference 1-16) describes the NRC staff's assessment of floating missiles in Section 3.4.1. This evaluation concludes that the potential for riverborne missiles being transported up the Delaware Bay during the PMH event, onto the Hope Creek site and impacting safety-related structures is extremely small. Although the staff's analysis indicates that the potential for waterborne missiles being transported onto the Hope Creek site and endangering the powerblock safety-related structures is acceptably low, the service water intake structure, because of its location on the bank of the Delaware River, is exposed to riverborne missiles. In considering riverborne missiles resulting from severe storms, the staff evaluated the effects of a PMH, a model hurricane and extreme wind events. The resulting probability of riverborne missiles resulting from severe storms damaging the intake structure is 8×10^{-8} impacts per year, which is considered acceptably small.

1.2.10.3 Erosion and Sedimentation

Effects of erosion and sedimentation during extreme flooding events are not analyzed in the HCGS CLB.

1.2.10.4 Concurrent Site Conditions

The current licensing basis for HCGS (Reference 1-6) does not specifically discuss evaluation of concurrent site conditions, such as storm conditions during the event. However, as discussed in Section 1.5, the flood protection features for HCGS do not require operator action outside of flood protected structures concurrent with the severe storm event. All outside actions (e.g., traveling to the service water intake structure and closing watertight doors) take place prior to the onset of the inundation portion of the flood event.

1.2.10.5 Groundwater Ingress

The current licensing basis for HCGS (Reference 1-6) indicates the maximum expected water table elevation is 96 ft. PSD. Below grade structures at HCGS are designed to mitigate the effects of the continuous presence of groundwater through the use of flood protection features including penetration seals, waterproofing and waterstops. The HCGS flood protection features are rated to elevations greater than the design basis flood, further discussed in Reference 1-13.

1.2.10.6 Other Pertinent Factors

The other pertinent factor for flood causing mechanisms at HCGS is the flood event duration. Flood event duration is defined in Reference 1-6 as the length of time the flood event affects the site, beginning with conditions being met for entry into a flood procedure or notification of and impending flood (e.g., a flood forecast or notification of dam failure), including preparation for the flood and the period of inundation, and ending when water has receded from the site and the plant has reached a safe and stable state that can be maintained indefinitely. The flood protection features at HCGS are designed as permanent features and therefore, are not affected by the period of inundation and recession of flood waters from the site. The preparation for a severe weather event is covered both administratively via procedure and through Technical Specification Action Statements at HCGS. Further discussion of the action levels associated with implementing flood protection features is provided in Subsection 3.10.6.

Table 1.2-1
CLB Flooding Mechanisms

Flooding Mechanism	Affected Structure	Still Water Height (ft. PSD)	Max Flood Height (ft. PSD)	HCGS UFSAR Section
Local Intense Precipitation	Not Analyzed - Design rainfall rates for site drainage system provided.			2.4.2.3
Flooding in Streams and Rivers	Site	97.3	109.8	2.4.3
Dam Breaches and Failures	Site			2.4.4
Proposed Tock's Island dam with one half of local probable maximum flood (PMF)		98	110.6	
Modified Francis E. Walter dam with one half of local PMF		94	105.9	
Multiple dam break-Modified Cannonsville and Pepacton with potential domino-type hydrologic failure of proposed Tock's Island		102.5	115.3	
Probable Maximum Hurricane	Powerblock	113.8	120.0/124.4	2.4.5
	Service Water Intake Structure	113.8	134.4	
Seiche	No Flooding Expected			2.4.5
Probable Maximum Tsunami	Service Water Intake Structure(a)	95	107.1	2.4.6
Ice Induced Flooding	No Flooding Expected			2.4.7
Channel Migration or Diversion	No Flooding Expected			2.4.9

a) The effect of the maximum waves height on safety-related facilities above the plant grade was determined to be insignificant. The maximum wave run-up elevation on safety-related facilities below the plant grade, such as the service water intake structure, is at 18.1 ft. MSL.

Table 1.2-2
CLB 1 Square Mile PMP Rainfall Depths

Duration	Cumulative Depth (in)
5 min	6
15 min	9.5
30 min	13.7
1 hr	18.1
6 hr	27.5

Table 1.2-3
CLB Design Wave Forces(a)

Building	Wall	Design Wave Force (kips/ft) Static Load	Dynamic Load	Total
Power Block	East	7.00	39.80	46.80
	North	7.00	39.80	46.80
	South	7.00	39.80	46.80
	West	7.00	39.80	46.80
SWIS	East	7.00	39.80	46.80
	North	2.00	8.60	10.60
	South	13.50	77.00	90.50
	West	53.80	27.60	81.40

a) Reference 1-6, Table 2.4-11a

Table 1.2-4
CLB Waterborne Debris Spectrum[a]

Debris Type	Weight (lbs)	Contact Area	Impact Velocity (mph)
Telephone Pole	1490	13.5 in. diameter	20
Automobile	4000	20 ft^2 frontal area	20
House	4000	50 ft^2 frontal area	20
Boat	25,000	10 in. diameter	20

a) Reference 1-15

1.3 FLOOD-RELATED CHANGES AND FLOOD PROTECTION CHANGES

The plant design features and their functional requirements that provide protection against the design basis external flood mechanisms are provided in the UFSAR (Reference 1-6). The credited flood protection related attributes of the overall plant configuration that support the design for mitigation against external flooding have not changed from the time of initial licensing. Enhancements to procedural guidance supporting the implementation of protective actions against external flooding have been made over time. The HCGS Flood Protection Feature Inspections (References 1-13 and 1-17) found that the SGS flood protection active and passive features, e.g., walls, floors, roofs, penetration seals, doors, check valves, etc., were confirmed to be installed per design, functional, in good material condition, and appropriately controlled procedurally to ensure continued functionality. Changes to the hydrosphere around the PSEG Site and physical changes to the PSEG Site (e.g., security changes, buildings, etc.) are discussed in Section 1.4.

1.4 CHANGES TO THE WATERSHED AND LOCAL AREA

Local area changes have been minimal since plant operation began at the site. Both SGS units were in operation by the time of HCGS operation. Offsite areas within 5 miles of the plant to the east remain dominated by the open waters of Delaware Bay and low coastal wetlands to the east and west of the bay. Much of these coastal wetlands are under state ownership and managed as wildlife areas that are protected from future development. Additionally, most of the land on the New Jersey side within 2 miles of HCGS is owned by PSEG, the USACE, or the New Jersey Department of Environmental Protection. Most of the privately owned land within 5 miles is managed for agricultural production and/or private access hunting/fishing.

Although credited in the HCGS CLB, the proposed Tocks Island Dam was not constructed. As such, the HCGS CLB is extremely conservative with regard to the flood levels considered for single and multiple dam break scenarios.

The USACE is authorized by Congress (Water Resources Development Act of 1992, modified in 1996) to deepen the existing Delaware River Federal Navigation Channel from 40 ft. to 45 ft. from Philadelphia, PA, and Camden, NJ, to the mouth of the Delaware Bay, with appropriate bend widening. This project is partially completed with a target completion in 2017.

On site, major changes include the addition of the Materials Center, Low Level Radwaste, Nuclear Department Administration, Processing Center and Security Entrance buildings. Additionally, a security Vehicle Barrier System (VBS) has been added around the plant as well as the Independent Spent Fuel Storage Installation (ISFSI) storage area, which is inside the VBS and north of the HCGS Reactor Building. There have been no changes to site grade.

PSEG Power, LLC, and PSEG Nuclear, LLC (PSEG) submitted an ESP Application for a new plant located north of HCGS. The location and design of stormwater management systems for the new plant have not been determined, as discussed in the PSEG ESP Application. In general, the stormwater management system developed for new plant facilities will be integrated with the existing facilities. The new plant would modify the current site layout but the changes are not expected to impact the flooding behavior of the site.

1.5 CURRENT LICENSING BASIS FLOOD PROTECTION AND MITIGATION FEATURES

Multiple passive and active flood protection features are credited in the CLB. The passive features consist of the waterfront/shoreline protection, location, SGS, construction design/characteristics, waterproofing/waterstops and penetration seals. Active flood protection features include watertight doors, river water level monitor, leak detection sensors, and floor drainage system. Details of these features are provided in the References 1-13 and 1-17. The HCGS flood protection features are designed to be permanent and are therefore not dependent on flood duration.

Seismic Category I structures affected by design basis floods are designed to withstand the floods postulated in HCGS UFSAR Section 2.4. The hardened flood protection approach is used to incorporate structural provisions into the design of the plant for protection of safety-related structures, systems, and components from the combined static and dynamic effects of a flood.

Safety-related systems and components are not affected by a flood when they are located above the postulated maximum flood level. When located below flood level, these systems and components are enclosed in reinforced concrete Seismic Category I structures that have:

- Exterior wall thicknesses below flood level of not less than 2 ft.
- Waterstops provided in exterior wall construction joints and seismic separation joints below flood level.
- A minimum number of openings in exterior walls and slabs below flood level (these openings are designed to prevent intrusion of flood water).
- Water pressure tight doors installed in exterior walls below flood level.
- Exposed equipment hatches installed above flood level; those below flood level installed behind exterior walls designed to prevent intrusion of water. One exception to this condition is the exterior hatch located at grade level in the north Radwaste Building. This hatch is designed to be water pressure tight.
- Continuous waterproofing systems applied to the underside of base slabs and on exterior walls to grade.

Temporary passive and active flood protection features are not credited in the HCGS CLB.

1.6 ADDITIONAL SITE DETAIL

There are no additional site details.

1.7 REFERENCES

1-1 American Nuclear Society, An American National Standard Standards for Determining Design Basis Flooding at Power Reactor Sites, ANSI/ANS-2.8-1981, La Grange, IL, 1981.

1-2 C.L. Bretschneider, Hurricane Surge Predictions for Delaware Bay and River, Miscellaneous Paper No. 4-59, U.S. Army Corps of Engineers Beach Erosion Board, November 1959.

1-3 Coastal Engineering Research Center, Shore Protection Manual, Volumes I-III, U.S. Army Corps of Engineers, Fort Belvoir, VA, 1977.

1-4 D.R. Harleman, Tidal Dynamics in Estuaries: II, Real Estuaries, in "Estuary and Coastline Hydrodynamics," and Chapter 10 ed.T., 1966.

1-5 G. Marinos and J.W. Woodward, "Estimation of Hurricane Surge Hydrographs," " Journal of the Waterways and Harbors Division," American Society of Civil Engineers, Vol 94, No. WW2, 1968, pp 189-216.

1-6 PSEG Nuclear LLC, "Hope Creek Generating Station Updated Final Safety Analysis Report," Revision 19, 2012.

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2.0 FLOODING HAZARD REEVALUATION

Flooding hazards from various flood-causing mechanisms were evaluated for Hope Creek Generating Station (HCGS) in accordance with Enclosure 2 of the NRC's March 12, 2012, 50.54(f) Request for Information Letter, which identifies the requirements for the flooding hazard reevaluations associated with NTTF Recommendation 2.1. The flooding hazard reevaluation for HCGS follows, where appropriate, the hierarchical hazard assessment (HHA) process described in NUREG/CR-7046. As explained in Attachment 1 to Enclosure 2 of the NRC's 50.54(f) letter, HHA is a progressively refined, stepwise estimation of the site-specific hazards that evaluates the safety of the site with the most conservative, plausible assumptions consistent with available data. Consistent with the HHA approach, flooding mechanisms that are determined to not be the controlling factors for external flood hazards will be screened out using order-of-magnitude analysis or qualitative assessments, where appropriate, with conservative assumptions and physical reasoning based on the physical, hydrological and geological settings of the site. The flooding hazards that can potentially affect the PSEG Site are well understood and in cases where it is known external flooding can exceed site grade (e.g., local intense precipitation and storm surge) a detailed analysis was undertaken without progressing through the stepwise HHA approach.

The HCGS flooding reevaluation applies the flooding hazard analysis approaches, regulatory guidance, and methodologies used in support of the preparation of the PSEG Site Early Site Permit Application (ESPA) Site Safety Analysis Report (SSAR) for a future unit at the site, which are also augmented by recent regulatory guidance. The principal regulatory guidance related to flooding hazard evaluations include:

- Regulatory Guides 1.59, 1.102, and 1.206
- Standard Review Plan (NUREG-0800) Sections 2.4.1 to 2.4.7 and 2.4.9 to 2.4.10
- NUREG/CR-7046
- NUREG/CR-7134
- NUREG/CR-6966
- ANSI/ANS-2.8-1992
- JLD-ISG-2012-06
- JLD-ISG-2013-01

PSEG submitted an application for an ESP for the PSEG Site on May 25, 2010. The application is revised annually and is currently under NRC review. As part of this review, in October 2012 NRC requested PSEG reevaluate their storm surge hazard in light of the Fukushima events. The PSEG response to this request, submitted in November of 2013, forms the basis for Section 2.4 of this report. The flooding causal mechanisms and design basis flood elevation described in the PSEG Site ESPA SSAR are applicable to the flood reevaluation for HCGS because HCGS and the PSEG Site are located physically adjacent to each other and share a common site. The analyses described here are performed for the entire site, including HCGS, as the flooding events impact the entire site area. Throughout this report the term PSEG Site is defined to mean the entire property, including SGS, HCGS and the potential new plant.

This chapter describes in detail the reevaluation effort for each plausible flooding mechanism and the potential impacts to the safety-related SSCs of the plant: flooding impacts due to local intense precipitation (Section 2.1), flooding in streams and rivers (Section 2.2), dam breaches and failures (Section 2.3), storm surge (Section 2.4), seiche (Section 2.5) tsunami (Section 2.6), ice induced flooding (Section 2.7), channel migration or diversion (Section 2.8), combined flood effect (Section 2.9), and associated effects of flooding (Section 2.10).

2.1 LOCAL INTENSE PRECIPITATION

Local Intense Precipitation (LIP) is the measure of the extreme precipitation (high intensity/short duration) at a given location. Generally, for smaller basin areas, shorter storm durations produce the most critical runoff scenario as the amount of extreme precipitation decreases with increasing duration and increasing area. Also, for small areas, short times of concentration result in high intensity rainfall which creates a larger peak runoff. Therefore, the shorter storm over a small watershed will result in higher flow rates for the HCGS LIP analysis. The LIP analysis prepared for HCGS is part of an overall analysis prepared for the PSEG Site, including the entire property on which SGS and HCGS are co-located.

2.1.1 LIP Intensity and Distribution

As prescribed in NUREG/CR-7046 (Reference 2.1-1), the LIP used in the analysis is the 1-hour, 1-square mile probable maximum precipitation (PMP) at the HCGS site location. Parameters to estimate the local intense precipitation are from the Hydrometeorological Report 52 (HMR-52, Reference 2.1-2). Point rainfall (1-square mile) LIP depths for durations of one hour and less are determined using the charts provided in HMR-52. HMR-52 is used to determine the 1-hour duration LIP estimates based on the location of the drainage basin. Using Figure 24 in HMR-52 and the site location, the 1-hour, 1-square mile precipitation depth estimate is 18.1 inches. HMR-52 Figures 36, 37, and 38 are used to estimate the 5-minute, 15-minute, and 30-minute 1-square mile precipitation depths. The LIP depths for the site are presented in Table 2.1-1.

A cumulative rainfall distribution curve is then plotted from the 5-minute, 15-minute, 30-minute, 1-hour 1-square mile precipitation depths as shown on Figure 2.1-1. A synthetic hyetograph for the 1-hour LIP is developed from cumulative precipitation depths shown in Table 2.1-1, in accordance with methodology presented in NUREG/CR-7046. A five-minute time step is used such that the most intense 5-minute interval is placed at the beginning of the distribution, and then the successively diminishing depth intervals are placed on after the initial high point of the distribution. The rainfall distribution synthetic hyetograph for the 1-hour (60-minute) LIP is presented graphically in Figure 2.1-2.

2.1.2 LIP Model Development

Two-dimensional modeling is often the most accurate approach for assessing hydrologic and hydraulic conditions in areas where flood volume dictates the area of inundation or when flow is unconfined with a high degree of flow path uncertainty. It is appropriate for this LIP analysis, which is characterized by shallow, overland flow in a developed area with walls, building obstructions, and flow path uncertainty resulting from a LIP event.

FLO-2D (Version 2009.06, Reference 2.1-3) is used to conduct the LIP analysis. FLO-2D is a process-based physical model that routes rainfall-runoff and flood hydrographs over unconfined flow surfaces or in channels using the fully dynamic wave approximation to the momentum equation. It has a number of components that makes it capable of simulating sheet flow, obstructions, sediment transport, spatially variable rainfall, infiltration, floodways, and many other flooding details. Predicted flow depth and velocity between the grid elements represent average hydraulic flow conditions computed for a small time-step (on the order of seconds).

Typical applications have grid elements that range from 10 ft. to 500 ft. on a side and the number of grid elements is unlimited (Reference 2.1-3).

The PSEG Site topography is based on the recent 2008 site survey (Reference 2.1-5). A walkdown of the PSEG Site in 2013 further refined the 2008 data to document any significant changes to structures or buildings since the topography data was acquired. The topographic mapping was created using Light Detection Radar (LiDAR) with a resolution of 1-foot. The 1-foot contour dataset is used to establish grid elevations in the model. Several data processing steps are performed using ArcGIS to convert the contour data into a representative surface in FLO-2D. First, the contour dataset is converted to a triangulated-irregular-network (TIN) to represent the bare ground surface of the site using 3D Analyst Tools in ArcGIS. The TIN is then converted to an ASCII raster dataset with a resolution of 5 x 5 feet. The ASCII raster format is fully compatible with the FLO-2D model. Using the ASCII raster, an average grid cell elevation was computed and assigned to each grid cell in the model (i.e., one elevation value per grid cell to represent the average ground elevation in that cell).

Once the initial grid elevations were established, the grid cell elevations within many of the building footprints were raised to a nominal roof elevation, well above potential LIP flood levels, to represent an obstruction to overland flow. This was done for large and small buildings that are located near critical doors and buildings or directly in critical LIP flow paths. Small buildings located far away from the area of interest, on high ground above LIP flow paths, or upstream of the critical plant areas were not represented in the model as obstructions to flow, and no adjustments were made to the ground elevation at these locations.

The PSEG Site FLO-2D model domain is presented as Figure 2.1-3. Figure 2.1-3 presents the location of all of the buildings that were represented in the FLO-2D model as obstructions to LIP runoff.

Floodplain outflow grid elements were established in FLO-2D around the entire model domain. These outflow grid elements totally remove flow volume from the model. The west and south model boundaries are located along the Delaware River and Delaware Bay, while the north and east boundaries were established at a buffer distance beyond the watershed boundary impacting the site. The boundary condition considered at the outflow grid elements in the FLO-2D model is based on normal depth. This is appropriate for the LIP analysis since the surrounding topography directs runoff away from the site beyond the model domain, and the normal depth boundary condition will not be affected by water levels and flow conditions downstream of the model boundary. Additionally, the site is elevated above the Delaware River and the Delaware Bay, and there are no combined events criteria that require consideration of elevated flood levels in adjacent bodies of water coincident with the LIP event. Thus, the water levels resulting from the LIP analysis will not be impacted by backwater from the Delaware River and Delaware Bay. Refer to Figure 2.1-3 for an overview of the model domain and outflow boundary.

2.1.2.1 Surface Infiltration

Per NUREG/CR-7046 recommendations, the following assumptions were applied to the LIP analysis:

- Runoff losses were ignored during the local intense precipitation event (i.e., no infiltration) in order to maximize the runoff and resulting flood elevations from the event.
- Conservatively, all active and passive drainage system components on the site were considered nonfunctional or clogged for the analysis of LIP flooding. The model is based primarily on overland flow over the whole plant site and does not credit any flow through gravity storm drain systems, roof drains, pipe networks, culverts, etc.

2.1.2.2 Surface Roughness

Manning's roughness coefficients (n-values) are assigned to the model grid cells based primarily on aerial imagery and site survey described in Subsection 2.1.2. The Manning's n-values and classifications were verified based on local land cover and land use observed during the on-site field investigation. The land use classifications and Manning's n-values are shown in Table 2.1-2, which are based on guidance from the FLO-2D Reference Manual (Reference 2.1-3).

The n-values used in the FLO-2D model are different than n-values used for steady, uniform flow in prismatic channels. The n-values in a FLO-2D model account for two dimensional flow, vegetation, surface irregularity, and non-uniform, unsteady flow. All three of the land use categories in Table 2.1-2 have been assigned Manning's n-values that are on the lower end of the range provided in the FLO-2D reference manual. This is primarily due to observations during the site visit and the fact that the site is developed and actively maintained, rather than being in a naturally vegetated or undeveloped condition.

The grass near critical buildings is short and only located in a few isolated locations. Grass and vegetation located far away from critical buildings is not essential in the analysis or results. Developed/paved areas are highly compacted and covered with gravel or pavement/concrete. Since gravel has a higher roughness coefficient than concrete, an average Manning's n-value was used from the range of values. The debris/obstruction land use category was used in a relatively small area at the southern end of the site at the Salem barge slip area to represent obstructions that were identified in this area during the site visit, such as jersey barriers, fencing, large boulders, etc. Since there were some gaps and openings where flow can still be conveyed, a lower n-value in the range of values was selected.

It is important to note that the area of inundation and flood levels in a FLO-2D model are much more dependent on the volume of the rainfall/runoff rather than on the peak discharge or Manning's n-values. Thus, the model is relatively insensitive to slight variations in Manning's n. The Manning's n-value classifications in the FLO-2D model domain are shown on Figure 2.1-4.

2.1.2.3 Obstructions and Impediments to Flow

As discussed in Subsection 2.1.2, the buildings in the model domain were represented by raising the grid cell elevation within the building footprint. Rainfall was applied to these building locations and runoff was allowed to flow off the roof onto adjacent ground. Storage on top of the roofs and routing through roof drainage systems is conservatively neglected, and runoff from building roofs contributes directly to overland flow adjacent to the buildings. The elevated grid cells prevent overland flows from being routed "through" the buildings.

Area and width reduction factors (ARFs, and WRFs, respectively) were used to represent the HCGS cooling tower location in the model. These reduction factors were appropriate in the cooling tower area since rainfall in this area will not contribute to overland flow because precipitation falls directly into the cooling tower basin. A grid cell can be assigned an ARF value from 0 to 1, with 0 representing no blockage and 1 representing a grid cell that is completely blocked and removed from receiving any inflow. Similarly, any of the eight flow directions in FLO-2D can be assigned a WRF value from 0 to 1, with 1 representing a flow direction that is completely blocked by a flow obstruction. All grid cells in the cooling tower area were completely blocked out from the model domain (i.e., ARFs and WRFs = 1).

Levee elements were used in the FLO-2D model to represent the drainage impacts of the Vehicle Barrier System (VBS) and seawalls. The alignment of the VBS and seawalls were derived from the site survey. These alignments were imported into the FLO-2D model and a levee was created to represent the walls. The VBS consists of concrete blocks approximately 10 ft. wide by 3.5 ft. tall. The levee crest elevation along the VBS was set to be a constant 3.5 ft. above grade. The VBS restricts overland flow and is located in various locations around the perimeter of the Salem and Hope Creek Generating Stations. There are limited openings along the VBS to allow for pedestrian access and drainage. Gaps in the VBS were also added to the model to represent the opening widths shown on the site survey.

The seawalls located near the SGS Service Water Intake Structure and Circulating Water Intake Structure at the southwestern corner of the site are also added to the model as levees to represent the blocked flow paths in this area. The elevation of the top of the seawalls is derived directly from the site survey. WRF values were applied along the VBS levee in the model. This is done to accurately represent the levee crest length that is used in overtopping weir calculations. Since the levee alignment in FLO-2D is based on octagonal sides, if a physical levee is a straight line across a grid element then the weir length represented by octagon sides in FLO-2D could be too long resulting in higher overtopping flows. This was corrected by computing the entire length of the actual VBS levee and then comparing it with the octagonal levee length in FLO-2D, which resulted in a WRF value of 0.21. The addition of the WRFs to the model is fairly insignificant, since there is minimal levee overtopping in the model results. Refer to Figure 2.1-3 for a detailed overview of the site and the FLO-2D model input (e.g., model boundaries, 1 ft. contours, building footprints, VBS, seawalls, critical door locations, etc.).

2.1.3 LIP Simulation

FLO-2D is a dynamic, two-dimensional hydrologic/hydraulic flood routing model that conserves volume as it routes hydrographs in 8 flow directions over a system of square grid elements. The model routes runoff over the grid using the full dynamic wave momentum equation and a central finite difference routing scheme. The floodwave progression is affected by the surface topography and roughness values (Manning's n-values) associated with land use characteristics.

After the FLO-2D model is developed, the model is run with the PMP rainfall event starting at time zero and following the hyetograph discussed in Subsection 2.1.1. The model simulation is run for a total of 12 hours to ensure that the maximum flood depths/elevations are captured and the duration of the flooding event established. The model output interval is set to 0.1 hours for the depth, flow, and velocity results. The default Courant Number of 0.6 is used to govern the numerical stability of the model.